Geotechnical Site Characterization

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VOLUME 1
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Published by
A. A. Balkema, P.O. Box 1675, 3000 DR Rotterdam, Netherlands (Fax: +31.10.413.5947)
A. A. Balkema Publishers, Old Post Road, Brookfield, VT 05036-9704, USA (Fax: 802.276.3837)

For the complete set of two volumes, ISBN 90 5410 939 4
For Volume 1, ISBN 90 5410 940 8
For Volume 2, ISBN 90 5410 941 6

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Printed in the Netherlands
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Introduction

In the past 25 years there have been many interesting and informative conferences on topics related to site characterization for geotechnical engineering concerns. In addition to conventional drilling and sampling techniques, there have emerged a number of new in-situ tests, including the cone penetration test, piezocene, flat dilatometer, pressuremeter, and other specialty probes, blades, and push-in devices. New drilling and sampling methods have been introduced, including sonic drilling, continuous sampling, combined rotary/hydraulic/percussive tools, as well as automated devices. A marked increase in the utilization of geophysical techniques has also occurred, notably the profiling of shear wave velocities, ground penetrating radar, resistivity, and other nondestructive measurements. As a consequence, a number of successful symposia were held on penetration testing (ESOP1-1, 1974; CPT’81; ESOP2, 1982; ESOP1, 1988; CPT’95), conferences on in-situ testing (In-Situ’86; PTUK, 1988), symposia on pressuremeter (ISP-1, 1982; ISP-2, 1986; ISP-3, 1990; ISP-4, 1995), vane shear testing (ASTM STP 1014, 1987), chamber testing (ISOCCT-1, 1991), geophysical testing (STP 654, 1978; STP 1213, 1994), and others (see list to this introduction).

These prior conferences have set high standards in the communication of research and practice related to site characterization and in-situ testing. The members of the Technical Committee (TC 16) on Ground Property Characterization from In-situ Testing as part of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) felt that it was appropriate for a major international conference that would link all the different areas related to geotechnical site characterization. Hence, with the assistance of ISSMGE TC 10 (Geophysical Site Characterization) and TC 24 (Soil Sampling) the concept of the First International Conference on Site Characterization (ISC’98) was born.

The objectives of ISC’98 are to provide a forum for discussion on all aspects of geotechnical and geoenvironmental site characterization including: planning, specification, drilling, sampling, in-situ testing and geophysical testing. Particular emphasis has been placed on the exchange of practical experience and the application of research results. An aim of this conference has been to enhance the exchange of knowledge between researchers and practitioners from countries all over the world and to facilitate interaction between experienced and younger geotechnical engineers and engineering geologists. A further theme of the conference has been the exchange of novel and innovative ideas, new technologies, and practical applications related to geotechnical and geoenvironmental site characterization.

The conference proceedings have been arranged into sessions that cover: Objectives, scope and planning for site characterization; Geophysical testing and remote sensing; Geoenvironmental testing; Deformation and in-situ stress testing; Penetration testing; and Advanced interpretation of field tests. The Technical Program also comprises six Theme Lectures by eminent international experts in the main areas of site characterization. Discussion Sessions on each main Theme include presentations by leading experts as Discussion Leaders and selected presentations from authors of papers. Poster Sessions, a Technical Exhibition, and a Field Demonstration Session have been
arranged to allow participants an opportunity to see and discuss new techniques in greater detail.

The Proceedings also contain a report entitled Pressuremeter Testing in Onshore Investigations prepared by B.G. Clarke and M. Garnbin for TC 16. This report represents the first in an expected series of reports on major in-situ test methods. The objective of the report is to describe pressuremeter equipment, site operations, interpretations and applications to guide practitioners in the use and application of pressuremeters.

The Conference would not have been possible without the dedicated work and competence of the many authors who have submitted papers and the hard work and enthusiasm of the many individuals and organizations and companies that have provided the basis for the planning and successful implementation of ISC'98.

We hope that you enjoy reading and assimilating the 206 papers in these two-volume proceedings and find use for these technical publications in your daily practice and research.

Peter K. Robertson  
University of Alberta

Paul W. Mayne  
Georgia Institute of Technology

References

(Conferences related to Site Characterization, given in chronological order):


Cone Penetration Testing and Experience (1981), Sessions Proceedings at ASCE National Convention, St. Louis, Missouri; published by American Society of Civil Engineers, Reston, Virginia (CPT® 81).

European Symposium on Penetration Testing (ESOPT-2, 1982), Amsterdam, The Netherlands; published as Penetration Testing, A.A. Balkema, Rotterdam.


Use of In-Situ Tests in Geotechnical Engineering (In-Situ '86), Blacksburg, Virginia, USA; published as Geotechnical Specialty Publication (GSP) No. 56, American Society of Civil Engineers, Reston, Virginia, 1986.


Fourth International Symposium on Pressuremeters (ISP4, 1995) Sherbrooke, Quebec, Canada; published as The Pressuremeter and Its New Avenues by A.A. Balkema, Rotterdam.


XIV
Theme lectures
WAC Bennett Dam — The characterization of a crest sinkhole

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ABSTRACT: In June 1996, a small cavity was discovered on the crest of the 183 m high WAC Bennett Dam in British Columbia, Canada. Subsequent drilling to investigate the condition of the dam core below the cavity resulted in a sinkhole on the dam crest with a surface expression 2.5 m in diameter and 7 m in depth. Following this incident the safety status of this very large dam was uncertain. A comprehensive investigation program was urgently planned and executed to characterize the sinkhole. This paper describes the objectives and criteria developed for the program, the scope of the key activities at the sinkhole, and some important lessons learned during the investigation.

1. INTRODUCTION

Site investigation and characterization are among the first steps in determining a project’s feasibility and cost. In areas where the ground conditions and the performance of structures are well known, site characterization may be routine. For greenfield sites with poor ground conditions and sensitive new facilities, foundation design and site development may be a significant factor in the overall project cost and viability. In many cases a significant component of construction risk may be site development itself. Performance and design requirements are defined by an owner or owner’s representative in concert with his engineering team. Site investigation methods are selected to match these requirements, usually in the context of competitive bidding. The objective is to obtain all geotechnical information necessary for design, construction, and dispute resolution.

Due to the inherent variability of ground conditions, the most challenging aspect of site characterization can be determining what constitutes adequate investigation coverage. In effect, the development of the scope of a site investigation is an exercise in risk management, generally done intuitively by suitably experienced practitioners. The scope of the investigation is balanced among existing information, local knowledge, anticipated variation in ground conditions, sensitivity of the structures to variable conditions, the performance requirements, consequences of poor performance, and the expected or available budget. On large projects with considerable risk (probability of failure \( \times \) consequence), decision analyses and risk management techniques have been applied to site investigation design (Vick & Broonwell 1989).

Another type of site characterization is the post-construction foundation investigation of structures subject to distress. The difficulties of this type of work are often compounded by the spatial and logistical restrictions of working in an operating facility. The case history described in this paper deals with the investigation of a sinkhole in a very large dam under an operating reservoir about 170 m deep. The overriding objective was to determine the safety of the dam.

At the start of the investigation, it was not known whether the disturbed core of the dam was confined to the zone directly below the sinkhole or resulted from a more pervasive mechanism for which the collapse at the surface was but an early warning. In the latter case, drilling to understand the safety of the dam could actually reduce safety through hydraulic fracturing and/or erosion collapse. Central to all investigation planning and execution was the minimization of risk due to drilling.

Although the objective was clear, planning was complicated by the necessity to immediately understand the dam safety status, by limitations in the state of practice in drilling technologies able to investigate a dam core under full reservoir, and by an uncertain scope. This paper describes planning issues and lessons learned during this challenging project. There is also limited discussion of the investigation techniques and results.
2. 1996 WAC BENNETT DAM INCIDENT

On June 14, 1996 a passing motorist discovered a 450 mm diameter hole in the asphaltic concrete road surface on the crest of Bennett Dam. Examination of the hole within an hour of its discovery revealed a cavity with a volume of 2 m$^3$ and a maximum depth of 2 m extending beneath the crest pavement. The top of a vertical steel pipe (later determined to be a construction survey benchmark tube) was barely visible at the base of the cavity. The following day a careful excavation to 5 m using a backhoe confirmed the presence of the survey benchmark tube (see Figure 1), and established that the cavity was not evident below 2 m. The benchmark tube comprised concentric 65 mm inner and 150 mm outer thin walled steel pipes; the outer pipe was designed to act as a sectional sleeve to protect the inner survey pipe.

The existence of the buried benchmark tube was not known to the surveillance engineers who regularly inspected the dam, nor did it appear on any readily available construction drawings. It was later determined that the benchmark tube extended from the rock foundation at a depth of 115 m to the dam crest.

Our immediate assessment speculated that the cavity was likely associated in some as yet undetermined way with the survey benchmark tube. The volume of the surface cavity was similar to the open annular space between the concentric pipes over their 115 m depth to bedrock. This volume balance provided a plausible early working hypothesis for the cavity development. That is, over the 24 years since reservoir filling, core material had slowly migrated into the annular space between the pipes. In an attempt to evaluate this initial hypothesis, it was decided to investigate the condition of the dam core at depth around the benchmark tube using a Becker Penetration Test (BPT).

On June 17, 1996 the first BPT hole was drilled within 1.5 m of the survey benchmark tube, using a close-ended 140 mm OD pipe, driven by a single acting diesel hammer. This pipe advanced to 32 m with virtually no resistance. While drilling was suspended to attach additional pipe at 32 m the dam crest surrounding the survey benchmark suddenly dropped 7 m, leaving a cylindrical, vertical sided hole 2.5 m in diameter (see Figure 2). Coincident with the collapse, the water level in an open "leaky" inclinometer casing (Observation well OW-5) 50 m away rose suddenly by more than 2.5 m. This unexpected local crest collapse constitutes the 1996 incident at Bennett Dam.

In early July a second survey benchmark tube, and an associated second more subtle sinkhole, was discovered in the upstream shell overlying the core just upstream of the dam crest.
damage to the core, and to assess alternatives for rehabilitation as required.

The collapse occurred during the spring freshet as the reservoir was rising to its maximum pool elevation. In consultation with the Office of the
Controller of Water Rights, British Columbia's
dam safety regulator, and the Advisory Board (Drs.
R.D. Peck and N.R. Morgenstern) the decision was
made to lower the reservoir. Intensive investigations
of the core were delayed until freeboard was
increased. This would improve the safety margin in the
event that another crest collapse occurred. On
June 22, BC Hydro opened the spillway gates.
Discharges through the spillway and powerhouse, at
a maximum rate of 5100 m³/s, lasted eight weeks.
Over this period the reservoir drawdown was less
than 2 m.

During reservoir drawdown, various surface
geophysical techniques were employed to investigate
the condition of the dam beyond the sinkholes.
During this time the intrusive investigations at the
sinkholes were planned. Part of this planning
involved trial drilling and downhole geophysical
surveys in intact portions of the core at locations
distant from the sinkhole.

During the early investigation stages, the
uncertain safety status of the dam drove the urgency
to complete the work. As the investigation
proceeded, and the safety of the dam was better
understood, the need to complete the investigations
was driven by the requirement to remediate the dam
so that the entire 1997 spring freshet could be
captured. The investigations were planned to
accommodate these goals, incorporating dam safety
risk mitigation measures to the extent possible.

3. INVESTIGATION OBJECTIVES AND
CRITERIA

The objectives which formed the cornerstone of the
investigation were to:
• characterize the extent and nature of the
disturbed core beneath the sinkhole(s);
• establish whether the loose zones and their
cause constituted a dam safety hazard;
• gather sufficient information for remediation
design and construction; and
• complete the investigations before the harsh
winter to permit return of the reservoir to full
service.

Although the scope of the work could not be
fully defined, the planning was facilitated by the
delay needed to meet the requirement for additional
freeboard. This investigation delay of 6 weeks was
used to develop the criteria and protocols, and to
determine the test equipment and techniques. All
The investigation planning was done at site by a resident team of senior personnel brought together from BC Hydro and consulting companies across Canada. Experts in specific investigation techniques were retained as needed.

Key investigation criteria were as follows:
- minimize all dam safety risks during drilling;
- maximize the use of surface geophysical (non-intrusive) techniques to examine at least the upper part of the dam;
- use proven technology to drill at the sinkhole, with modifications to minimize core damage;
- use the least intrusive techniques for investigating the sinkholes;
- develop the program incrementally, building on the early results;
- access the global experience of others who have investigated similar problems;
- minimize the number of drill holes, recognizing that the information gathered must be more valuable than the potential for immediate or long term damage to the core;
- maximize the amount of information gathered at each drill hole;
- drill 24 hours/day and use multiple drills where practicable;
- assess drilling data as obtained and adapt the program as conditions become better understood;
- test any unproved methods under conditions similar to those expected at the dam;
- develop safety and surveillance protocols, both for personnel and emergency response (e.g., heavy mechanical equipment and emergency grouting equipment to be in "ready" mode on the dam crest);
- develop detailed drilling procedures, maintain a continuous presence of an experienced senior engineer on crest for "first response", and develop detailed response plans for all foreseeable contingencies.

The investigation plan incorporated all objectives and criteria noted above. The following sections discuss some individual components of the investigation program, particularly identifying planning considerations and lessons learned. This paper describes the investigation at Sinkhole No. 1. Investigations at Sinkhole No. 2 and beyond the sinkholes are not discussed.

4. CONE PENETRATION TESTING

4.1 Selection of Method and Objectives

The seismic piezocene penetration test (SCPTU) was the principal investigation technique used to define the characteristics and extent of the disturbed dam core beneath the sinkhole. The SCPTU was selected because, of all intrusive investigation techniques available, it yields the most information with the least disturbance. It is also a very controlled, sensitive penetration method with real time data output.

While considering the seismic piezocone, we were concerned about whether it could be pushed into the core at all, because of the likely presence of inter-layered loose and dense materials. Piezocones, or for that matter cone penetrometers of any kind, cannot be pushed into well compacted soils.

The basis for believing that the piezocone could penetrate the core came from the minimal resistance experienced by the Becker Penetration Test at the sinkhole. We were also aware of dynamic cone penetration tests of the backfill materials around vertical riser pipes in the glacial till core at the LG4 development in Quebec (Bieniawski et al. 1989).

The SCPTU program objectives were to determine the depth and extent of the disturbed zones, and to gather as much data as possible from each SCPTU profile.
4.2 Description of SCPTU Equipment and Procedures

The piezocene included transducers for measuring tip resistance, sleeve friction, pore pressure and temperature. It also included a bi-axial geophone for measuring compression wave/shear wave velocity.

The equipment and penetration procedures followed ISSMUPE (1989) standards, except that the tip area was 15 cm² instead of 10 cm² and the penetration rate was generally slower than 2 cm/s. A larger tip was used because of the potential need for greater sensitivity in very loose core. A slower penetration rate was used because of concern that the cone rods would buckle on dense material before the hydraulic ram could be shut down. Both concerns proved to be valid.

After collaring each SCPTU hole (Sonic, Barber, and mud rotary rigs were used for this purpose) initial drilling was carried out by a Simco 5000 midi rotary drill rig to reach the test level. Procedures for drilling to support the SCPTU tests were similar to those described in Section 6 of this paper.

The piezocene was pushed with a hydraulic ram fitted to the Simco 5000 drill rig. Before cone testing, HWL rods and BWL rods were lowered inside the HW casing to laterally support the cone rods. The piezocene was then lowered to the bottom of the casing and pushed into the soil. Testing continued until:

- the piezocene refused in a stiff zone, or
- bending in the piezocene rods became excessive (due to a long, laterally unsupported section beyond the casing in soft ground).

The piezocene was then removed from the hole, the casing was advanced, the hole cleaned out, and testing continued. Because of friction buildup, the casing had to be reeled several times to reach target depths.

Before resumption of SCPTU testing in uncertain ground conditions, a “dummy” cone mounted on BWL rods was quickly lowered to the bottom of the casing and pushed into the soil. On several occasions the dummy cone encountered stiff ground. This triad procedure saved time in lowering and then removing the 1 in threaded SCPTU rods.

Every 1 or 2 metres, piezocene testing was paused to allow pore pressures and temperatures to equilibrate around the tip, and to carry out downhole seismic velocity testing. Seismic “shots” were made by discharging a heavy gauge shotgun into a prepared hole. The shot pattern varied, but generally included locations on four sides of the SCPTU hole at several prescribed offsets.

4.3 SCPTU Test Program and Results

Six SCPTUs were completed at Sirkhole No. 1. Typical results are shown on the following figures:

- Figure 6 shows the location of selected drill holes at Sirkhole No. 1.
- Figure 7 shows the standard test results (tip resistance, shaft friction, friction ratio, and dynamic and equilibrium pore pressures) for the first SCPTU hole in Sirkhole No. 1.
- Figure 8 is a section showing piezocene tip resistance with depth for a number of SCPTUs at varying distances from the centre of Sirkhole No. 1.

Note that in the first SCPTU the tip resistance is remarkably low to a depth of 33.5 m, with values approaching the equilibrium pore water pressures. The cone rods essentially fell under their own weight to 33.5 m. The low tip resistance was first thought to be due to low density but subsequent testing showed that it was primarily a function of very low stresses in the disturbed zone. Below the low tip resistance zone is dam core with rapidly varying density. The characterization of the dam core is described in a later section.

4.4 Lessons Learned

The first SCPTU test was carried out, of course, without knowledge of the general conditions below the sinkhole. There was much debate about whether there would be a repeat of core collapse and/or pore pressure response in adjacent piezocene. Consequently, an attempt was made to anticipate all possible dam responses, and to develop procedures and contingency plans to address these. However, when venturing into unknown ground conditions, there is always much to be learned.

We recognized at the beginning of the program that the piezocene investigation had an inherent limitation. Because the piezocene cannot be pushed through compacted soil, the location of the loose zone must be estimated a priori. The piezocene is therefore a characterization tool, not a general investigation tool in this situation.

Normal methods of interpreting SCPTU data (Lunne et al. 1997) are based on measured parameters and some knowledge of in situ stress state. In this case, in situ stresses were extremely low. Since this was not known at the start of the program, interpretations of soil type using SCPTU results were misleading. Fortunately, soil classification was not an issue, as material type was well known. On other projects involving unexpected and unusual stress conditions, this issue could be more important.
The "dummy" cone proved to be a valuable tool for proving out ground after drillholes because drill rods can be lowered much faster than SCPTU rods. Drilling fluid was routinely lost when cleaning out the inside of the casing, as the drill bit approached the bottom of the casing. This issue is discussed later.

Despite the above shortcomings, the SCPTU exceeded our expectations in providing comprehensive ground information quickly and, most importantly, without reducing the safety of the dam. Indeed, the success of the SCPTU program permitted the use of more intrusive investigation techniques at the sinkhole.

5. SONIC DRILLING

5.1 Objectives

In addition to the SCPTU program, we recognized that it would be necessary to drill holes into the dam to probe the extent of the disturbed core, to define pore pressures, and to install instruments of various types around the sinkhole. However, the spectre of damaging the core by the drilling itself caused great concern, and resulted in a considerable planning effort to select a suitable drill rig and drilling technique. The drilling method would have to be capable of investigating intact core and any zones of intermediate disturbance where the SCPTU could not be pushed.

Listed approximately in order of decreasing priority, the drilling method should:

1. Not damage the dam core in any significant way. In particular, it should not cause hydraulic fracturing under conditions of low total stress and/or low pore pressures which might exist in the dam core.
2. Not trigger collapse of loose zones (as occurred with the Becker rig).
3. Be capable of drilling to depths in excess of 120 m.
4. Allow recovery of soil samples for logging and laboratory tests. As a minimum this should include standard classification tests and preservation
5. Allow measurement of in situ density.
6. Permit installation of piezometers and casings (for later geophysical testing).
7. Allow proper completion of the hole, including grouting and sealing to prevent migration of water vertically through the drilled zone.

5.2 Selection of Drilling Method

A drilling method that could meet the above criteria equally well would be a rare find, but the main concern was meeting the foremost criterion, that of not damaging the core. This seemed to rule out standard techniques that employed drilling fluids. Air-drilling methods had the greatest potential to cause hydraulic fracturing, but methods using water and drill mud were also undesirable.

Hydro Quebec was in the process of drilling several relatively shallow experimental holes into dams with similar cores using very heavy drill mud. The benefit of the heavy mud appears to be lack of mobility along fracture planes despite the increased likelihood that heavy mud will cause hydraulic fracturing compared to water or air. However, this technique was in the research stage at shallow depths, and thus not considered suitable for immediate use at Snettisham Dam.

Drilling methods that drive a casing into the ground were also considered unsuitable, mainly because of the energy required and because they were too slow or would not reach the necessary depths. This included the Becker hammer drill and cable tool drilling.

One drilling method met most of the criteria given earlier. This method, called “sonic” drilling, had been successfully used to similar large depths in the Fraser River delta. It is used extensively in environmental work and had been used previously by BC Hydro in seismic assessments in granular materials in dams. Reviewing past performance of the sonic drill, it appeared capable of meeting most of the objectives stated earlier. The exceptions:

- The objective of not triggering collapse in a moderate to large loose core zone was probably not achievable with a rig that had sufficient power to drill to the desired depth and that transferred significant vibration into the ground.
The continuous sample recovered by a sonic drill is slightly disturbed and thus it is not possible to directly measure density of the core.

5.3 Description of Sonic Drilling

The sonic drill is truck-mounted and accompanied by a support truck which carries casing and drill rods (Figure 9). The rig has a hydraulic head with an oscillator that generates and transmits a cyclic axial force down the drill rods to the core barrel and drill bit (Figure 10). The vibrations at the bit loosen moist soil and liquefy saturated soil, allowing the core barrel to penetrate the ground. The sonic drill alternately advances the core barrel and then the casing, normally in 3 or 6 m sections. The equipment is not wireline, consequently the drill rods and core barrel must be removed from the hole to retrieve the sample. Holes are usually started with a large casing and core barrel, and then telescoped down in size as friction builds up on the casing. Table 1 shows typical rod sizes.

<table>
<thead>
<tr>
<th>Item</th>
<th>Nominal Size (mm)</th>
<th>Inside Diameter, ID (mm)</th>
<th>Outside Diameter, OD (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Barrels</td>
<td>115</td>
<td>96.2</td>
<td>114.5</td>
</tr>
<tr>
<td></td>
<td>165</td>
<td>142.4</td>
<td>161.9</td>
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<tr>
<td>Drill Rods</td>
<td>90</td>
<td>76.0</td>
<td>89.8</td>
</tr>
<tr>
<td>Drill</td>
<td>140</td>
<td>122.8</td>
<td>146.3</td>
</tr>
<tr>
<td>Casings</td>
<td>190</td>
<td>174.6</td>
<td>193.7</td>
</tr>
<tr>
<td></td>
<td>220</td>
<td>203.2</td>
<td>219.1</td>
</tr>
</tbody>
</table>
core before drilling near the sinkholes. The first sonic hole was drilled in early July 1996, on the right abutment of the dam, to a depth of 68 m. No problems were encountered in this hole. The next two sonic holes on the crest were drilled in mid- and late July 1996, in the terrace section (Figure 4), to depths of 120 m and 123 m. In the first of these holes, the 140 mm OD casing stuck at a depth of 117 m but was recovered with a 40-tonne jack. While drilling in the rock foundation, the core barrel stuck and was eventually lost. The second hole was drilled without incident.

Even though problems were encountered, the prototype drilling was successful in that it identified the need to use at least 3 sizes of casing when drilling to 120 m depth, and allowed development of drilling, sample recovery, and hole completion techniques while still drilling in non-critical areas of the dam. It also provided the opportunity to test crosshole tomography techniques.

5.5 Production Drilling

Thirteen additional sonic holes were drilled from the crest of the dam into the core. Most of these were about 120 m deep, but two holes drilled in the canyon section extended deeper, one to a record depth of 140 m. This was considered to be the depth limit in the core using three sets of casing. The depth from the dam crest to base in the deepest part of the canyon is 183 m. To reach this depth with the sonic drill, one would need to telescope an additional casing size, either starting with a larger or finishing with a smaller diameter casing than is commonly available.

As part of the risk mitigation procedures and in anticipation of settlement during drilling close to the sinkholes, all drill rigs were supported on steel beams spanning the dam crest, and the work area was overlaid with geogrid to protect personnel from sudden drops in the ground surface. In addition, surveillance crews monitored surface and subsurface movement while drilling was underway.

After completion of drilling, 70 mm diameter alope inclinometer casing or 76 mm diameter plastic casing was lowered into the hole, then the hole was gROUTed in stages (to limit grout pressure and thus prevent hydraulic fracturing) while simultaneously removing the steel casing. In areas of very soft and/or collapsing ground, tube-

Figure 9. Sonic drill and support truck.

modification on this project was to add only enough water to balance in situ pore pressure. To address the shortcomings of the sonic drill, the following were implemented:

- Investigations with the sonic drill were started at a distance from the sinkholes, with the intention of drilling in ground that was largely undisturbed, to avoid triggering ground collapse.
- The sonic rig was instrumented to measure the following drilling parameters: hydraulic pressure and rate of fluid flow to the vibratory head, vibration frequency and acceleration of the head, hydraulic pressure driving the torque and downward pressure on the drill rods, and drilling advance rate. This had never been done before, and it was hoped that correlations could be developed between these drilling parameters and ground conditions such as density.

5.4 Testing of Sonic Drilling

The capabilities of the sonic rig were tested in sound

5.6 Results

Two figures are included which show results from
sonic drilling. Figure 11 shows fines content and void ratio (from water content) with distance from the sinkhole. Figure 12 shows a typical plot of drill parameters with depth.

5.7 Lessons Learned

Many lessons were learned throughout the sonic drilling program, first as drilling and sampling procedures were developed, and then as specific problems were encountered in each hole.

The first lesson was the need to carefully control water level in the drill hole. The water level needed to be sufficiently high to prevent heaving of soil, but also sufficiently low to prevent hydraulic fracturing. Control of water level was even more important when carrying out falling head permeability tests. Proper control in most cases was obtained only where the piezometric pressure was known with a reasonable degree of accuracy before drilling. This information was gathered from previous drilling, adjacent SCPTU testing, seepage analysis and existing piezometers in other parts of the core.

Another lesson was the difficulty of retrieving continuous samples in loose or soft ground. Various types of bits and core catchers were available, but experimentation was necessary to develop a reliable sampling procedure. In some cases, despite these measures, samples in loose ground were repeatedly dropped, causing severe sample disturbance (e.g., mixing, segregation when dropped through water).

The safety precautions taken during the sonic drilling proved to be invaluable. Surface settlements of up to 1.5 m occurred over an area of 2 to 3 m diameter around individual drill holes in or close to the sinkhole.

A group of five field inspectors supervised the drilling on a 24-hour basis (2 rigs), monitored instrument response, and logged all samples. Factors which contributed to the quality of this work were: written drilling, testing, and safety procedures that were updated as knowledge was gained; specific written instructions for each hole; establishment of standards for logging holes; and thorough and frequent communication between senior engineering staff and field staff.

In regards to logging standards, we found that in the early part of the program all staff were consistently underestimating the fines content of the dam core samples. The fines in the samples were almost entirely coarse silt, which was difficult to distinguish visually from fine sand. A set of standard samples was prepared, and all staff were trained in the visual estimation of fines content, after which time the logging became more consistent and accurate. This issue was important, because other field and laboratory testing was triggered by the identification of genuine fines deficient zones in the core.

Drilling parameter data (such as advance rate, head vibration or acceleration, etc.) were collected on most holes. Very extensive and detailed analysis of this data, both on a theoretical basis and looking for trends, provided no correlations with the geotechnical characteristics of the ground. This was a keen disappointment.

Completion grouting of the holes proved to be very difficult. Grout was placed in the hole in controlled lifts to prevent hydraulic fracturing of the core. Typically these lifts were 15 to 20 m thick. Since the grout needed to set up before another lift was placed, it was a time consuming process (even with the use of accelerators), one that ideally should proceed independently of the drilling except for the need to remove casing from the ground. In some holes where a higher permeability zone was present, it was necessary to place the grout in lifts only 2 to 5 m thick, which further slowed the completion process.

Mixing and pumping of non-standard grout mixes was also a problem (see Section 7). Keeping sand and fines in suspension required careful control of water content, which was also necessary in the maintenance of the mixing and pumping equipment.
The use of accelerators under variable temperature conditions caused particular challenges.

One behaviour that was not anticipated was the amount of bending and in some cases buckling that occurred in the slope inclinometer and other plastic casing, which led to problems with insertion of geophysical probes and the tube-a-manchette packer/gouting unit. The buckling effect was most pronounced close to the sinkholes, but some effect was evident even in holes 10 to 20 m away, from which we concluded that there were at least two causes:

- bending of plastic casing during installation and gouting, and
- lateral ground movement occurring after installation of the casing, due to later drilling or other reasons.

When the problem was recognized, two actions were taken: we switched to a larger plastic casing (to give greater clearance for the probes), and we modified procedures by keeping a string of drill steel inside the plastic casing during gouting, to reduce casing movement and bending during this process.

An important non-technical lesson was the necessity for clearly defined contracts with the drillers, even under near emergency response conditions. Particular challenges on this project included obtaining commitment for rigs when requirements changed daily, scheduling of crews when the drilling extended for long periods of time and/or around the clock, retaining good drilling crews who understood the site, and cost inefficiencies related to use of short term contracts over a long period where production efficiencies should reduce costs.

In retrospect, after an initial period in which the site staff became familiar with the drilling method and the drillers became familiar with the site conditions, the sonic drilling method performed essentially as expected. With care, all objectives except items 2 and 5 (see Section 5.1) were met. Use of the sonic rig to set surface casing for some of the SCPTU holes confirmed the potential of this method to cause ground settlement at the sinkholes, and hence it should not be used where collapse of loose ground is anticipated. Despite our best efforts, the recorded drilling parameter data were not useful in assessing ground conditions. The parameters which can be assessed for this method are very limited, but the technique does provide essentially a continuous relatively undisturbed sample. The vibrations can cause breakaways of coarser particles and this should be evaluated on a material and site basis.

6. UNDISTURBED SAMPLING AND PRESSUREMETER TESTING

6.1 Selection of Drilling and Sampling Methods

In assessing the processes responsible for the development of the sinkhole and for remediation design, it was important to obtain samples for gradation analyses and to measure the stress conditions within the sinkhole. Despite its merits, the SCPTU test could not reliably provide this
Figure 12. Sonic drill parameters for DH96-22 (55-65m).

information. Since the sonic drill was also not suitable for this purpose (because of the potential for further ground settlement and also because of poor sampling capabilities in soft ground), alternate methods of obtaining soil samples were considered.

A technique for mud rotary drilling in disturbed core (described below) had been developed during SCPTU testing. At this stage in the investigation, mud loss in the sinkhole was less of a concern because it had been decided to remediate the sinkhole with compaction grouting. Mud rotary drilling was therefore selected for the sampling hole.

Because of the variable ground conditions, several different samplers were tried: fixed piston sampler; heavy wall tube sampler with core catcher; side wall sampler; HQ3 core barrel; and a “slush” sampler which had originally been used in the arctic for sampling soft ice and which was modified for use in soft ground. Of these, the slush sampler was the most successful and is described herein. Two pressuremeters were also brought to site - a standard high pressure “insertion” model, and a low pressure, self-boring model.

6.2 Drilling, Sampling and Pressuremeter Test Procedures

The holes were drilled using a mud rotary rig, but the drilling procedure was unconventional.

- The upper portion of the hole (undisturbed core, above the water table) was cased off. For the sampling hole, a sonic rig installed 194 mm OD casing to 15 m depth. For the pressuremeter hole, a Barber rig installed 152 mm OD casing to 18 m depth.
- The sonic (or Barber) rig was moved off the hole and replaced by a Slimco 5000 rotary rig.
- PW casing was advanced into the disturbed down core, ahead of the drill bit, in 3 to 6 m increments, using a combination of down pressure and rotation with no fluid circulation.
- The inside of the casing was then drilled out with a tricone bit and mud circulation. The drill bit was not allowed to extend beyond the bottom of the casing.
- The last two steps were repeated to advance the hole between sampling or pressuremeter test depths.
- When excessive friction developed on the outside of the PW casing, the drillers switched to HW casing and continued with the same procedure.

Test hole DH96-36 was continuously sampled. It was drilled in November 1996 in sub zero conditions. Samples were taken frequently (every 1 to 2 m), mostly with the slush sampler affectionately known as “Frostie”. This sampler is similar to a long, very thick wall tube sampler, with an enlarged
tip (Figure 13). It was attached to DWL drill rods, lowered to the bottom of the casing, and pushed into the soil. A plug of soil at the bottom of the sampler was then frozen by passing liquid CO₂ from the surface through a 6.4 mm diameter plastic supply line into an expansion chamber at the tip of the sampler, and returning CO₂ gas through a similar line to the ground surface. Freezing of the plug typically required 2 to 4 minutes. Once freezing was complete, the CO₂ supply was shut off, the sampler was withdrawn from the hole, and the bottom of the sampler submerged in hot water to thaw the frozen plug. The sample was then extruded manually, logged, photographed and partitioned for gradation testing and water content determinations.

Pressuremeter testing was carried out at drill hole DH97-1 in early March 1997. A comparatively robust high-pressure pressuremeter was used for the first two tests because the pressuremeter operator was concerned about damage to the membrane from coarse material. The limiting pressure at 10% strain was very low in the first test (less than 50 kPa). Because of its low sensitivity, the high-pressure tool was replaced by the self-boring pressuremeter for the remainder of the tests. A total of 23 pressuremeter tests were completed over a depth range of 28 m to 91 m.

6.3 Results

The dam core was successfully sampled in DH96-36 from a depth of 32 m to the bottom of the disturbed core at 109 m, using the slush sampler. This feat itself was remarkable, and served as an indication of the low stress conditions. The lower, much harder and likely undisturbed dam core was drilled with an HÖ/3 retractor core barrel from 110 m to bedrock at 115 m.

The disturbed dam core was tested with the pressuremeter in DH97-1. Liftoff pressures were very low, varying from 15 kPa at 28.5 m depth to 220 kPa at 90.7 m depth. The following figures illustrate the sampling and test results:

- plots of void ratio vs. depth and fines content vs. depth (Figure 11) for DH96-36;
- a typical pressuremeter plot (Figure 14); and
- a plot of horizontal stress with depth (Figure 15).

6.4 Lessons Learned

As anticipated, mud rotary drilling beneath the sinkhole proved to be difficult. The most important consideration was hydraulic fracturing due to mud pressure. The drilling procedure provided some protection against hydraulic fracturing as long as the

Figure 13. "Frostie" sampler.

Figure 14. A pressuremeter result at Sinkhole No. 1.
hydraulically fracturing the ground, as long as a plug of soil is maintained at the bottom of the casing. However, if sampling or in situ testing is desired, it is necessary to clean out the entire casing which inevitably resulted in mud loss possibly due to hydraulic fracturing.

The second lesson learned was also an important one: that loose ground could be effectively and efficiently sampled with the “sloth” sampler, permitting recovery of slightly disturbed material. Near continuous samples are possible, although actual sample lengths may vary if ground conditions are variable. Minor problems that occurred when the sampler (designed for soft ice) was used in soil, were overcome with small modifications. This type of sampler has application in many other geotechnical problems involving soils that are too soft for conventional sampling techniques.

Drilling the mud rotary sampling hole was more difficult than the sonic holes. The main problem was high grout loss, despite the use of small lifts, grout accelerators, and even permitting the grout to partially set inside the drill casing. This was due to the extremely loose ground conditions around this hole. Significant time and effort were required to complete the hole.

Some of the performance issues noted above were accepted in this specific application because remediation of the poor ground within the sinkholes was planned. Under other circumstances the performance of the mud rotary drill would not have been tolerable because of the lack of fluid control and the uncertainty of damage to the core.

7. CROSSHOLE TOMOGRAPHY

7.1 Objectives

A variety of geophysical tests were used during the sinkhole investigation. The majority of these were deployed from the surface to search for potential defects in the upper part of the dam. These surface methods met with very limited success.

This section describes two downhole procedures that were used to investigate the dam core around the known sinkholes, over the full depth of the embankment. They are crosshole radar tomography and crosshole seismic tomography, both deployed from within cased drill holes.

Tomography, meaning measurement along a cut or section, is an imaging process originally developed for medical purposes. It allows the spatial distribution of some property within a test zone to be calculated from measurements made at the boundaries of the zone.

Drilling within the confines of the sinkholes was minimized to reduce the risk of precipitating another ground collapse. Tomography, in theory at least, promised the ability to characterize the sinkholes using drill holes that were safely beyond the disturbed zones.

7.2 Basic Principles

In crosshole tomography, a signal source is positioned in one cased drill hole and one or more receivers are located in another drill hole. Transmission of energy from source to receiver defines one ray path. Measurement of ray path travel time usually forms the basis of the procedure although in some cases other parameters may be measured as well. The discussion that follows is restricted to travel time; identical techniques apply for other parameters.

In a typical application, a dense array of overlapping ray paths, with a wide range of orientations, is generated. The travel time for each ray path represents one piece of information. Considered individually, this information allows the average velocity along one ray path to be determined. A collection of arrival times from a large suite of ray paths, however, permits the variation of velocity throughout the measurement plane to be calculated. This is carried out by subdivision of the zone traversed by the rays into N cells. The velocity in each cell is unknown but can be calculated from the data supplied by the N arrival times. Data processing is actually more complicated than this simple example suggests. A more detailed discussion of tomography can be found in Wong (1987).
Output from a typical tomography analysis gives the signal propagation velocity in each of the cells under consideration. These results are contoured to produce a tomogram that shows the distribution of velocity in the test zone. The overall accuracy of a tomogram is a complicated function of many factors:
- drill hole separation;
- number of ray paths;
- number of cells in the tomography model;
- number and accuracy of the boundary conditions;
- number of adjacent and intersecting planes analyzed simultaneously;
- orientation of ray paths with respect to the target;
- wavelength of the energy;
- velocity contrasts;
- error content of the arrival time data; and
- contouring procedures.

Although some guidelines on the influence of these factors are available, their combined effects on the accuracy of the final product cannot be quantified.

7.3 Planning And Implementation

Vattenfall HydroPower Ltd. of Sweden used crosshole radar tomography to image a sinkhole in one of their embankment dams (Carlsten et al. 1995). On the basis of this case history, plans were made, shortly after the discovery of Sinkhole No. 1, to conduct a similar investigation at Bennett Dam. At this stage use of crosshole seismic tomography was not anticipated. Examples documenting the successful use of the seismic technique under conditions similar to Bennett Dam were not known.

Crosshole tomography requires cased drill holes of sufficient size to accommodate the transmitter and receiver tools. For radar measurements, the casing must be non-metallic and held in place with grout transparent to the signal. Standard geotechnical grout used for drill hole backfill typically has a high water content and also some bentonite. Bentonite’s low electrical resistivity causes strong attenuation of radar signals. A number of modified grouts, eliminating the bentonite while attempting to maintain appropriate strength, stiffness and shrinkage characteristics, were tested at Bennett Dam. Ultimately, a standard “machine base” cementitious grout was used. This grout had a high resistivity and so did not significantly attenuate the radar signals.

Tomography requires that the start and end coordinates of each ray path be precisely specified. In crosshole applications this requirement can be satisfied by installing grooved casing that can accommodate a slope inclinometer probe. This allows the casing’s position to be accurately determined.

Original plans called for a ring of five to eight drill holes to be arrayed around Sinkhole No. 1 on a diameter of about 10 m. Crosshole tomography was to have been conducted between pairs of opposing drill holes. This layout, ideal for imaging the sinkhole, was modified for two reasons - one logistical, the other technical. Sonic drilling in the vicinity of the sinkhole and SCPTU testing directly in the sinkhole had to proceed simultaneously because of schedule constraints. With two and sometimes three rigs working in close proximity it was not possible to position all drill holes according to original plans. Secondly, preliminary testing results suggested that the sinkhole might be surrounded by a zone of intermediate disturbance. This led to a wider distribution of so called “step-out” drill holes. The resulting network of tomogram planes in the Sinkhole No. 1 area is shown on Figure 6.

During the reservoir drawdown period, preliminary crosshole radar measurements were made in two planes to test the feasibility of the procedure. This work involved three drill holes near the west abutment of the dam in an undisturbed part of the core, distant from Sinkhole No. 1 (the prototype test site). However, the decision to proceed with testing in the sinkhole areas was made, by necessity, before the preliminary radar results were available.

In the interim, a limited field trial of crosshole shear wave measurements had been conducted in two of the drill holes at the prototype test site. The small mechanical hammer sized to fit inside the plastic casing was capable of generating good quality signals. This result, together with general seismic velocity testing experience, was considered sufficient to warrant full scale crosshole seismic tomography at the sinkholes to complement the radar testing.

7.4 Crosshole Radar Tomography

The term radar is a misnomer that refers to the use of reflected electromagnetic energy in the radio frequency band, (used despite the fact that in the crosshole tests the energy is transmitted, not reflected). The propagation of radar waves through soil is a function of dielectric constant, electrical conductivity and magnetic permeability of the material. The dielectric constant of most non-clay minerals is 5 to 7, while it is about 80 for water. As a result, the bulk dielectric constant of soil is strongly affected by its water content. Assuming that other factors remain more or less constant, changes in radar velocity should correlate to
variations in water content (Carleton et al. 1995). If the material is saturated, changes in water content will correspond directly to void ratio.

The fact that radar transmitters generate repeatable signals affords an additional opportunity for tomography. By comparing the amplitude of a signal at its source to its amplitude at the receiver, attenuation can be determined. Thus a ray path, in addition to having an arrival time, will also be tagged with an attenuation value in dB/m. Tomography works on any quantity that has spatial variability, so calculation of radar attenuation tomograms is possible. This may reveal a localized area within the test zone where signal attenuation is very high.

Radar tomography at Bennett Dam was not very successful. Although the sinkholes can be discerned in the tomograms, they are indistinct, largely because the radar velocity contrasts are small. In addition, inconsistencies in the results occur: one sinkhole appears as a lower, and the other as a higher, velocity zone. An identical pattern appears in the attenuation tomograms. The poor quality of the results raised doubts about the usefulness of radar tomography as a diagnostic technique in this case.

The observed deficiencies likely arise from two factors. It appears that the correlation between radar propagation and water content, at least for Bennett Dam core material, is weak. Attempts to correlate radar velocities extracted from the tomograms with measured water contents from similarly located drill holes samples were not successful.

The combination of electrical and magnetic factors that affect radar propagation nevertheless produced tomograms that imaged the sinkholes. That these results contained inconsistencies suggests that no single parameter exerts primary influence. Radar response appears to be a complicated function of poorly understood parameters that may or may not relate to mechanical soil properties.

7.3 Crosshole Seismic Tomography

Crosshole seismic tomography, based on shear wave velocities, successfully imaged the sinkholes. An example tomogram, between DH96-24 and DH96-26 passing directly through Sinkhole No. 1, is given in Figure 16. This shows a narrow vertical zone, with shear wave velocities as low as 150 m/s. Undisturbed core, in areas well removed from the sinkholes, was found to have a typical shear wave velocity of about 550 m/s. The gap in the lower left hand corner of the tomogram is the result of a caving jam that prevented a full sweep of measurements.

The shear wave velocity tomograms correlated reasonably well with available geotechnical information and data obtained during compaction grouting of the sinkholes. In general, however, a tomogram can easily appear more authoritative than it actually is. We noted, on successive reinterpretations of the data, that the final result could change considerably. It is also possible to skew the results when they are being contoured. This is particularly true if contour lines are used, as visual emphasis can be created at arbitrary velocities. Monochrome shading leaves but does not eliminate this problem.

Areas of unusually low or high velocity in a tomogram often occur in areas where the ray path...
coverage is sparse. A tomogram should not be interpreted without reference to the ray path pattern upon which it is based. Alternatively, a ray path with an erroneous arrival time or incorrect geometry may be included in the data set inadvertently. This can have a subtle or dramatic effect upon the tomogram. Procedures to quickly assess the quality of the tomography data and allow errors to be corrected are not discussed here. The reader is referred to Wong (1987).

7.6 Lessons Learned

Some of the lessons learned during the course of the geophysical investigations at Bennett Dam are listed below. Several are specific to crosshole tomography; others, while applicable to tomography, relate to geophysical investigations more generally.

- Simultaneous analysis of multiple tomogram planes is essential for success. Drill holes should be located so that a maximum number of adjacent and/or intersecting planes can be included within a single three-dimensional tomography model.
- Slight casing bends can cause tools with minimal clearance to jam. Simple procedures to straighten casings prior to and during grouting should not be overlooked.
- Information that can be used as boundary conditions in tomography analysis (e.g., downhole shear wave velocities) should be obtained and applied if practical. This can significantly improve the quality of the results.
- Tomography works best when ray paths parallel to the boundaries of the target zone can be generated.
- Bad data for even a single ray path can have widespread effects on the quality of a tomogram. Rigorous quality control of all data is required.
- Crosshole tomography is slow, complicated, and expensive. An experienced contractor is essential for success. Even then, independent review of all aspects of the work is necessary. This should include reproducibility checks of selected results.
- Crosshole tomography makes no direct measurements within the test zone between drill holes. All results are inferred from measurements made at the boundaries of the test zone. A tomogram is best viewed as an approximate representation of actual conditions. Reasonable, but nevertheless non-unique solutions, can be obtained.
- Core should be taken in producing contoured tomograms. Monochrome representations are recommended over colour to minimize opportunities for visually skewing interpretation of the results.
- Interpretation of a tomogram should not be done without reference to the ray path coverage for that plane. Less credence should be given to results in areas where coverage is sparse.

- Analytical modelling of a proposed tomography program can give considerable insight into how successful the procedure may be.
- Attempt to determine if a proposed geophysical technique responds with sufficient sensitivity to variations in the geotechnical parameter(s) of interest.
- Prototype tests are valuable but may not reveal all of the potential problems that can occur in wider application of a technique.
- Geophysics is very data intensive. Rigorous data management procedures should be in place at the start of any investigation if inefficiencies and errors are to be avoided.
- Communication between geophysical and geotechnical practitioners is difficult; considerable effort is required if misunderstandings are to be avoided.
- Clearly defined requirements for data presentation and reporting should be specified in all contracting for geophysics work.
- Like many fields, instructive failures are reported in the geophysics literature less commonly than the successes. Even these, we suspect, may be coloured by an optimism. At a minimum, recognize that not all experience reported in the literature may be applicable to a new situation.

8. CHARACTERIZATION OF CORE BENEATH SINKHOLE

8.1 Geometry

The sinkhole geometry was interpreted from cross sections of the SCPTU results, primarily the tip resistance and the pore water pressure response. An example cross section is shown on Figure 8. It is not immediately clear what the lateral limits of the sinkhole are, due to the spatial variation of the tip resistance.

Very low tip resistances (less than 1 MPa) were encountered to a depth of 35 m and moderately low values (1 to 10 MPa) to a depth of 95 m. The latter are interlayered with zones of higher tip resistance and zones where the piezocene met refusal. There is a suggestion from the data reviewed in 3D that the most disturbed zone is drifting downstream at depth (reference CPT96-5 and 6).

The SCPTU results show a short distance from the sinkhole (reference CPT96-3 and 4) show a variable tip resistance profile, but no significant or vertically continuous zones of low tip resistance. This is interpreted as a transitional disturbed zone between the intact core and the sinkhole.
The results of the sonic drilling were used to extrapolate beyond the SCPTU locations to the intact core. Since the measured drilling parameters could not be correlated with density, the moisture contents from the samples were used to determine void ratio. Figure 11 shows the result for four drill holes at increasing distance from the benchmark tube at Sinkhole No.1. The reference line was calculated assuming an initial void ratio of 0.307, corresponding to the average fill density measured during construction, and using the compressibility of the core material when subjected to overburden stress.

It is clear from Figure 11 that void ratios decrease with distance from the sinkhole. For example, the average void ratio below a depth of 30 m is 0.38 in DIH96-36 (1.2 m from the centre of the sinkhole), 0.33 in DIH96-34 (2.4 m away), 0.22 in DIH96-24 (11 m away), and 0.26 in DIH96-37 (52 m away). The average void ratios obtained in these holes are representative of other holes at similar distances from the sinkhole. A linear interpolation of the results would suggest the edge of the disturbed zone at about 5 m from the benchmark tube. This is consistent with the results of the SCPTU which shows disturbed core at 4 m from the centre of the sinkhole.

Sinkhole No. 1 is a vertical to sub-vertical feature with a downstream drift at depth. The diameter of the most disturbed material is 2 to 3 m to a depth of 90 m. There are some stiffer zones between the loose zones, which may be continuous, or which may reflect a wander in the vertical sinkhole.

Between the inner zone and intact core is a disturbed zone, with an outside diameter greater than 10 m and less than 20 m. The seismic tomography results shown on Figure 16 support these dimensions, with a vertical column showing very low shear wave velocities (less than 200 m/s) to depths in excess of 70 m with a diameter of 2 to 3 m. The tomography also shows a downstream drift to the vertical feature.

8.2 Soil Properties

The SCPTU data (tip resistance, friction ratio, pore pressure response and decay), the 24 pressuremeter test results, and the relatively high quality continuous "frostie" samples were used to assess the geotechnical properties of the sinkhole material.

The following summarizes the main characteristics:

- Very low stress conditions (less than 10% of the estimated initial total vertical stress) exist to 90 m depth (Figure 15).
- The material is very loose, with void ratios of 0.37 compared to intact core with void ratios of 0.25 (Figure 11).
- The fines content is essentially unchanged from the as-placed fines content (Figure 11).
- The samples from the sinkhole showed the existence of "wet seams", similar to those described by Sherman (1973), throughout the full depth. These seams were 25 mm to 50 mm thick, randomly and frequently distributed, and were clearly visible as the samples were extruded from the sample tubes.
- Based on the SCPTU pore pressure response, significant zones of "contractive" material were noted in the sinkhole to a depth of 35 m.

9. CONCLUSIONS

All the objectives of the investigation program were successfully met. The sinkholes were defined and characterized to the extent required to select and design the remediation method. Along the way, and under conditions of severe schedule, technical, political and environmental stress, a great many lessons were learned. Perhaps the most important general lessons are as follows:

- We did not discover any "good" way to drill into the core of this dam, under operating reservoir conditions.
- For a project like this careful planning, meticulous attention to detail, high quality inspection, and detailed safety protocols are critical.
- Prototype testing of new and innovative methods is very helpful, but one must remain vigilant for unforeseen problems which are certain to arise.
- In situ conditions can have dramatic influence on the suitability of various investigation methods. In this case the extremely low stresses in the sinkhole affected the most suitable geophysical tomography methods, permitting the SCPTU (but influenced the interpretation of the SCPTU), and compounded the problems of drilling with fluids.
- Consider all risks with the investigation program and build risk mitigation strategies into all the work.

10. ACKNOWLEDGMENTS

The success of the Bennett Dam sinkhole investigation was due to the dedication of a large number of individuals drawn from companies across Canada. In particular, the exemplary contributions of Mr. David Hill, P.Eng, of Thorner Engineering Ltd. and his drilling investigation team are acknowledged. Likewise, Mr. Denis Diggle of Foundex Excavations Ltd. warrants special
recognition for the innovative drilling and sampling solutions he developed. Mr. David Wooller, P.Eng., and his ConeTec Investigations Ltd. staff are largely responsible for the success of the SCPTU testing. The crosshole seismic tomography would not have succeeded without the dedication and expertise of Mr. Patrick Lapointe, P.Eng. of Geophysics GPR International.

11. REFERENCES


Trends in geophysical site characterization

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ABSTRACT: Geophysical site characterization (GSC) is a rich and varied discipline. Sometimes misunderstood, it challenges the creativity and scientific abilities of its practitioners in areas as diverse as archaeology, forensics, environmental assessments, hydrogeology, rock mechanics and civil engineering. We review trends in GSC by identifying the major forces for change of the past decade and the responses of the GSC community to those forces. The forces include the rise of environmentalism, the consequent increase in value of near-surface resources of all kinds, and demands for higher resolution of those resources. Responses to these stimuli include restructuring of professional societies to begin to integrate and engage the various sectors of the GSC community and their clients, and the development of methodologies, hardware and software to achieve the levels of resolution and visualization required. For the future we see these trends continuing, though perhaps with a shift in emphasis from the developed to the developing countries. There are tremendous challenges and opportunities for research in many areas of GSC, including organic contaminant detection, infrastructure testing, fractured rock hydrogeology and novel non-invasive ways to probe the earth.

1 PREAMBLE

Geophysics is a very broad subject. The dictionary definition – physics applied to planet Earth – encompasses a huge variety of scientific and engineering activities. Individuals who might refer to themselves as geophysicists work in areas ranging from the evolution of the planets, through atmospheric physics, oceanography and hydrology, and the microstructure of pores. The boundaries between geophysics and disciplines such as geochemistry, remote sensing, soil physics and geological engineering are both nebulous and transient. For example, is a survey carried out with Radarsat imagery geophysics? Is time domain reflectometry (TDR) for soil geophysics? Are cone penetrometers (CPT) fitted with seismic or electrical sensors geophysical devices?

Geophysical site characterization (GSC) is also a rich and varied field, and one providing tremendous challenges to the small group of engineers and scientists whose profession it is. This group, however, has problems. They are split amongst several professional organizations and for the most part lack professional standards and licensing. The results of their work, simple representations of the complicated subsurface, are often difficult to explain and justify to a client. Perhaps as a result this profession lacks widespread credibility.

This paper is not a comprehensive overview of the subject. We try simply to give some broader geophysical context to the technical papers that follow in this session. We look first at the organizational and technical evolution of GSC as a whole over the past 10 to 20 years. We briefly describe the main geophysical methodologies, with an emphasis on recent developments. We show some survey examples that emphasize the breadth of the subject rather than its strengths. Finally we hazard some guesses as to where GSC is heading in the next few years.

Our opinions reflect our point of view of the profession. We work for a small consulting company specializing in environmental and hydrogeological applications. Our “bread-and-butter” work is in support of Phase II audits, mainly leaking tanks and groundwater contaminant phases.
Bedrock and overburden mapping, pipeline construction surveys, locating voids beneath office floors or golf courses, mapping unexploded ordinance, identifying graves (usually ancient but occasionally modern), and a host of other small and usually very interesting jobs fill our schedule.

2 INTRODUCTION

GSC can be defined as a reconnaissance survey of a site, undertaken preliminary to more detailed and usually more intrusive investigations. The geophysical survey images the subsurface in terms of physical properties averaged over comparatively large volumes. Compared to geochemical sampling or visual core analysis, undertaken on very small volumes of rock, geophysics has advantages of scale, cost and speed. Disadvantages include lack of resolution compared to point samples (but such precision can be deceptive in an earth which is almost always heterogeneous), and the fact that the data obtained are at least one step removed from the information required. The measured electrical resistivity must be interpreted in terms of porosity, permeability or contamination levels; seismic velocities must be interpreted in terms of rock or elastic moduli, and so on.

GSC is undertaken on many lateral scales, ranging from centimeters to hundreds of meters. "Geophysical" site characterization seems to start at depths of a few centimeters and extend to a few tens of meters. Remote sensing, for example, with its very minimal penetration, is not usually thought of a GSC method; neither is TDR testing of soils (although these distinctions are very arbitrary). While "sites" can extend to hundreds of meters in some cases, most targets are shallow.

GSC activities can be crudely subdivided into three main areas. Two of them are quite traditional: testing (soils, foundations, pavement etc.); and engineering (geotechnical characterization, mechanical properties, reliability, elastic moduli, depth to bedrock, base geology, fracturing, voids, tunneling detection, etc.). The third, environmental, is a bit of a catchall word. Used strictly, it refers to work performed as part of environmental assessments required by regulatory agencies. Increasingly it is used in a broader sense that includes surveys undertaken for anything that pertains to "the environment", including groundwater quantity and quality, buried hazards, and so forth. We prefer this broader definition because it identifies geophysics with some of the major issues of the day, but the result is an ambiguous boundary between what is termed environmental and what is termed engineering.

GSC professionals tend to fall into sectors which have emerged on their own accord. Again generalizing broadly, they are explorationists who would refer to themselves as geophysicists and whose roots are with the major exploration geophysical societies, and geophysical practitioners within other professions who would more likely refer to themselves as (for example) a civil engineer who does geophysics. These practitioners can be grouped into sectors in civil engineering, engineering geology and hydrogeology. These four sectors have been somewhat isolated by virtue of their training, their professional affiliations and preferred journals and congresses, and also somewhat marginalized by the large professional societies which they are affiliated. The general subdivision of GSC and its practitioners is shown schematically in Figure 1.

Several factors have conspired to change the shallow geophysics industry in the past 10 to 15 years. Some of these are briefly described below.

3 FORCES FOR CHANGE

a. There has been a steady increase in the perceived value of near-surface resources such as groundwater, soils, aggregate and archaeological remains. This reevaluation either precipitated or followed on from the
public's new preoccupation with all things environmental. The increasingly litigious nature of North America has dramatically increased the importance of subsurface information, and the value of non-invasive technologies for obtaining it.

b. The collapse of the mining exploration sector in the late 1980's (not unrelated to environmentalism) left many exploration geophysicists and geophysical instrument manufacturers looking for work. Some found it in the environmental market.

c. Groundwater contamination, perhaps the dominant environmental issue for the geophysical community, continues to challenge our industry. The conductive, inorganic plumes of the 80's were comparatively easy to locate. In the 90's the hydrogeological emphasis has shifted to organic contaminants, a much more difficult target.

d. The growth and - more importantly for us - the decay of infrastructure, highways, bridges, parking lots, etc. rough and parallel population growth and now provides tremendous opportunity for fast and reliable GSC methods that can map concrete and pavement.

e. As military bases are closed down, the need for non-contact methods for clearing these areas of unexploded ordinance (UXOs) and chemical contamination of the ground has grown dramatically. Landmines, although a rather different problem from UXOs, beg for a non-invasive identification technology.

f. New contexts for geophysics brought new sites and new targets. Sites included landfills, industrial waste lagoons, heavily urbanized (and contaminated) neighbourhoods, inside buildings, major intersections, airport runways, ex-army bases, and artillery ranges. New targets leachate plumes, industrial solvents, diesel fuel, steel drums, tanks, mine tailings, unexploded ordinance, and leaking ponds and barriers. These demanded new approaches.

g. The increasing value of near-surface resources to society, coupled with the public's increasing familiarity with computer technology, has raised the expectation of clients. Specifically they want higher vertical and horizontal resolution, image-quality presentations and even three dimensional representation of their targets. A generation raised on computer games like Quake is unlikely to settle for plane layered model interpretation.

h. Characterizing these sites and targets increasingly required measurement and presentation in three dimensions of space, and in time. Anthropogenic processes like the migration of contaminants in groundwater, or the remediation of a landfill, have time scales more nearly human than geologic. Increasingly it is the temporal development of the target, rather than its location, that has to be characterized.

i. Environmental science and engineering has blurred the traditional boundaries between disciplines (physics, geology, biology, civil engineering, etc.) to which we had become accustomed. Environmental projects foster a team approach, in which professionals from many disciplines meet regularly and on site.

j. Stringent safety procedures are enforced for hazardous sites. These, along with concepts like quality assurance and just-in-time data production, have considerably altered the geophysical working environment.

k. Geophysics has enjoyed an unfortunate reputation as an inaccessible subject, probably due to (a) the rigour of physics compounded by (b) a certain air of mysticism often associated with its interpretations. In the team-oriented environmental area there is a healthy trend towards more open dissemination of methods and their limitations.

1. Truly phenomenal progress in the processing speed, data handling capabilities and miniaturization of computing clearly overprint all these other factors, as they have in all professions. Only slightly less impressive are the advances in positioning. A decade ago perhaps half the time required for an electromagnetic (EM) survey could be taken up by positioning stations, and half the time for processing in coordinating the survey with local maps, other surveys, etc. GPS and GIS technology clearly is changing that.

4 ADAPTING

How has the GSC community responded to these trends? Without trying to uniquely identify cause and effect in each case, here are some of the changes we have noted.

4.1 Institutional.

The post decade has seen the opening of a what
could be termed a “third front” for exploration geophysics to go with mining and petroleum; the name “environmental geophysics” first appeared formally in a landmark publication from the Society of Exploration Geophysicists (SEG) (Ward, 1990).

The exploration societies became increasingly less relevant and insufficiently flexible as a forum for GSC. In North America, in the late 1980’s, GSC groups within these societies drifted towards client societies like the National Ground Water Association and their Outdoor Action Conference, then towards a small meeting with the improbable acronym of SAGEEP (Symposium on Applications of Geophysics to Engineering and Environmental Problems). From the 1992 SAGEEP meeting came a concerted attempt to bridge the gap between the exploration and civil engineering sectors of the profession through the formation of the Environmental and Engineering Geophysical Society (EEGS). EEGS now has over 800 members, a respected journal (Journal of Environmental and Engineering Geophysics) and a strong European section. Spurred by this development, the SEG has formed a parallel organization called the Near Surface Geophysics Section (NSGS).

There has been some progress towards the registration of geophysicists within state or provincial jurisdictions. Although strongly opposed in some quarters as an attack on the intuitive or “prospector” geophysicist, this move towards licensing can only be positive for the profession. Alarming, there have been recent setbacks in two US states. A bill to register geophysicists was recently thrown out in Texas and, as we write, there is a movement to repeal the current licensing requirement for some 300 geophysicists in California.

4.2 Technological.

As the value of near-surface resources as a whole increased, money became available for the development of geophysical instrumentation that could resolve them better. The dominant trends have been the refinement of existing equipment, and the “downloading” and down-sizing of software, hardware and methodologies from the mining, oil and even deep earth geophysical sectors. Developments in shallow seismic reflection and radar fit this pattern. Similar claims can be made for the development of small, high resolution time domain metal detectors like the Geonics EM-61 (see Figure 3), which build on the extensive experience in mining. The driving force behind the development of the so-called “push” geophysical technologies in which seismic, dielectric and resistivity detectors are mounted on cave penetrators are likewise the need for more detailed information on a progressively more valuable resource. The upsurge in seismic, electrical, EM and radar tomography is similarly based on, and funded by, the demand for better data from the shallow subsurface.

Regarding the accessibility of geophysics, there are certain improvements. Instrument manufacturers have made equipment that is much easier to use. There has been a notable increase in the research and marketing of semi-automated interpretation procedures such as neural network analysis (Barki and Poulton, 1997), Euler deconvolution (Ravat, 1994) and data fusion (Johnson and Harbes, 1997). These methods offer advantages when dealing with large surveys for specified targets (e.g., UXOs), and a consistent and well documented approach to an interpretation. Clearly there are downsides to both these developments; they are dangerous in the wrong hands.

The advent of inexpensive color printing, combined with inexpensive software presentation packages, has had a huge impact on the GSC profession in the past decade. Color, and shading with varying light angles, have brought out and, in effect, “explained” nuances in data that black and white contours could not. Increasingly visualization software is making its way into the industry, allowing the viewer to fly, game-like, through a data set.

Research funding levels in shallow geophysics have increased substantially, though by definition insufficiently. In the matter of geophysics and organic contamination, for example, a substantial and reasonably concerted effort is being made by both the civil engineering (Santamarina and Finn, 1997) and exploration (Endres and Breiman, 1996) sectors to understand the physical properties of light and dense organic compounds in the subsurface.

5 METHODS

We next briefly describe some geophysical methods and their place in modern GSC. There is a large selection to choose from and our coverage is not comprehensive. A few examples are given which are chosen to emphasize the diversity rather than the main strengths of modern GSC.
5.1 Resistivity methods.

Direct current passed through the earth between two grounded electrodes will expend energy (voltage) at different rates along its path depending on the distribution of electrical conductivity it encounters. This process can be observed remotely by observing the distortion of equipotentials (from what would be observed in a uniform half space) with two or more potential electrodes placed on or beneath the surface. The method is comparatively simple, robust, and has application wherever there are contrasts in electrical conductivity in space or (as monitoring) in time.

A workhorse of GSC for the past eight decades, resistivity is undergoing a renaissance thanks to the use of multi-electrode, micro-processor managed arrays, coupled with the software to display and interpret the huge quantities of data collected (Dahlin, 1996). Two- and three-dimensional arrays are increasingly used to provide tomographic images of the subsurface, in both space and time (Binley et al, 1997). Inversion of data in two dimensions is now common; at time of writing a very acceptable code was available free from the World Wide Web (Leke, 1997). Three-dimensional forward code is becoming available and affordable, and standard groundwater modeling code like MODFLOW can be readily adapted to this purpose.

Gravity and magnetics.

Like resistivity, these two classic techniques continue to see widespread use. Total magnetic field and magnetic gradient measurements are unique in their ability to detect ferromagnetic materials, and are relatively insensitive to the ambient electrical and noise which, in urbanized area, can make difficulty for EM. Steel drums aren't the only targets, however. Bauman et al (1994), from Komex Ltd in Calgary, used a magnetometer to map the thermoremanent magnetization of soil around wooden foundations of a fort that burned in 1861 near Rocky Mountain House, Alberta (Figure 7). Easily mounted on aircraft or helicopters, the magnetometer can be used for large environmental sites surveys such as for detecting abandoned well casings (Phillips et al, 1994), or mapping mine
wastes (Paterson, 1997). Inversion techniques are described by Li and Oldenburg (1996).

Gravity measurements are much easier and faster today with modern electronic and highly automated instruments like the Scintrex CG3 gravimeter. The gravity meter remains a mainstay of the tunnel and void detection industry, although ground penetrating radar (GPR) may come to dominate this area (see Lane, 1997, this volume).

5.2 Electromagnetics.

EM instruments energize the ground with a time-varying magnetic field that induces eddy currents in conductors. These currents in turn transmit their own (secondary) magnetic fields back to a surface or borehole receiver. In a discrete subsurface conductor these eddy currents (and their secondary fields) decay at a rate that is inversely proportional to the conductivity and size of the body. Through a layered earth the currents diffuse downwards and outwards like smoke rings beneath the energizing coil, with a velocity inversely proportional to the conductivity of the earth in which the currents flow.

The time varying inducing field may be either a sharp onset or turn off (Time Domain EM or TDEM), or a continuous single frequency. Single frequency instruments, operating at low frequencies over soils of normal conductivity, are termed ground (or terrain) conductivity meters and their meters read directly in mS/m.

The Geonics ground conductivity meters have been the backbone of the environmental geophysics industry for almost two decades. However, these instruments produce typically 1 to 10 numbers per station and their resolution is low. The demand for higher resolution, and the feasibility of recording large quantities of data routinely, are leading to the development of wideband EM devices which are better focused. Examples are the operational Geonics EM-61 (Figure 3), and the prototype GEM-3 (Won et al., 1997) and VTEM (Pelletier et al., 1997). Figure 4 shows a survey of a parking lot for underground tanks. Note that the anomalies are "tight"; that is, the targets are well resolved horizontally.

Airborne EM also sees some use in characterizing large sites, such as for mine wastes (Paterson, 1997) and oil field salinity (Paime et al., 1997) problems.

5.3 Seismology

Seismology involves the inversion of travel times and amplitudes of arrivals of the compressional (P), shear (S) and surface waves from natural or artificial sources. The inversion is in terms of the shear and bulk moduli, and the density of the materials through which the waves have passed. The past two decades have been marked by adaptations to the shallow environment, of oil field technologies for P and S waves on the one hand, and earthquake seismology for surface waves on the other.

The environmental field has seen a dramatic increase in the use of the seismic reflection method, and the
transfer of deep seismic technology to the shallow environment. A standard engineering seismic system (e.g. Figure 5) now records at least 24 and often 48 channels of data, and processing packages combining all the features of exploration seismology sell for less than $10,000. The seismic sources for shallow work remain unique, however, and include shotguns, rifles, slingshots and sledge hammers. Refraction surveys still dominate the market for shallow seismic work, however, again aided by inexpensive, powerful processing software.

In the civil engineering sector, the development of surface wave techniques, in particular the Spectral Analysis of Surface Waves (SASW) has been remarkable. Originally associated with very shallow targets such as soils and pavement (Stokoe and Nazarian, 1988) this methodology (which can be viewed as an adaptation of earthquake seismology) is now used effectively to measure shear wave velocities to depth of tens of meters (e.g. Rix 1997). Thus SASW complements and may even replace conventional seismic refraction and reflection shear wave methods to these depths. Several examples of these surveys are to be found in this volume. The general concept of SASW is depicted in Figure 6.

5.4 Radar.
Starting as a glacier survey in 1929, still largely an instrument for sounding ice through the 1960s, ground generating radar (GPR) has experienced a remarkable expansion in applications and instrumentation during the past two decades. From the grainy analogue records of a few years ago we now have high resolution, colored images with a striking resemblance to real geology. GPR is very analogous to seismology in that pulses of electromagnetic energy, having pulsewidths of a few nanoseconds, are transmitted into the subsurface where they are reflected and refracted before emerging at the surface to be recorded by a receiving antenna. The velocity of the pulse is determined primarily by the dielectric constant of the materials it passes through, and the attenuation primarily by their electrical conductivity. The power of exploration seismic processing has again been brought to this field, including resolution enhancement techniques such as deconvolution and migration (Fisher et al., 1992) and 3-D (McMechan et al., 1997). Borehole GPR systems capable of use in reflection and transmission modes are available from at least two manufacturers, making radar tomography available to a wide spectrum of users (Lane et al., 1996).

In the last few years GPR has made great inroads into the area of materials testing, for example of bridge decks (e.g. Meshier et al., 1996) and of...
concrete (e.g. Robert, 1996). An example of testing wood for rot is shown in Figures 8 and 7.

5.5 Geophysical logging.

Probably the main advances in borehole technology that impact (i.e. are available to) the GSC community, are the stable, narrow diameter induction log and the high resolution (millidegree) temperature log, and good PC-based software for displaying and processing log data. Vertical seismic profiling (VSP) of shallow stratigraphy is also accessible to small contractors who can match a hydrophone string to their 12 or 24 channel seismograph (e.g. Milligan et al., 1997). Again, the power of exploration VSP processing technology is available for the PC at a very reasonable cost.

More exotic tools now finding their way from the oilfield into shallow geophysical practice include nuclear magnetic resonance logging, dielectric logs, and various formation imaging technologies. Wyatt and Cumbest (1997, this volume) look at some of these in their paper.

Not all innovation comes with a high price tag. A final year Geological Engineering student at the University of Waterloo is experimenting with temperature logging of heated boreholes (using the cables made for melting the snow on roofs suspended in the holes). The temperature log measures the ability of the formation to conduct away the heat. In Figure 9 the temperature log, recorded after 4 hours heating of a PVC-cased hole through glacial overburden, is compared to gamma and neutron logs. The highs and lows in the thermal log represent areas of lesser or greater ability to conduct away the heat provided by the cable. In the section shown there is an interesting correlation with the neutron log, suggesting that the thermal conductivity is inversely proportional to the water content of the formation. Research continues on this approach, and also on a technique of using the heating cables to amplify the effects of fracture flow in uncased boreholes.

Push technologies.

Wireline logging has limitations for environmental investigations. It requires an invasive (usually cased) borehole and has, with most tools, a low vertical resolution relative to the requirements for some targets. A number of short-spacing, in-situ electrical probes (e.g. Schneider et al., 1993) have been developed, largely for monitoring changes in physical properties. Such devices are overshadowed by the advent of modified cone penetrometers which include (amongst others) resistivity, seismic and dielectric measuring devices behind the tip (e.g. Krarup-Jensen et al., 1997). These devices were mainly

Figure 7. The radar record recorded along the first 7 metres of Figure 8. The wood rot shows up as the disturbance centred on 100 cm.

Figure 8. Scanning a utility pole for rot with 1000 MHz. radar. Hong Kong.
developed, and remain strongly rooted, within the civil engineering sector. They are primarily used for mapping as opposed to monitoring. They offer excellent vertical control, though at considerable cost. Several examples of CPT geophysical applications are to be found in these proceedings.

5.6 Remote sensing.

As was mentioned earlier, remote sensing from satellites or aircraft as a means of site characterization is not generally viewed as being "geophysics" because of its very limited penetration.

The most productive technique for mapping anomalous subsurface conditions has always been thermography, in which variations in the thermal response to solar heating is measured with infrared scanners operating in the 1 to 15 micron range. Coverage of this kind is expensive, and success highly dependent on site and weather conditions. On the positive side large areas can be covered with resolutions (for helicopter-based scanners) approaching one meter. An excellent example of a thermographic survey over a chemical waste landfill is described by Havlena and Knowlton (1993).

5.7 New technologies.

Three promising methods in the early stages of development are electrostatic profiling, nuclear magnetic resonance sounding, and seismo-electric effect sounding.

Electrostatic profiling. This is basically a form of resistivity profiling but performed using high frequencies and capacitive plates instead of electrodes (Benderitter et al. 1994). The method was developed originally by the Russians for permafrost work and adapted by French scientists for archaeological and geotechnical applications where surface resistivities are very high (e.g. pavement, dry sand). Published results appear promising (e.g. Chaplet et al., this volume) but the equipment is not yet widely available.

Nuclear magnetic resonance (NMR) sounding. Another Russian development, originally called the "Hydroscope" and now commercialized by the French equipment manufacturer BRIS, it is claimed that NMR sounding can energize and detect the resulting precession of protons to depths of 100 meters (e.g. Goldman et al., 1994; Mikhailov et al., 1997).

The device is unique amongst geophysical instruments in that it responds to water directly; it is claimed that it can detect the amount of water and the average grain size of the aquifer as a function of depth. Research accounts are encouraging, but there is as yet insufficient independent work to allow a proper assessment.

Seismo-electric effect methods. Geophysicists at the University of British Columbia have devoted considerable effort to adapting this obscure but well documented effect to mining and GSC (e.g. Russell et al., 1997). Seismic waves, incident on a boundary such as the water table, can generate weak electrical potentials that can be picked up at the surface using electrodes. The field layout is a curious mixture of seismic refraction and resistivity.

Figure 9. Geophysical logs recorded in a PVC-lined borehole through glacial materials. The temperature log was recorded four hours after heating of the borehole with an electric cable had commenced. Note the similarity of the temperature and (inverted) neutron logs.
6 FUTURE TRENDS.

The competition for work, in the environmental area at least, is increasingly stiff. It is said that client firms for GSC are in some cases taking an aggressive approach. The situation is not helped by the overall lack of standards and regulations. Unqualified firms start up, undercut the more reputable firms, then fold leaving the profession in a disrepute.

There is a question as to where future GSC work will come from. The pessimists hold that environmental site characterization in the developed countries has peaked and will hereafter decline. The optimists (e.g., Roesig 1996) counter that the inherent value of geophysical information - and hence GSC work - on the shallow subsurface will continually increase in North America. On balance we think the market will grow fastest in developing countries. These countries have limited ability to pay for this work themselves, but the multi-nationals moving there know that it is in their best interest to be good corporate citizens.

In the short term UXO surveys will continue to generate good business. Butler (1997) estimates that unexploded ordnance contaminates 11 million acres of the United States alone. Landmines, manufactured primarily from plastics and inherently unstable, are a tremendous challenge to geophysical technology (e.g., Brauchl et al., 1996; Robert, 1996), and in some ways to the conscience of our professionists, that has not really been taken up.

Strategies such as Expedited Site Characterization (Benson, 1997) will become more important and will be generally beneficial to geophysics. They emphasize standards, multi-disciplinary, on-site interactions, and results produced in real time and on a common GIS database.

The demand for three dimensional data acquisition and presentation will continue to grow. Seismic, electrical and particularly radar tomography will be major growth areas.

Most geophysical methods are inherently more precise when used to monitor changes in time. There are many applications for geophysical monitoring that need development. For example, Doser et al. (1997, this volume) describe monitoring water infiltration under a levee. Site remediation and (leaking) engineered barriers should generate profitable work for monitoring geophysics, but this type of work will have to be aggressively sought out.

Similarly, infrastructure work on decaying bridges, paviours, pipelines, city services, etc., has the potential to provide considerable future work but the GSC community must continue to develop and market its capabilities.

Demands for higher sensitivity and resolution will continue. This will favour "push" technologies.

We detect some resistance in the profession to excessive automation of the measuring and interpretation processes, because they encourage (in the wrong hands at least) "formula" geophysics.

Unexploded ordnance, minefields, fractured rock hydrology, organic contaminant detection, monitoring engineered barriers are some of the geophysical application areas where research will be rewarded.

The trend towards drawing together the sectors, through organizations like EESG and through joint meetings between EESG, NSEG, ASCE, NWWA, CGS, etc., must continue. The diversity of GSC practitioners is a strength of the discipline, but our objectives should be to "round off the corners" on the schematic of Figure 1.

Problems that will continue include expanding and educating the client base for geophysics, countering the "lowest bid" mentality for awarding jobs, registration of GSC professionals, and contending with fraud.

ACKNOWLEDGEMENTS.

The ideas in this paper have been discussed in two other forums during 1997, informally at the Geo-Institute (ASCE) meeting in Logan, Utah, and as an invited paper at the 50th Canadian Geotechnical Conference in Ottawa (Greenhouse et al., 1997). We are both grateful for the feedback from those meetings which has allowed us to correct and refine our views, and apologetic to those who may have heard or read them twice.
REFERENCES


Bruschini, C., B. Gros, F. Guzene, P.Y. Piece, and O. Carmona. 1996. Ground penetrating radar and in situ coin sensor imaging for anti-personnel mines detection. Proceedings of GPR’96, the 6th International Conference on Ground Penetrating Radar (available from Tohoku University, Faculty of Engineering), Sendai, Japan, October, 211-216.


Endres, A. and Redman,J.D.(1996) “Modelling the electrical properties of porous rocks and soils containing immiscible contaminants”, JEEG, 0,#2, pp105-112.


Loke, M.H., 1997. Free 3D resistivity and IP modeling software. School of Physics, Universiti Sains Malaysia, 11800U5M, Penang, Malaysia (nhloke@usm.my).


Mester, D.E., C.B. Dawley and B.C. Pulles, 1996. A comprehensive radar hardware, interpretation software and survey methodology paradigm for bridge deck assessment. Proceedings of GPR’96, the 6th International Conference on Ground Penetrating Radar (available from Tohoku University, Faculty of Engineering), Sendai, Japan, October, 333-359


Robert, A. 1996. Dielectric permittivity of concrete between 50 MHz and 1GHz and GPR measurements for building materials evaluation. Proceedings of GPR`96, the 6th International Conference on Ground Penetrating Radar (available from Tohoku University, Faculty of Engineering), Sendai, Japan, October, 117-122.


Santamaria, J.C. and Fam,M., (1997). "Dielectric permittivity of soils mixed with organic and inorganic fluids (0.02GHz to 1.30GHz)". JEEG, 2, #1, pp.37-52.


Geo-environmental applications of penetration testing

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ABSTRACT: The application of penetration tests for geo-environmental purposes has grown rapidly in recent years. The Cone Penetration Test (CPT) has become recognized as a valuable in-situ testing technique because of its speed, reliability, cost-effectiveness and excellent soil profiling capabilities. In recent years additional sensors have been added to the CPT equipment to enhance and expand its capabilities for environmental purposes. These additional sensors include, electrical resistivity/conductivity, pH and ice detectors, gamma/neutron as well as special fiber optic devices for fluorescence measurements. Special penetrometer sampling probes have also been developed to obtain liquid and/or vapor samples as well as soil samples. These sample probes are installed with CPT equipment and vary from simple inexpensive devices for bulk water samples to sophisticated probes for very high quality water and/or vapor samples from specific zones. These sample zones are often identified using the continuous profiles from CPT equipment with special sensors. A summary of recent advances in CPT and penetrometer equipment related to geo-environmental applications is presented and examples are shown to illustrate these applications.

1 INTRODUCTION

In recent years there has been a steady increase in geoenvironmental engineering projects where geotechnical engineering has been combined with environmental concerns. Many of these projects involve some form of contaminant in the ground; these can take the form of vapors, liquids and solids. Hence, in recent years there has been a change in site characterization techniques to accommodate these environmental issues related to contaminants.

Drilling techniques have been modified to account for possible contaminated ground. However, drilling techniques generally produce considerable disturbance to the materials surrounding the drill hole, which can have a significant effect on subsequent sample quality.

With increasing application of data quality management, drilling and sampling techniques are becoming less acceptable. Also, drilling and sampling methods produce cuttings of the material removed from the drill hole. If these cuttings are contaminated they may require special handling and disposal methods. In many states of the USA there are regulations that require all drill cuttings removed from geo-environmental site investigations to be disposed or stored in an acceptable manner. This can increase the cost of a day of drilling by as much as US$1,000. Hence, there have been clear incentives to develop techniques that do not produce cuttings from the subsurface.

The most rapidly developing site characterization techniques for geo-environmental purposes involve direct push technology, that is, penetration tests. The direct push devices generate essentially no cuttings, produce little disturbance and reduce contact between field personnel and contaminants, since the penetrometer push rods can be decontaminated during retrieval.

A variety of penetrometer tests exist for both geotechnical and geo-environmental investigations. These tests can be divided into three main categories: logging, specific and combined tests. The most popular logging test for geotechnical investigations in soil is the Cone Penetration Test (CPT). The CPT provides a continuous profile of measurements, and it is
rapid, repeatable, reliable and cost effective. Specific tests include the field vane test and the pressuremeter test since these measure specific soil parameters and are often carried out in locations identified by the logging test. Combined tests typically combine the features of logging and specific tests into one test, examples of which are the seismic CPT and the cone pressuremeter. Full details of CPT technology is provided in a recent book by Lunne, Robertson and Powell (1997).

2. OBJECTIVES FOR GEO-ENVIRONMENTAL SITE CHARACTERIZATION

The main objectives for a geotechnical site investigation are to determine the following:

1. Nature and sequence of the subsurface strata (geologic regime)
2. Ground water conditions (hydrogeologic regime)
3. Physical and mechanical properties of the subsurface strata.

For geo-environmental site investigations where contaminants are possible, the above objectives have the additional requirement to determine:

4. Distribution, composition and concentration of the contaminants.

The contaminants can exist in vapor, liquid and solid forms. The above investigation should be carried out in sufficient detail based on the risk of the project. For geotechnical projects this is usually a function of the proposed structure and the associated hazards and consequences. The geotechnical engineer is often in control of the risk process and hence, selection of the required site investigation detail. For geo-environmental projects the extent of detail required for the determination of the distribution and composition of the contaminants may be controlled by various regulatory agencies, for which the engineer may have little control. With the rapid improvements in measurement technology, the in-site detection limits required by some agencies for certain contaminants are decreasing at a rapid rate.

For geotechnical investigations the information is often obtained at one instance in time and projections are made to predict changes in ground conditions due to such factors as seasonal rainfall, etc. For major projects where the observational method may be applied, critical parameters such as deformations can be monitored to evaluate the changing mechanical conditions. For geo-environmental projects where potential contaminants are identified, long term monitoring and sampling may be required for both design and either remediation or containment. Hence, the objectives for geo-environmental site characterization can be quite different from those for a more typical geotechnical site characterization.

3. CPT TECHNOLOGY FOR SITE CHARACTERIZATION

The Cone Penetration Test (CPT) has become an important in-situ test for the characterization of soils where penetration is possible. Penetration can be difficult through cemented materials and materials with large particle sizes although new equipment has improved the range of soils in which CPT is possible. The CPT provides excellent near continuous profiles of soil type and detailed stratigraphy. If pore pressures are measured (CPTU), improved stratigraphic detail can be obtained as well as important additional information on equilibrium ground water conditions, consolidation characteristics and hydraulic conductivity. Empirical and semi-theoretical correlation's are available to estimate a full range of mechanical properties (Lunne et al., 1997). The CPTU measures the mechanical response of the ground or material to the penetration process through cone penetration resistance (q), sleeve friction (f) and the pore pressure (u).

If a solid contaminant has mechanical properties significantly different from those of the surrounding soil then the CPT can identify the presence of the material. However, the CPT cannot quantitatively identify the chemical composition of the contaminants. Hence, sensors have been developed that can be added to cone penetrometers in an attempt to identify certain contaminants.

The measurement of equilibrium pore pressures can be an important part of an investigation to evaluate the direction of ground water flow and vertical pressure head distribution and hence the hydrogeologic regime. Most cone penetrometers that measure pore pressures contain high capacity pressure transducers because penetration pore pressures can be very large in soft soils. To improve the measurement of the equilibrium pore pressure some cone penetrometers include a low pressure transducer connected to the outside of the probe via a control valve so that equilibrium pore pressures can be measured to a very high degree of accuracy (e.g. 20 mm head of water). This can minimize the number of possible permanent monitoring wells to measure ground water flow regimes. However, care is needed when penetrating soft fine grained soils since it can take a considerable time to
dissipate the high penetration pore pressures. The use of small diameter probes (e.g., 2 cm²) can be advantageous for speeding up dissipation time in fine grained soil in cases where knowledge on in-situ pore pressure is vital.

4. GEO-ENVIRONMENTAL PENETROMETER LOGGING DEVICES

4.1 Temperature

The CPT and CPTU are excellent logging devices that provide near continuous profiles of mechanical parameters. Sensors have been added to cone penetrometers to enhance their application for geoenvironmental site characterization. One of the earliest sensors included to a cone penetrometer was a temperature sensor. Initially temperature sensors were used to aid in either calibration corrections or to locate zones of different ground temperature, such as frozen soil. Recently temperature sensors have been used to aid in the identification of contaminants that generate heat due to biological or chemical activity.

4.2 Electrical resistivity and conductivity

The next major sensor that has been added to the CPT is for the measurement of electrical resistivity or conductivity. The conductivity is the inverse of resistivity, with the following as a useful guide:

\[
\text{Conductivity (µS/cm)} = \frac{10,000}{\text{Resistivity (Ω-m)}}
\]

The measurement of electrical properties was first developed to evaluate in-situ density of sands (Knoezen, 1981) but more recently it has been used to evaluate contaminated soils (Horsnell, 1988; Camparella and Weenkees, 1989; Woelfer et al., 1991; Struzewsky et al., 1991). The rationale for making electrical measurements is that in many circumstances, the electrical properties of the soil will be changed by the presence of contaminants. Therefore, by measuring soil resistivity, the lateral and vertical extent of soil contamination can be evaluated. Unsaturated soils and saturated soils with many non-aqueous-phase-liquid (NAPL) compounds exhibit very high electrical resistivity (low conductivity). Dissolved inorganic compounds, such as those contained in brines and landfill leachates, significantly decrease soil resistivity. Table 1 presents some typical values of resistivity and conductivity for some soils and contaminants. The resistivity CPT works on the principle that the measured voltage drop across two electrodes in the soil, at a given excitation current, is proportional to the electrical resistivity of the soil. The resistivity electrodes are typically steel rings from 5 mm to 15 mm wide that are set apart by distances that vary from 10 mm to 150 mm. The larger the spacing the greater the depth of penetration for the electrical field into the surrounding soil. Some probes have several electrode spacings so that lateral penetration varies. Some devices use small circular electrodes mounted around the circumference of the probe. Figure 1 shows a typical resistivity cone penetrometer with two ring electrodes (Woelfer et al., 1991). The electrodes are designed to be reasonably wear resistant and have a high electrical conductivity. A non-conducting plastic or other material is used as the insulator separating the electrodes. The resistivity measurements are typically made by applying a sinusoidal current across the electrodes and measuring the resultant potential difference between the electrodes.
<table>
<thead>
<tr>
<th>Material Type</th>
<th>Bulk Resistivity (Ω-m)</th>
<th>Fluid Resistivity (Ω-m)</th>
<th>Bulk Conductivity (µS/cm)</th>
<th>Fluid Conductivity (µS/cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sea water</td>
<td>-</td>
<td>0.2</td>
<td>-</td>
<td>50,000</td>
</tr>
<tr>
<td>Drinking water</td>
<td>-</td>
<td>&gt;15</td>
<td>-</td>
<td>&lt;665</td>
</tr>
<tr>
<td>Mine tailings sand with acid drainage</td>
<td>1-40</td>
<td>2-27</td>
<td>10,000-250</td>
<td>5,000-370</td>
</tr>
<tr>
<td>Mine tailings sand without acid drainage</td>
<td>70-100</td>
<td>15-50</td>
<td>145-100</td>
<td>665-200</td>
</tr>
<tr>
<td>APL contaminants in sand</td>
<td>0.5-1.5</td>
<td>0.3-0.5</td>
<td>20,000-6,000</td>
<td>33,000-20,000</td>
</tr>
<tr>
<td>NAPL contaminants in sand</td>
<td>125</td>
<td>48</td>
<td>80</td>
<td>210</td>
</tr>
<tr>
<td>100% ethylene dichloride (ED)</td>
<td>-</td>
<td>20,000</td>
<td>-</td>
<td>0.5</td>
</tr>
<tr>
<td>50% ED/50% water in sand</td>
<td>700</td>
<td>-</td>
<td>14</td>
<td>-</td>
</tr>
<tr>
<td>17% ED/83% water in sand</td>
<td>275</td>
<td>-</td>
<td>36</td>
<td>-</td>
</tr>
<tr>
<td>Clays</td>
<td>1-100</td>
<td>-</td>
<td>10,000-100</td>
<td>-</td>
</tr>
<tr>
<td>Alluvium and sands (non-marine)</td>
<td>10-800</td>
<td>-</td>
<td>1,000-12</td>
<td>-</td>
</tr>
<tr>
<td>Oil Sands</td>
<td>4-800</td>
<td>-</td>
<td>2,500-12</td>
<td>-</td>
</tr>
</tbody>
</table>

This enables resistivity measurements between 1 and 250 ohm-m to be made with an accuracy of ±0.2 ohm-m. A 1000Hz source is typically used to avoid polarization of the electrodes.

Electrical resistance is not a material property but a function of the electrode spacing and size. To convert from resistance to resistivity, which is a material property, a laboratory calibration of the probe geometry is necessary. The resistivity of soil is for the most part influenced by the resistivity of the pore liquid, which in turn is a measure of the pore liquid chemical composition. Hydrocarbons are non-conductive and will therefore exhibit high resistivity. The resistivity CPT has been used successfully in acidic ground conditions. The main disadvantage with electrical measurements is that the bulk resistivity or conductivity is not directly controlled by the chemical properties of the surrounding material. The measurements are strongly influenced by the background soil and pore liquid. Hence, it is important to obtain measurements of the background uncontaminated ground for comparison. For relatively uniform soil conditions it is possible to develop local site specific correlations between the bulk resistivity and selected contaminants (Campanella et al., 1994). The primary advantage of the resistivity CPT is that it provides continuous profiles of bulk resistivity along with the full CPT data in a rapid cost effective manner. The profiles of resistivity measurements can then be used to identify potentially critical zones where detailed sampling and/or monitoring can be carried out.

An example of a resistivity CPTU profile is shown in Figure 2. This profile was obtained at a site where the main contaminant was creosote from a timber treatment plant (Campanella et al., 1994). The measured bulk resistivity values are larger compared to the background values in zones with the contaminant. The free product was verified by monitoring well sampling.

![Figure 2. Example resistivity CPTU sounding at a site with organic contamination by heavy oils (After Campanella et al., 1994).](image)
4.3 Dielectric measurements

The electrical measurements discussed above relate to resistivity. However, electrical measurements can also be made to measure the dielectric constant of the material surrounding a penetrometer. The resistivity is somewhat insensitive to contaminant type whereas, the dielectric constant can be very sensitive to contaminant type. The dielectric constant is frequency dependent and is therefore dispersive. However, above about 100 MHz the dielectric constant is essentially constant. Delft Geotechnics (1994) developed a High-frequency-impedance-measuring (HIM) probe to measure both dielectric constant and conductivity of soil samples and hence, detect the presence of contaminants. The probe is a cone penetrometer based instrument with a retractable cone. The cone is pushed into the ground in the closed position and at the required depth the tip is retracted so that a soil sample fills the space. The sample chamber contains a central pin which acts as an antenna and the rim of the sample chamber acts as a receiver. A high frequency electromagnetic field pulse is generated at the ground surface and transmitted through a coaxial cable to the sample in the HIM probe. The resulting dielectric constant and conductivity are determined as a function of frequency (10-500 MHz). After the measurement the sample is pushed out of the inner cylinder re-establishing the cone shape. At the next depth the procedure is repeated. Accuracy of the HIM probe stated to be is 5% for both the dielectric constant and conductivity, however, the issue of cross-contamination may be a problem. Contaminants such as light and dense NAPL zones, if in sufficient quantity, can be detected by the low values of dielectric constant and conductivity in relation to the background surrounding water saturated soil. The location of the contaminant in the sample cup showed some effect on the measured values (Delft Geotechnics, 1994).

Newer dielectric CPT modules have been developed (Shin et al., 1998, Rosé et al., 1998) where the dielectric constant is measured between two rings mounted similar to those of the resistivity CPT. Measurements can be made at high frequency (100 MHz) or in the time domain (TDR) (Lelebvre, 1997). In uncontaminated soils the dielectric measurement can be used to estimate soil moisture content.

4.4 pH sensors

The acidity of a material can be measured using pH sensors mounted either inside a cone or on the surface of the probe. The major disadvantage of sensors mounted inside the cone is that a sample of pore liquid must be drawn into the measuring cell and then expelled. This process can be difficult in low permeability soils such as clay and the cell and sensor must be cleaned after each reading. More details about sampling techniques will be given in a later section. Sensors mounted on the outside of the cone have the advantage of direct exposure to the surrounding material, however, abrasion and damage to the sensor can be a major problem. Several CPT manufacturers and operators have placed a pH sensor in a small recess a short distance behind the friction sleeve. This recess is designed to produce a small vortex for pore liquid to enter during cone penetration. The sensor is then well protected from abrasion and damage from the surrounding solid material and can measure continuous variations of the pH of the pore liquid. The pH measurements are sensitive to temperature changes and generally a temperature sensor is mounted adjacent to the pH sensor to allow automatic correction for temperature effects. The continuous measurement of pH can be a useful guide for detecting certain contaminants with significantly different pH values from that of the background soil. A pH sensor was successfully used to differentiate between acidic tar material and waste drilling mud at an old disposal site in California. The temperature profile was also elevated over the acid tar depth interval and confirmed that chemical reactions were occurring in the acid materials. At this site the resistivity profiles could not delineate between the acid tar material and the drilling muds (Bratton, 1994).

4.5 Redox potential

The oxygen exchange capacity of a material can be measured with a sensor for redox potential. A CPT probe (Chemoprobe, Figure 3) described by Olle et al. (1992) carries out the measurements of redox potential, pH and conductivity. These three parameters are major variables of chemical equilibrium for inorganic substances. The sensors are mounted inside the cone and a sample is drawn into a measuring cell located a short distance behind the cone tip. The measurements are made under a nitrogen atmosphere to reduce the exchange of atmospheric oxygen with dissolved gases from ground water. A slight
excess pressure of nitrogen is used during penetration of the probe to stop the flow of liquid into the 15 mL measuring cell. The nitrogen is supplied from a small nitrogen cylinder in the probe. The stainless steel porous filter is cleaned by the excess nitrogen, which in turn cleans the sensors. Deionised water is also used to clean the sensors and check the calibration. A pressure sensor is also included to monitor the flow of liquid into the measuring cell and to estimate the hydraulic conductivity of the surrounding soil. Olste et al. (1992) showed that the measurement of redox potential along with pH and electrical conductivity enabled the detection of a floating layer of versoic acid beneath a storage tank at a petrochemical plant in the Netherlands. A similar probe (Chemicon) was described by Woeller et al. (1991) and is shown in Figure 4. Further details of devices that sample the surrounding pore liquid or vapor are given in later sections.

4.6 Gamma and neutron sensors

Gamma and neutron sources and sensors have been added to cone penetrometers in the past in an effort to measure in-situ density and moisture content (e.g. Mitchell et al., 1988; Sully et al., 1988; Minnen et al., 1995). However, these devices have not become popular because they contained active radiaction sources which can present significant problems if the probes should become lost in the ground and require expensive recovery. Recently there has been a trend toward adding passive gamma-ray sensors in an effort to detect radioactive contaminants (Marton et al., 1988; Singha et al., 1997). A variety of different passive sensors are available, the selection of the appropriate sensor is based on efficiency, range of gamma-ray energies expected, temperature dependence of the sensor and sensor ruggedness. An example of a radiation detection cone is shown in Figure 5. The application of gamma-ray sensors is clearly limited to environments where specific gamma-ray emitting contaminants are possible.

4.7 Laser induced Fluorescence

The most recent sensors to be added to the cone penetrometer for environmental applications are those that involve laser induced fluorescence (LIF). Hydrocarbons are one of the most common ground contaminants and most hydrocarbons, because of the poly-aromatic constituents, produce fluorescence when irradiated with various forms
quantified with a time-grated, one-dimensional photo diode array. Readout of a fluorescence emission spectrum requires approximately 16 milli-seconds. A microcomputer based data acquisition and processing system controls the fluorometer system, acquires and stores sensor data once a second, and plots the data in real-time as vertical profiles on a CRT display. A schematic diagram of the SCAPS probe is shown in Figure 6. Field trials at petroleum-oil-lubricant (POL) contaminated sites have been promising although results have been qualitative since calibration for specific contaminants is difficult.

Research has shown that common fuel contaminants such as heating oil, jet fuels, gasoline and diesel fuel exhibit strong fluorescence signatures, with the degree of fluorescence depending on the excitation wavelength (Gillispie and St. Germain, 1993; Chudyk et al., 1983). However, common
chemical contaminants such as chlorinated hydrocarbons (e.g., TCE, PCE) do not fluoresce and are not suitable for the fluorescence technique. The intensity of fluorescence is a function of excitation wavelength and recent efforts have been made to develop tunable laser fluorimeters, i.e. systems that can vary the wavelength of the laser light source (Bratton et al., 1993), and hence, detect a greater range of contaminants. More information about the contaminants can be obtained if the complete wavelength-time-intensity matrix is recorded, although this measurement takes a little longer to perform and a pause in the penetration is required. Fluorescence research to date has concentrated on the aqueous phase and very little work has been carried out evaluating the LIF characteristics of contaminated soils (Apitz et al., 1992). The intensity of the LIF signal is strongly dependent on soil type, with sands having a stronger signal for a given concentration than clayey soils. Bratton et al. (1993) suggest that LIF research in soils is in the infancy stage and even standard laboratory procedures for evaluating LIF response in soil materials have not yet been developed. The calibration and detection limits of LIF in soils are complicated by soil type, soil grain size effects, natural organic compounds (humic acid) and the influence of time on the contaminant degradation. There are also problems of low signal to noise ratios for the LIF systems. Currently, field correlation of LIF intensity to contaminant concentration are preferred since laboratory calibrations are still uncertain (Bratton et al., 1993). Concerns also exist over the long term durability and maintenance of the fiber optic cable.

Olle et al. (1994) have also developed a fluorescence CPT probe. The probe is 55 mm in diameter and contains a ultra-violet (UV) light source as well the fluorescence detection system. No fiber optic cables are required since the complete sensing system is located in the probe. During penetration measurements are made by illuminating the material surrounding the probe with a small mercury lamp to produce the UV light source placed behind a clear window. The fluorescence emitted by the hydrocarbons is detected in the probe by a small photomultiplier tube. The signal is conducted through the electrical CPT cable to a data processing system at the ground surface. A detection limit of 50 mg/kg dry weight for light NAPL is claimed. The intensity of the radiation emitted by the contaminant is an indication of the concentration of the product in the soil, however, specific calibration is required. The detection system can also handle other wavelengths for identification of other contaminants by using filters to control the excitation wavelength. Preliminary results of a demonstration site where NAPL layers of domestic fuel oil were present show excellent results (Olle et al., 1994). An example profile from the hydrocarbon probe is shown in Figure 7. These results show excellent baseline stability and clear sharp peaks in a series of alternating NAPL layers. Van Bee and Olle (1993) also investigated the effects of smearing and the displacement of the measured contaminant due to the penetration process. The limited results indicate that the soil effectively cleans the window on the surface of the cone and that the displacement (smearing) of the detected layer can be as much as 5 cm.

A newer version of the ultra-violet induced fluorescence (UVIF) CPT has been developed (Shian and Bratton, 1995) and incorporated as a module behind a standard cone penetrometer. The

![Figure 7. Example profile from ultra-violet induced fluorescence (UVIF) probe. (After Olle et al., 1994).](image-url)
UVIP module contains a UV source downhole and a single optic fibre to transmit the induced fluorescence to a simple data acquisition at the surface. Results using this simplified UVIF system look promising.

4.8 Vision CPT

A recent development has been a cone penetrometer that can provide real-time visual observations and recording of the subsurface (Raschke and Hryciw, 1997; Hryciw et al., 1998). High and low magnification miniature CCD cameras housed within the vision cone penetrometer (Vis CPT) having fields of view of 2.3 and 18.0 mm, respectively, provide the continuous stream of images. A field video recording system allows the video sequence from each camera to be recorded in real time. The subsequent images can be digitized for computer image analyses. Preliminary results look promising.

5. GEO ENVIRONMENTAL PENETROMETER SAMPLING DEVICES

The cone penetrometers described in the above section are primarily screening devices that log the ground profile for geotechnical and chemical measurements. Based on these measurements it is often possible to identify potentially critical zones or regions that may require more selective testing to measure or monitor specific contaminants. Sampling and monitoring wells are usually installed for this purpose and penetrometer technology has been used to develop a complete range of short and long term sampling probes. Sampling probes have been developed to sample either vapors, liquids or solids. The following section describes some of the main developments in this area.

Most of the available vapor and liquid samplers have some common features. Almost all the samplers are pushed to the desired depth based on adjacent CPT profiles. Sometimes the sampler is pushed down the same hole as the CPT, and since the sampler is of a larger diameter (typically 50 mm) than the CPT contact, it can be maintained with the surrounding material. Generally, the push rods are pulled back to expose a filter to the surrounding material. This avoids contamination of the filter before reaching the required depth. Selection of the appropriate filter material is based on the type of sample (gas or liquid) and the expected contaminant type. A sample is pulled into the sampler using either a vacuum or the natural in-situ fluid pressure. The sampler can then be either withdrawn to the surface for sample retrieval and sampler decontamination or the sample can be taken to the surface via a tube or by wicking and the process repeated at a greater depth. The following describes some typical samplers.

5.1 Liquid samplers

The most common discrete depth in-situ water samplers are the Hydropunch and BAT Enviroprobe. The Hydropunch and its variations is a simple sample tool that is pushed to the desired depth and the push rods withdrawn slightly to expose the filter screen. The filter screens can be made from a range of materials although stainless steel is the most common. Screened intervals can vary in length from 100 mm to 1500 mm depending on ground conditions, required sample depth, contaminant type and hydraulic conductivity of surrounding material. A small diameter bailer is lowered through the hollow push rods and body of the sampler to collect a liquid sample. Figure 8 illustrates a typical Hydropunch sampler. A peristaltic pump can also be used to pump larger volumes of non-volatile liquids to the surface. This type of push-in liquid sampler is very common, simple to use and can produce large samples. However, there is little control over the sampling process and liquid samples are often turbid (i.e. contain suspended
solids), especially when using a coarse screen. Modified versions of the Hydropunch concept have been developed to install long term monitoring wells, an example is shown in Figure 9. These include innovative techniques to seal and grout the sampler into the ground after the sampler is pushed to the required depth.

The BAT Enviroprobe, shown in Figure 10, was developed by B.A. Torstensson (1984) and consists of three basic components:
1. A sealed filter tip with retractive sleeve attached to the push rods.
2. Evacuated and sterilized glass sample vials, enclosed in a housing and lowered to the filter tip via a wireline system.
3. A disposable, double-ended hypodermic needle which makes a hydraulic connection with the pore fluid by puncturing the self-sealing flexible septum in the filter tip.

The filling rate of the sample vials can be monitored using a pore pressure transducer attached to the vial. The monitoring shows when the fluid infiltration is complete, assuring that pressure inside the vial is equal to the in-situ fluid pressure. These measurements can be used to estimate the hydraulic conductivity of the surrounding material. Various modified versions of this concept are now in use for ground fluid sampling. The system is most applicable for retrieving fluid samples where limited volumes are sufficient (<150 ml). Small filter size allows for discrete sampling intervals, but can create longer sampling times in less permeable materials. The protective sleeve and fine filter screen produces fluid samples with low turbidity. However, sometimes the needle can become blocked with fine material passing through the filter resulting in incomplete filling of the vial.

Modified versions of the above samplers have been developed to provide a mixture of the two techniques. Samplers exist that allow large volume (>1200 ml) samples to be taken under a back pressure of argon or nitrogen gas to avoid volatilization. Carpinteri et al. (1998) have produced a larger BAT type probe for sampling in sandy soils. The measurement of pressure with
time can be used to monitor the sampling process. O'Neill et al., (1995) describe a cone penetrometer that can also take in a liquid sample and pump it to the surface for analysis. However, sampling time can become excessive in soils of low hydraulic conductivity.

5.2 Vapor samplers

Vapor (gas) samples can be obtained in a manner similar to that described above for liquid samples. However, special care is required to purge the sample tubes and store the samples. Typically, the sampler is pushed to the required depth, the filter element exposed and a vacuum applied to draw a vapor sample to the surface. The volume of gas can be monitored using special monitoring equipment and typical sample containers are Texlar bags, gas tight syringes and glass or steel sampling vessels. Special disposable plastic tubing is used to draw the sample to the surface.

Vapor sampling modules have also been added to cone penetrometers to allow sampling during a CPT. The sampling module is typically located a short distance behind the cone. Samples can be taken during short pauses in the penetration. To minimize cross-contamination for subsequent samples within one vertical sounding requires either a two line sampling design for purging of the vapor collection lines between samples or a single line system with nitrogen or argon gas purging. A small positive internal gas pressure can stop the inflow of gases during the penetration process.

A recent sampling module (Bratton and Barrington, 1998) has been developed to purge volatile gases to detect small quantities of contaminants.

5.3 Solid samplers

A variety of push-in soil (solid) samplers are available. Most are based on designs similar to the Gouda or MOSTAP soil samplers. The samplers are pushed to the required depth in a closed position. The Gouda type samplers, shown in Figure 11, have an inner cone tip that is retracted to the locked position leaving a hollow sampler with 31 mm diameter stainless steel or brass sample tubes. The hollow sampler is then pushed to collect a sample. The filled sampler and push rods are then retrieved to the ground surface. The MOSTAP type samplers, shown in Figure 12, contain a wire to fix the position of the inner cone tip before pushing to obtain a sample. Modifications have also been made to include a wireline system so that solid samples can be retrieved at multiple depths rather than retrieving and redeploying the sampler and rods at each interval.

6. SEALING AND DECONTAMINATION PROCEDURES

The hole produced by the penetrometer requires sealing using a special grout. A summary of techniques for sealing CPT holes was presented by Lutenegger and DeGroote (1985). The grouting can be carried out either after the push rods are removed or during the removal process. It is common for the grouting to be carried out after rod removal using special grout push rods with disposable tips. The hole is grouted from the bottom up as the grout rod is pulled from the ground. The grout rods generally follow the previous penetrometer hole since the rods follow the path of least resistance. Recently operators are using penetrometers that allow grouting during penetrometer retrieval (refraction grouting). The grout is usually delivered to the penetrometer through a small diameter grout tube pre-threaded
7. FUTURE TRENDS

The potential cost savings of penetrometer based geo-environmental site investigations has fostered considerable research expenditures into additional sensors. Research is underway on improvements to the fluorescence techniques. Research is also underway at various centers around the world to investigate new sensors that could be incorporated into cone penetrometers, such as, Raman spectroscopy, fiber optic chemical sensors, laser-induced-breakdown-spectroscopy (LIBS), time-domain-reflectometry (TDR), ground penetration radar and integrated optoelectronic chemical sensors.

Raman spectroscopy involves a powerful laser light which is focused onto the reverse side of a glass or quartz slide which has been coated with a species-specific chemical. Laser-induced-breakdown-spectroscopy (LIBS) can be used to identify hazardous metal compounds. The laser light must be focused directly onto the material to form a short lived plasma which emits light that can be collected using fiber optics and analyzed.

Ground penetrating radar (GPR) technology for surface measurements has developed considerably in recent years and systems are under development that will incorporate the technique into penetrometer probes. GPR responds to changes in dielectric constant of the material which can be sensitive to contaminant type. However, the dielectric constant of soils varies over a wide range and is strongly influenced by the presence of water. The depth of penetration is also controlled by soil type and excitation frequency. Penetration of the GPR can be very limited in saturated clayey soils.

An interesting area of current research is on the development of integrated optoelectric chemical sensors (Burns and Mayne, 1998). They contain a small diode laser which is focused on a chemically selective overlay. These sensors can be small and in modular form and can be inexpensive.

A major problem with the development and application of many chemical sensors relates to the interaction of the sensor with the contaminant. Most of the sensors require that the contaminant (usually in vapor or liquid form) be pulled into the probe so that the chemicals can interact with the sensor. This produces problems related to the cleaning of the sensor, measuring cell and filter element to avoid cross-contamination, as well as...
the time required for this process. Little research has been carried out to evaluate these problems and the issues related to the interaction of the contaminant and soil, the interaction of the contaminant and measuring device, the contaminant state and contaminate mixtures.

8. SUMMARY

To better characterize potentially hazardous sites, improved investigation devices and methods are being developed which use cone penetrometers. The CPT gathers high quality in-situ geotechnical information in a rapid cost effective manner. Electrical and chemical sensors have been adapted to cone penetrometer probes to enable mapping of subsurface contamination in sufficient detail to reduce the need for more costly invasive subsurface sampling and monitoring points. Traditional drilling and sampling methods when compared to CPT methods have high waste management costs from handling and disposal of contaminated materials. Also, CPT methods minimize exposure of field personnel to hazardous environments.

Significant developments have been made in recent years to improve geo-environmental site characterization using penetrometer technology. Sensors have been added to the cone penetrometer to enhance the logging capabilities for both mechanical and chemical measurements.

Bratton and Higgins (1992) describe a synergistic approach to 3-D site characterization by utilizing a combination of surface geophysical technology and direct-push (i.e. penetration test) technology. By taking advantage of the synergism between the two a significant improvement can be achieved in the interpretation of the geophysical data.

9. REFERENCES


Bratton, W.L., 1994, Personal communication.


Deformation and in situ stress measurement

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ABSTRACT: This paper deals with the topics of measurement of deformation parameters and in situ stress parameters, concentrating on in situ test methods. The discussion on the former topic is based on recognizing that even the "elastic" behaviour of stiff geomaterials is highly non-linear. This non-linearity is expressed by a simple model. The calibration of the model involves using seismic methods to determine $G_s$, the initial tangent shear modulus, and some other larger-strain test to determine the rate of degradation of the stiffness with increasing shear stress or shear strain. The discussion on the measurement of in situ horizontal stress examines some of the conventional in situ test methods, and the potential of some seismic methods.

1. INTRODUCTION

The topic of this Thesis Paper is the measurement of deformation and in situ stress parameters in soil. The paper deals mainly with in situ test methods of determining these parameters, but appropriate laboratory test methods will also be discussed briefly, where appropriate.

As will be emphasized later, the two areas dealt with (measurement of deformation and in situ stress parameters) are at very different stages of development. Thus, whereas I believe that we are making very rapid progress in our ability to measure deformation parameters, measurement of in situ stress is still very problematic. Most of the discussion in this paper will therefore be devoted to the former topic.

Parameters required for deformation analyses depend on the type of soil and the loading conditions. Thus, for soft clays, the deformations and strains under embankment loading can be very large, whereas for "well designed footings" in stiff soils, the deformations and strains are generally very small. In this paper, we will be concentrating on the latter.

The brief given emphasized that this is not meant to be a State-of-the-Art paper, but rather a paper setting out some themes for discussion. In a number of recent international conferences, the concern has been raised repeatedly that the gap between research and practice is widening. The central theme of this paper is therefore how to close this gap, and this is done by showing how practitioners can start to implement some of the recent advances in the area of measurement of deformation parameters.

The starting point for this paper is the fact that one of the most important advances in geotechnical engineering in the past decade or so is the general realization and acceptance of the fact that the stress-strain behaviour of almost all soils is highly non-linear, even for stiff soils in the "elastic" region of the stress-strain response. Once this is accepted, and methods of dealing with it are devised, the whole problem of how to go about predicting deformation becomes much clearer.

Traditionally, geotechnical engineers have spent considerable time and energy searching for empirical correlations between "the Young's modulus $E$" of the soil and various in situ test measurements (SPT N-values, CPT $q_v$ values, etc), using the observed settlement response of footings. This value of $E$ would then be used in settlement or deformation calculations. One of the reasons why this has proved to be of limited accuracy is that trying to fit a linear elastic model to non-linear behaviour means that the correlations obtained must relate only to that particular combination of footing type and size, soil type and level of loading.

Though, as already stated, this paper is not meant to be a State-of-the-Art review, it is worth giving a brief summary of some of the most important sources of information on the topics discussed in this paper. This is done in the next section.

2. STATE-OF-THE-ART REVIEWS

Non-linear stress-strain response in the "elastic" region has been accepted for a long time in the area of earthquake engineering. In this context, the dependence of secant shear modulus $G$ on strain level for cyclic (dynamic) loading was illustrated by...
a number of researchers using laboratory resonant column testing (eg. Hardin and Drnevich, 1972a, 1972b). More recently, the applicability of the same ideas to "static" loading problems has been emphasized again and again.

A number of Conferences, and a number of individual papers, have been particularly important in the development of this topic. From a personal point of view, the most important have been:

- many of the contributions to the 1st International Conference on Pre-Failure Deformation of Geomaterials held in Sapporo, Japan, in 1994;
- a number of contributions to the 10th European Conference on Soil Mechanics and Foundation Engineering held in 1991 in Florence, Italy, particularly the General Reports by Atkinson and Salfors (1991) and Burguignoli et al. (1991);
- a number of papers by Professor Tatsuoka and his colleagues at the University of Tokyo Institute of Industrial Research and various collaborators with this group (eg. Tatsuoka and Shibuya, 1991), the most recent example being the Theme Lecture on the topic of pre-failure deformation properties presented at the 14th International Conference on Soil Mechanics and Foundation Engineering in Hamburg, Germany (Tatsuoka et al., 1997);
- many papers in the proceedings of the International Conference on Advances in Site Investigation Practice in London, UK (Craig, 1995);
- a number of papers on the stiffness of London Clay by researchers at Imperial College, London and others (eg. Burland, 1989, Jardine et al, 1986, Simpson et al., 1979);
- a number of papers by researchers in Italy (eg. Balbi et al., 1979; Bellotti et al., 1989; Jarnoikowski et al., 1985).

An indication of the rate of advance in this area is the striking contrast between the almost sole reference to non-linear elasticity made at the 8th European Conference on Soil Mechanics and Foundation Engineering held in Brighton, UK, by Simpson et al. (1979), compared to the very frequent recurrence of this theme at the 10th European Conference on SMFE in Florence, Italy, in 1991.

The breadth and depth of the State-of-the-Art reviews in this area (particularly in many of the Keynote Lectures in the 11th International Conference on Pre-Failure Deformation of Geomaterials) is such that there is no need to cover the same material again in this paper. Researchers have in recent years, or are at present, examining all aspects of this topic in exhaustive detail. An indication of this is provided by the contributions to the recent Symposium in Print on Pre-Failure Behaviour of Geomaterials published in Géotechnique (Vol 47, No. 3, 1997).

While the research output in this area increases dramatically, the state of practice in many areas has not changed for many years. The gap between research and practice therefore appears to be widening.

As already stated, what is needed now is to encourage practitioners to start to think of the implications of non-linear behaviour for prediction of ground deformation. This is the main aim of this paper.

3. BACKGROUND

The use of non-linear "elastodynamics" for almost-routine analysis of deformation problems appears to be nowhere more prevalent than in the London area, mainly due to the complexity of the problems being addressed in association with tunnel construction and the construction of excavations for very deep basements. The types of soil models being used for this work have been explained in detail by Hight and Higgins (1994) and Jardine (1994). These models are relatively sophisticated, having been developed specifically to deal with London Clay over a period of some 20 years or so. Ideally, this model, or models of similar sophistication, will eventually be employed for large and important projects in similar conditions everywhere in the world, and in some areas, this is already happening.

For small projects of a routine nature, it is likely that the current standard methods of predicting ground deformations (particularly foundation settlements) will continue to be used, since the potential savings to be made by more refined methods may not justify the extra expense associated with these methods. However, even for such projects, it is time to ask if some of the cruder standard methods need to be re-evaluated, and perhaps even discarded.

The approach that will be outlined here falls somewhere between the very sophisticated approaches (such as used in London) on the one hand, and the current very basic standard methods on the other. This approach is based on a simple non-linear elasticity model, and measurement of the soil stiffness at failure.

3.1 Non-Linear Elasticity

Many of the methods of representing the non-linear stress-strain behaviour of soil at small to medium strains are based on a hyperbolic stress-strain relationship, which may be expressed in the form:

$$\frac{G}{G_0} = \frac{1}{1 + \left(\sigma/\gamma_f\right)^n}$$

(1)

where $\gamma_f$ is the reference shear strain defined as:

$$\gamma_f = \frac{\sigma_{\text{ref}}}{G_0}$$

(2)
where $\tau_{\text{max}}$ is the shear strength. This equation may be written in an alternative form:

$$\frac{G}{G_0} = 1 - \frac{\tau}{\tau_{\text{max}}}$$

In these equations, $G_0$ refers to the initial tangent or "small strain" shear modulus ($G_{\\text{sec}}$ is also used in the literature for this value), and $G$ is the secant shear modulus measured at zero shear strain to the current shear strain.

For uncedented soils, the value of $G_0$ depends on the confining stress raised to some power:

$$G_0 = \left(\frac{\sigma'}{p_0}\right)^n$$

where $\sigma'$ is the mean effective confining pressure, and $p_0$ is the atmospheric pressure (used to ensure that the constant $C$ is dimensionless). In reality, the value of $G_0$ may not be equally dependent on all three principal stresses, but this will be discussed in more detail later.

If it is assumed for sand that $G_0$, (the shear strength) depends on $\rho'$, then $G_0$ and $\tau_{\text{max}}$ increase at different rates with increasing confining stress $\rho'$.

The rate of reduction in secant shear modulus $G$ with increasing shear strain $\gamma$ is conventionally shown on a plot of $G/G_0$ (or just $G$) plotted against the shear strain $\gamma$ on a logarithmic axis. Figure 1(a) shows such a plot. The three curves in this plot were obtained by assuming a base set of values of $G_0$ and $\tau_{\text{max}}$ (80 MPa and 50 kPa, respectively, in this case) and then assuming that the effective stress level increases by a factor of 4 and 16, giving a factor of 4 and 16 increase in $\tau_{\text{max}}$, but an increase in only 2 and 4 in $G_0$.

This simple exercise shows the wide scatter in the three curves, a scatter that is masked to some extent by the use of a logarithmic strain axis. Plotting the same data on linear axes (between 0 and 1% shear strain) as in Figure 1(b) shows the situation much more realistically. Note that if the shear strain values were normalised by the reference shear strain $\gamma_0$, as suggested by Equation 1, the three curves would be coincident.

There is much discussion in the literature about the existence of an elastic "threshold" – i.e., a threshold shear strain below which the soil response is linear elastic. The semilogarithmic plot in Figure 1(a) suggests that if there is such a limit, it is about 3 x 10^7 % for the lowest-strength case, and about 2 x 10^6 % for the highest strength case. However, these curves are simply plots of a hyperbolic stress-strain model, where the tangent stiffness reduces right from the start of shearing. Thus in this case, and also perhaps with much of the experimental data, the apparent elastic threshold is an illusion of the plotting method used.

Figure 2 shows data presented by Ishihara (1996) for cyclic triaxial tests on Toyoura sand. In this

![Figure 1. Shapes of hyperbolic curves for sand at three different stress levels, plotted as $G/G_0$ versus strain on (a) logarithmic axis and (b) linear axis.](image1)

![Figure 2. Modulus degradation curves for Toyoura sand, for 10th cycle in resonant column tests (replotted from Ishihara, 1996)](image2)
case, the stiffnesses shown are after 10 cycles for samples with the initial effective confining stresses shown on the Figure. This shows a similar spread of curves as in Figure 1(a).

A much more informative method of showing the modulus degradation is to plot \( \frac{G}{G_0} \) versus shear strain, normalised by the shear strength (i.e. versus \( \tau / \tau_{\text{max}} \)). For the true hyperbolic relationship, this plot is a straight line, as shown by the thick solid line in Figure 3(a).

The hyperbolic model has been found to provide a reasonable fit to some cyclic test data. However, when applied to monotonic data, the rate of degradation is found to be much faster than implied by the hyperbolic model.

Hardin and Drnevich (1972b) found that the hyperbolic model did not always fit even the cyclic data. To improve the fit, they used a distorted shear strain scale that they termed a “hyperbolic shear strain” \( \gamma_h \) defined as:

\[
\gamma_h = \frac{\tau}{\tau_{\text{max}}} \left[ 1 + a \exp(b \frac{\tau}{\tau_{\text{max}}}) \right]^{-1}
\]

(5)

The empirical parameters \( a \) and \( b \) effectively allow the hyperbolic model to be distorted.

Fahey and Carter (1993) proposed an alternative method of distorting the hyperbolic relationship:

\[
\frac{G}{G_0} = 1 - f \left( \frac{\tau}{\tau_{\text{max}}} \right)^g
\]

(6)

where \( f \) and \( g \) are also empirical fitting parameters.

Figure 3(a) gives an indication of the range of shapes of modulus degradation curves \( \frac{G}{G_0} = \frac{\tau}{\tau_{\text{max}}} \) that can be obtained by using different values of \( f \) and \( g \) in Equation 6. Figure 3(b) shows the same data as in Figure 3(a), but plotted as \( \frac{G}{G_0} \) versus shear strain \( \gamma \). Putting \( f \) and \( g \) equal to 1 recovers Equation 3 (the true hyperbolic relationship).

Values of \( g \) less than 1 allow the \( G/G_0 \) ratio to reduce more rapidly with increasing shear strain than occurs with the hyperbolic relationship. Conversely, values of \( g \) greater than 1 give a slower rate of degradation initially, with high values of \( g \) giving an extended elastic “threshold” as shown in Figure 3(b).

Values of \( f \) less than 1 allow the maximum shear stress to be reached after a finite shear strain (= \( \gamma_{\text{max}}(1-f) \)). This was required because the model was to be coupled with Mohr-Coulomb plasticity to model a complete pressuremeter test in sand. It also results in the stiffness just before reaching \( \tau_{\text{max}} \) being finite \( (G/G_0 = 1-f) \). Values of \( f \) greater than 1 have no meaning.

Figure 4 shows modulus degradation \( \frac{G}{G_0} \) curves presented by Tatsuoka and Shibuya (1992) for a wide range of geomaterials. This shows the types of degradation curves that are obtained in practice, and a comparison between the shapes of curves in Figures 4 and 3(a) shows that the proposed simple model is sufficiently versatile to represent the range of behaviour observed.

Much more elaborate distortion models have been proposed, some of which are much more versatile in fitting experimental data (a summary is given by Tatsuoka and Shibuya, 1991), but this model is sufficiently complicated for the present purpose.

Ishihara (1990) presents data from cyclic torsional shear tests on samples of undisturbed and remoulded Fujinawa sand. The undisturbed samples were recovered by freezing, and trimmed and set up in the torsional shear apparatus before being thawed. Figure 5(a) shows the modulus degradation curves \( G - \gamma \) for the undisturbed and remoulded samples, indicating a much higher \( G_0 \) value for the undisturbed samples, and a much higher rate of reduction in \( G \) with increasing shear strain. These data are replotted in Figure 5(b) as \( G/G_0 - \gamma \).

This shows that not only do the absolute value of \( G \) of the undisturbed samples reduce more rapidly,
but also the normalized value $G/G_c$, reduces more rapidly than for the reconstituted samples. This (and other evidence) suggests that for many materials, it is important to measure parameters in situ, unless very high-quality sampling procedures are used that are capable of providing undisturbed samples for laboratory testing.

Baldi et al. (1989) proposed a method of deriving a "working strain stiffness" $E'$, (the secant stiffness at a strain of 0.1%) for sands from the measured value of $G_c$. Various overconsolidation ratios were used. They assumed that the hyperbolic relationship of Hardin and Drnevich (1972) applied in all cases. The derived values were compared with $E'$ values measured directly in triaxial tests. The results are shown in Figure 6. This shows that in practically all cases, the hyperbolic model overpredicts the $E'$ value. (From Figure 6, the overprediction is about 3.8 for the NC sample, and by between 1.3 and 1.7 for the OC samples). This shows indirectly that the rate of modulus degradation is much greater than the hyperbolic model predicts, especially for NC sands. Clearly, some distortion to the hyperbolic model would be required in this case.

3.2 Small-strain stiffness

The modulus degradation models discussed above

![Figure 4](image4.png)

Figure 4. Modulus degradation curves for various geomaterials (Tatimura and Shibuya, 1991).

![Figure 5](image5.png)

(a) Shear modulus $G$ versus $\gamma$

(b) Shear modulus $G$ versus $\gamma$

Figure 5. Modulus degradation curves for Fujisawa sand plotted as (a) $G$ versus $\gamma$, and (b) $G/G_c$ versus $\gamma$ (replotted from Ishihara, 1996).

![Figure 6](image6.png)

Figure 6. "Working strain" stiffness $E'$ measured in triaxial tests compared to values calculated from $G_c$, assuming a hyperbolic relationship (Baldi et al., 1989).
assume that the small-strain shear modulus $G_s$ of the soil is known. For the purposes of this discussion, this parameter may be regarded as a fundamental soil parameter that can be measured with reasonable certainty, and it is therefore an appropriate basis for a deformation model.

While the $G_s$ value was originally investigated by dynamic tests (particularly resonant column tests in the laboratory), more recent "static" tests indicate that the value of $G_s$ is the same for both static and dynamic tests for a very wide range of material types. In effect, the value of $G_s$ may be regarded as being practically independent of shear strain rate. This has been shown over and over again by many researchers (eg. Tatsuoka and Shihanya, 1991).

The small strain stiffness is also ideal as the basis for a deformation model because it can be determined by a very wide range of methods, both in situ and in the laboratory. Of particular interest is the fact that the shear wave velocity in a soil depends on $G_s$ and the bulk density of the soil ($\rho$):

$$V_s = \sqrt{\frac{G_s}{\rho}}$$

and hence, if $V_s$ and $\rho$ are measured, then $G_s$ can be determined directly:

$$G_s = \rho V_s^2$$

A complicating factor is that the shear wave velocity (and hence the deduced value of $G_s$) is not necessarily isotropic, being affected both by stress anisotropy and anisotropy of soil "fabric". This will be discussed later in the context of measurement of in situ horizontal stress. For the sake of simplicity at this stage, the anisotropy in shear wave velocity (and hence in the deduced value of $G_s$) will be ignored, and $G_s$ will be assumed to be a unique value at any particular point in the soil.

3.3 Effect of Age

Contrary to traditional understanding, the age of even a clean sand deposit has an influence on its stiffness. This effect is manifest over both short time scales and geological time scales. Stokoe et al. (1994) show that the value of $G_s$ for both undisturbed and reconstituted silty sand samples increases slightly with confinement time prior to testing in the resonant column apparatus, for times up to 1000 minutes. This topic has also been discussed by Schnitternann (1991), Baldi et al. (1989) and Bennett et al. (1991), among others. Jamiołkowski and Manassero (1995) present data on the effect of age on $G_s$ for a range of sands, using the following equation:

$$G_s(t) = G_s(t_0) \left[1 + N_s \log\left(\frac{t}{t_0}\right)\right]$$

where $G_s(t)$ and $G_s(t_0)$ are the $G_s$ values at some reference time $t_0$ and any time $t$ greater than $t_0$, and $N_s$ is a dimensionless parameter indicating the rate of increase in $G_s$ per log cycle of time. For silica sands and gravels, the values of $N_s$ quoted by Jamiołkowski and Manassero are between 1 % and 3.5 %, while for calcarenite or glassy sand, the values ranged from 3.9 % to 12 %. Values for clays tended to be much greater (up to 19 %).

This effect of age is one of the reasons commonly advanced for the difference between in situ and laboratory values of $G_s$ even in clean sand deposits, where the laboratory values are obtained from samples reconstituted to the in situ void ratio. The data from Ishihara (1996) presented earlier in Figures 5(a) show that $G_s$ values from laboratory tests on truly undisturbed samples are about twice the values of the equivalent reconstituted samples. Using a value of $N_s$ of 2% (about the mean for sands, as suggested by Jamiołkowski and Manassero, 1995), the $G_s$ value of a relatively young sand deposit (say, 10,000 years) would still be only 13 % greater than a reconstituted sample of the same soil tested after 1 day's confinement (in this case, the age difference is equivalent to 6.5 log cycles of time). For the data presented by Ishihara (Figure 5(b)), the age of the deposit would have to be about 50 log cycles of time greater than the time of confinement of the laboratory tests to give a ratio of 2 between the undisturbed and reconstituted samples. Assuming the time of confinement in the laboratory tests to be 1 day, this would imply the age of the deposit to be greater than 4000 years! Thus, either the $N_s$ values are much greater than the range suggested by Jamiołkowski and Manassero, or there are other effects, such as slight cementation, that must be taken into account.

All of this suggests that unless very high quality undisturbed samples can be obtained for laboratory testing, it is necessary to measure $G_s$ using an in situ test.

4. DEFORMATION PREDICTION METHODOLOGY

Even the most basic deformation prediction methodology that uses non-linear elasticity requires the following elements:

- a method of determining the small-strain stiffness of the soil $G_s$;
- a method of determining the rate of degradation of secant stiffness ratio $G/G_s$ with increasing shear stress or shear strain;
- a technique for using this non-linear model to
predict the load-deformation behaviour. When high-quality undisturbed samples can be obtained, the first two requirements can be met by laboratory testing. However, it is emphasised that the type of testing and standard of testing required may not be available outside of sophisticated research laboratories. Similarly, the standard of sampling required may also not be available, at least in routine practice.

In the laboratory, $G_s$ can be determined by dynamic tests, such as the bender element method (Dyvik and Olson, 1989), or resonant column testing, or by cyclic triaxial or torsional shear tests, or by monotonic triaxial tests. The rate of degradation of stiffness ratio with increasing shear stress or shear strain may be different for monotonic loading and all but the first cycle of cyclic loading. Hence, if the problem being addressed involves monotonic loading, a monotonic loading test is required in the laboratory. This can be done in a triaxial or torsional shear apparatus, with the major requirement being very careful measurement of strains on a part of the sample remote from the effects of the boundary.

This approach coincides with what has previously been described as a very sophisticated procedure. In accordance with the philosophy of examining "moderately sophisticated" procedures, we shall concentrate here on using in situ testing methods to determine the $G_s$ value, and then examine potential methods of deriving the modulus degradation behaviour.

4.1 In situ determination of $G_s$

The topic of Session 2 at this Conference is geophysical methods. At the time of writing this paper, perusal of the Abstracts submitted to that Session indicated that a number of papers deal with the topic of determination of $G_s$ in situ, using geophysical methods, and it was also assumed that the Theme Lecture, and the Discussion Session 2, would deal with the topic. Therefore it is not necessary here to deal at any length with this topic. Nevertheless, some comments are warranted, because of the importance of in situ determination of $G_s$ in deformation prediction.

In areas of the world where the CPT is a standard method of site investigation, the obvious method of determining $G_s$ is to add a seismic capability to the CPT (Robertson et al., 1986). The additional cost of the seismic CPT above the standard CPT equipment is modest, and the additional time required to carry out shear wave velocity measurement is also quite reasonable. It therefore seems that there is no reason why seismic CPT testing should not become a routine addition to standard CPT testing, particularly when prediction of deformations in likely to be an important aspect of a project.

There is some discussion in the literature about whether geophones are required at only one point on the cone, or whether they are required at two points, one on the cone and the other some distance above the cone (generally 1 m above it, or sometimes closer). Butcher and Powell (1995a) show an example from a stiff clay site (Madingley) that indicates that the "two-point" method gives better data, and while others claim that the "single-point" cone gives equally good data, the two-point cone seems to be preferable.

Of course, more traditional down-hole and cross-hole seismic methods are also available for determining $G_s$, but these are regarded as being more specialised and less readily carried out than seismic CPTs.

Non-intrusive (surface) methods of determining $G_s$ profiles are also common. Rayleigh waves of different frequencies (and hence different wavelengths) can be used to determine the average $G_s$ value over different depth ranges, from which a profile of $G_s$ can be determined. The SAWS method (Steckoe et al., 1989) is a refinement of this technique, in that random waveforms can be generated, and inversion techniques used to determine the $G_s$ profile. From the number of references to the SAWS method in the abstracts submitted to this Conference, the method appears to be gaining in popularity.

Once the profile of $G_s$ is determined, this value can be used directly in deformation calculations. In some circumstances, this will give a good prediction of the deformations, particularly in stiff or hard soils. The example of the anchor block for the Rainbow Bridge in Tokyo Bay is given by Tatsuka and Kobata (1994). The foundation caisson for this block, founded in sedimentary mudstone, has a plan area of 70 m by 45 m, and a total weight of about 280,000 t, equivalent to a bearing pressure of about 870 kPa. When the anchor block (about 140,000 t weight) was constructed atop the caisson, the resulting deformations of the caisson were carefully determined, together with strains in the mudstone below the caisson. In this case, they used the laboratory or field $E_s$ values (the Young's modulus value corresponding to $G_s$), and a stiffness degradation curve derived from laboratory tests, to predict the movement of the foundation. This prediction was done after the bridge had been built, in an attempt to discover why the settlements predicted prior to construction had been so much greater than the observed values. They found that the largest strain predicted below the foundation was only 0.045%, and the stiffness was still almost linear at this stage. Thus, a very good (though slightly unconservative) prediction would have been obtained using $E_s$ without any reduction for strain level. Even in other cases, the use of $G_s$ (or $E_s$)

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directly in calculations of deformations will give a lower bound to the deformations, and this in itself can be very useful.

However, in most cases, it is necessary to use an appropriate modulus degradation curve in order to obtain a reasonable prediction of settlement. A first-order approximation would be to assume a hyperbolic curve, but as it has been demonstrated earlier, this could also be very unconservative in many cases.

4.2 Determining the modulus degradation curve using the SBP test

The proposed method of describing the modulus degradation curve has been outlined in Section 3.1 above. In effect, it requires choosing values of the parameters \( f \) and \( g \) in Equation 6.

Logically, this requires a test that imposes larger strains than are imposed in seismic tests. The pressuremeter test, and particularly the self-boring pressuremeter (SBP) test, is the most ideal test for this purpose. A detailed method of interpreting the SBP test in sand to provide the required information has been described by Fahey et al. (1994) and Soliman and Fahey (1995). The method is based on fitting an SBP test curve that includes at least two unloading-reloading loops with a finite element model incorporating the non-linear model. The details of the modelling were given by Fahey and Carter (1993).

If the disturbance due to pressuremeter insertion was minimal, there would be no need to use more than the initial loading section of the test curve. Figure 7 shows typical model outputs for the initial loading section of an ideal SBP test, where the in situ values of \( \sigma'_{uc} = \sigma'_e = 100 \) kPa, and \( G' = 100 \) MPa. The three curves were obtained by using \( g \) values of 0.1, 0.6 and 3.0 with \( \phi' = 45^\circ \) and \( v = 15^\circ \). One case, with \( g = 0.3 \) and \( \phi' \) reduced to 40\(^\circ\) and 10\(^\circ\), respectively, is also shown. Note that the value of \( f \) has very little effect on the shape of the curve at the stage of the test, and so \( f \) can be used for this fitting.

This figure shows that under ideal circumstances, with a "perfect" SBP pressure-expansion curve and assuming \( G' \) is known, it should be possible to determine the value of \( g \) with reasonable certainty, especially if the later part of the test is used to determine the values of \( \phi' \) and \( v \). The results show that a change in \( \phi' \) and \( v \) values does not have too much effect on the shape of the curve at this stage, so that even approximate values would suffice.

This process has been followed using the results of a SBP test in dense sand in Perth. The initial part of the pressure-expansion curve for this test is shown in Figure 8. The in situ pore water pressure has been subtracted from the total cavity pressure, and the strain has been zeroed at the best estimate of in situ horizontal effective stress. Seismic CPT tests at the site gave a \( G' \) value of about 230 MPa, and the gradient corresponding to this is shown as the dashed line in Figure 8.

The initial gradient of the pressuremeter curve is consistent with the independently-measured value of \( G' \), but clearly the value of \( G' \) could not be reliably determined from the pressure-expansion curve, because of "noise" or scatter in the data.

The model output shown in Figure 8 was obtained using the measured value of \( G' \), and a value of \( g \) of 0.24. A value of \( f \) less than 1 was used (i.e. 0.92) to force the stress state onto the Mohr-Coulomb failure surface. This was required because the same model was also used to fit all of the rest of the test, as discussed below. As already stated, this difference in the value of \( f \) has very little effect at the initial stage of the test.

![Figure 7](image1.png)

**Figure 7.** Initial loading section of pressuremeter expansion curves determined using FEM model, for various values of parameter \( g \) in Equation 6.

![Figure 8](image2.png)

**Figure 8.** Initial loading section of SBP test in sand fitted with FEM model.
This process seems from this example to be reasonably robust. However, in many cases, the initial loading part of the pressuremeter curve is affected by insertion disturbance to a greater extent than in this case. To allow for this, Fahey et al. (1994) proposed matching the behaviour in unload-reload loops as a more rigorous method of calibrating the model. A similar approach has been proposed to varying degrees by Byrne et al. (1990), Bellotti et al. (1990), Muir Wood (1990), Robertson and Ferrara (1992), Tsuchiushi et al. (1994) and Jardine (1992).

The complete pressure-expansion curve for the test presented in Figure 8 is shown in Figure 9. The test incorporated 3 unload-reload loops, with the "loop depth" (the pressure reduction in the loop Ap) chosen according to the criterion proposed by Fahey (1991):

\[ Ap = \frac{\sin \phi'}{1 - \sin \phi'} \phi' \]

(10)

where \( \phi' \) is the effective cavity pressure at the start of the loop, and \( \phi' \) is assumed to be 40°. The equivalent criterion in cohesive soils would be to unload by a pressure equal to the estimated shear strength. Note that sufficient time is allowed for creep to be completed before starting each loop. This is necessary even in sands, otherwise the creep will distort the initial part of the unloading curve.

It should be noted that using the above "loop depth" criterion tends to give almost identical secant unload-reload shear modulus values \( G_s \) for each loop, as shown in Figure 10(a), in which the three loops are superimposed at an enlarged scale, and the pressure and strain are measured from the values at the start of each loop. In this case, this gives a \( G_s \) value of about 75 MPa from each loop. The importance of this is that provided a consistent loop depth criterion is used, the derived value of \( G_s \) is independent of the stage in the test at which the unload-reload loop is carried out.

The model output that best fits the whole expansion curve is also plotted in Figure 9, superimposed on the test curve. Note that the model does not incorporate a creep component, so this aspect is not modelled at all. Note also that the reloading part of each loop is not very well modelled. This indicates that some of the assumptions made in the model for this phase of the test may not be correct. This is discussed in some detail by Fahey and Carter (1993). The poor fit to the final section of the loading curve could be due in part to the finite length of the SBP in the field test.

Nevertheless, the model does give a reasonable fit to the overall pressure-expansion curve, and more importantly, it gives a very good fit to the unloading section of each loop, as shown by the comparison between the test data and the model output for the unloading section of each loop in Figure 10(b).

This whole process is obviously quite a severe test of the model, since it must give a good fit to the initial "elastic" part of the pressure-expansion curve (Figure 8), to the overall curve (Figure 9), and to the unloading sections of the unload-reload loops.
4.3 Determining modulus degradation curve by other types of pressuremeter test

Unfortunately, the resources (equipment and personnel) required to carry out high-quality SBP testing are only available in some areas. It is therefore impractical to suggest that the approach outlined above will be routinely applied. Nevertheless, the principles behind the method can still be used. The main requirement, as already stated, is to use a test imposing larger strains on the soil than is the case in a seismic test. In some parts of the world, pre-bored pressuremeters (particularly Ménard pressuremeters) are widely used in site investigation practice. Though the creation of the borehole and the consequent stress relief and soil disturbance, are extra complicating factors, it should still be possible to carry out a full modelling of the pressuremeter curve from these types of instruments. This is the so-called computer aided modelling (CAM) method of deriving parameters from pressuremeter curves. Some of the abstracts of papers to this Conference appear to deal with this topic (eg. Blacez et al, 1990), but generally only linear elastic-plastic models are being used. If the test is combined with a measurement of \( G_s \) made using a seismic method, then non-linear elasticity could be incorporated into the process.

There are now also a number of "full-displacement" pressuremeters in common use, such as the cone pressuremeter (Faulder and Post, 1995) and the Apago mini-pressuremeter used in France. The cone pressuremeter, in particular, may be a very suitable alternative to SBP testing, since it also provides a cone resistance (\( q_c \)) value, which may also be useful in deducing the larger-strain behaviour, as discussed below. Taking this instrument one step further, and incorporating a seismic function in the cone to allow shear wave velocity measurement, would allow all the required measurements to be made with a single instrument. The full-displacement pressuremeter may also have an advantage over the pre-bored pressuremeters in that it may be possible to model the insertion process completely – in effect, the amount of disturbance due to insertion may be greater than for pre-bored pressuremeters, but it is well defined.

4.4 Indirect pressuremeter methods

The discussion on derivation of the modulus degradation curve from pressuremeter tests has so far been based on modelling the complete pressure-expansion curve using an appropriate non-linear model. This would require repeating the fitting process for each test, and this could be very time consuming. One possible alternative strategy is based on carrying out numerical parametric studies of pressuremeter tests in a particular soil type, including unload-reload loops, and using a consistent "loop depth" criterion (such as that suggested earlier). From the unload-reload loops in these "numerical experiments", the secant unload-reload shear modulus \( G_s \) can be determined. This is illustrated for one set of model parameters in Figure 11(b). Only the unload section of each loop is shown.

In this case, the input value of \( G_s \) is 100 MPa, with friction angle of 45° and dilation angle of 15°. The value of \( f \) was kept at 0.92, but the value of \( g \) was varied from 1 to 0.4, as shown in the legend. For \( g \) values of 1, 0.8, 0.6, 0.4 and 0.2, the derived values of \( G_s \) were 87, 77, 66, 49 and 30 MPa. These results are plotted in Figure 11(b) as \( G_s/G_0 \) versus the value of \( g \) used in the model run.

Figure 12 shows \( G_s/G_0 \) values obtained from SBP tests and seismic CPT tests at two soil sites in Perth, Western Australia, plotted against depth. Though the values are some higher than those for the SBP tests, they are based on a depth of 8 m, below which most of the values lie between 0.3 and 0.4. From Figure 11(b), this suggests the \( G_s/G_0 \) values should be about 0.3. This is close to the values obtained from the full fitting process for these tests.

Ghionna et al. (1994) give \( G_s \) values obtained from SBP tests and \( G_s \) values obtained from cross-hole seismic tests at a site in the valley of the Po River. In each SBP test, either two or three unload-reload loops were carried out. For each test, the results from all loops were quite similar (the loop depth criterion used in the tests was not stated). In Figure 13, the average value of \( G_s \) from each test, normalized by the \( G_s \) value from the equivalent cross-hole test, is plotted against depth. This shows a ratio that appears to decrease slightly with depth, but is generally between about 0.5 and 0.6. From Figure 14, this suggests a \( g \) value of about 0.5, or a slightly slower rate of modulus degradation than for the two sites in Perth (Figure 12). Bruzza et al. (1986) quote values of \( G_s/G_0 \) of 1.79 ± 0.30 for data from the
same site (in fact the same data, but over a greater depth range), which implies an average \( G_s/G_v \) value of 0.56, and a range from 0.67 to 0.48.

The difference in \( G_s/G_v \) ratios between the Perth and Po River sites could be due to the differences in geological age of the deposits, since the sands in the Perth sites are of Pliocene age (believed to be 50,000 to 100,000 years old; Seddon, 1972), while the Po River deposits are of Holocene age (i.e. less than 10,000 years old), according to Bruzzi et al. (1986). The discussion on the effects of age on \( G_s \) in Section 3.3 above indicated that in some cases, age difference alone is not sufficient to explain the high stiffness from some old sand deposits, and some slight cementation effect may also be involved.

This approach (using the \( G_s/G_v \) ratio to determine the value of \( g \)) obviously requires much more work to verify that it can be used as a general method, but it seems to offer considerable promise. Effectively, it suggests that the greater the rate of degradation of the stiffness ratio \( G_s/G_v \), the greater the difference between \( G_s \) and \( G_v \) obtained from a pressuremeter test, provided that \( G_v \) is obtained using some logical and consistent "loop depth" criterion. Though the data presented were obtained from SBP tests, the approach would be relevant for any type of pressuremeter test, including pushed-in instruments.

Other researchers have also studied the link between \( G_s \) and \( G_v \) from SBP tests either experimentally, numerically or analytically (e.g. de Alba et al., 1984; Bacso et al., 1991; Giannini et al., 1994; Bellotti et al., 1989; Byrne et al., 1990). However, in most cases, a hyperbolic model was assumed (i.e. \( f = g^{-1} \) in Equation 9), and the aim was to establish methods of deriving \( G_v \) from the SBP test. The key difference in what is proposed here is to measure \( G_s \) and use the ratio of \( G_s/G_v \) as an indication of the rate of modulus degradation with mobilised shear stress.

4.5 Other indirect methods

An approach similar to that outlined above could

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**Figure 11.** Model prediction of unload stiffness for various values of \( g \) in Equation 6, with \( G_v \) of 100 MPa in each case (a) complete unload curves, and (b) normalised \( G_s/G_v \) ratios versus parameter \( g \).

**Figure 12.** Ratios of \( G_s \) from SBP tests and \( G_v \) from SCPT test at two sites in Perth plotted against depth.

**Figure 13.** Ratios of \( G_s \) from SBP tests and \( G_v \) from cross-hole seismic test at the Po River site plotted against depth (data from Giannini et al., 1994).
also be developed for other types of tests. The most obvious tests to use are those that involve some form of direct measurement of stiffness. This requires developing relationships between the stiffness measured by the test and the small strain stiffness for soils with different rates of modulus degradation (for different values of $g$ in the model presented earlier).

4.5.1 Dilatometer test

The flat plate dilatometer test, or DMT (Marchetti, 1980), is particularly interesting for this application, since it provides a direct measurement of a stiffness (the "dilatometer modulus" $E_d$). To develop the relationship between $E_d$ and $G_s$ for soils with different rates of modulus degradation will require numerical modelling of the DMT to be carried out using a model such as that used by Fahey and Carter (1993) for the SPT test. Alternatively, an experimental approach could be used, in which $G_s$ measurements and DMT tests are carried out in soils with known modulus degradation behaviour.

Recent calibration chamber work by Bellotti et al. (1997) involved using both a standard dilatometer and a "research" dilatometer (RDMDT). The RDMDT incorporated direct measurement of the movement of the dilatometer membrane throughout the complete test, rather than just the two pressure/displacement measurements obtained from the standard DMT. With this device, the full loading curve, and unload-reload loops could be completely defined. Figure 14 shows typical dilatometer expansion curves for DMT and RDMDT tests in Toyoura sand, as reported by Bellotti et al. (1997). The standard DMT provides only the data points labelled A and B, but the RDMDT provides a complete curve.

The RDMDT clearly gives much more information about the detailed behaviour of the soil during the test. The shape of the RDMDT curve is very similar to the more familiar SPT pressure-expansion curves. The behaviour in the expansion part of the test clearly involves plastic behaviour, and hence the standard dilatometer modulus $E_d$ cannot be regarded as an elastic stiffness, since the shape of this part of the curve is a function also of the strength parameters ($q'$ and $v$). However, the unload-reload stiffness $E_{ur}$ is analogous to the pressuremeter $G_s$ value, and therefore it might be possible to use it in a similar way to that described earlier to determine the modulus degradation ratio, provided some logical criterion for choosing the loop "depth" is used.

Bellotti et al. (1997) explored the relationships between the values of $E_d$ and $E_{ur}$ from these tests and the small-strain shear moduli $G_s$. While the results are not conclusive, there is clearly the potential to develop a method of deriving the modulus degradation curve from this type of test.

Figure 14: Results of dilatometer tests in a calibration chamber (a) standard dilatometer and (b) "research" dilatometer (Bellotti et al., 1997).
different parameters to be used successfully outside the immediate context in which it was derived, that it should be:

- based on a physical appreciation of why the properties can be expected to be related;
- set against a background of theory, however idealised this may be; and
- expressed in terms of dimensionless variables so that advantage can be taken of the scaling laws of continuum mechanics.

In the case of SHP tests, there must be a clear and direct link between the unload-reload stiffness \( G_0 \) and the modulus degradation curve for the material, and therefore this test meets the Wentz criteria. The same is true for \( E_0 \) values from the "research" dilatometer used by Bellotti et al. (1997), and to a lesser extent to the \( E' \) values from the standard DMT test (and the \( E_0 \) value from the Ménard pressuremeter test).

However, for the CPT test, a theoretical analysis would indicate that the \( q_v \) value in sand depends on a complex way on the strength parameters, the interstitial stress (particularly \( \sigma_1' \)) and the stiffness, though of course these parameters are not completely independent of each other. Any method of deriving the larger-strain stiffness (i.e. the modulus degradation curve) from the ratio of \( q_v \) to \( G_0 \) would therefore probably have to take stress level and strength parameters into account.

A direct link between \( G_0 \), the N-value from the SPT test and the larger-strain stiffness is probably even more difficult to establish, and hence a method based on the SPT would be difficult to verify.

The attraction of the CPT test in this application is that it is possible to use a cone with a seismic capability, and thus the \( G_0 \) can be obtained from the same tests.

Various researchers have examined the relationship between \( G_0 \) and \( q_v \), and between larger strain stiffness for directly producing settlements and \( q_v \) also. The original calibration work in relating the \( q_v \) value to settlement in sand is due mainly to Schmertmann and colleagues (Schmertmann, 1970; Schmertmann et al., 1978), but this work was carried out in the calibration chamber, and hence the relationships obtained are relevant for clean "young" sands. Personal experience in the early 1980s with these relationships in the older sands of the Perth area showed that they gave settlements much greater than observed in practice. For these sands, the ratio of "working elastic stiffness" \( E' \) to \( q_v \) was much greater than the values proposed on the basis of the calibration chamber work.

Later work (in Italy in particular) showed that the ratio of \( E' \) to \( q_v \) depended on the stress history and the age of the deposit. The hypothesis advanced by Baldi et al. (1989) is expressed in Figure 15. This shows the \( E'/q_v \) ratio plotted against \( q_v/q_{\text{crit}} \). This is based on some data from the calibration chamber for normally consolidated (NC) and overconsolidated (OC) "young" sands, and a postulated relationship for "aged" NC sands. The dependence of \( E'/q_v \) on stress history and age appears logical. However, when the relationship between \( G_0 \) and \( q_v \) is examined, no such clear trend emerges from the early work. In fact, the suggestion is that \( G_0/q_v \) is only slightly dependent on stress history and age.

There is therefore not yet enough evidence on which to base a method of using the \( G_0/q_v \) ratio for deriving the modulus reduction rate for sands. Some very recent work at the University of Alberta has begun to provide the type of evidence required. Figure 16 shows a plot of \( G_0/q_v \) versus \( q_v \) for different sands, prepared from data provided by Robertson (1997). The parameter \( q_{\text{crit}} \) is a normalised version of the ratio \( q_v/q_{\text{crit}} \), used previously, defined as

\[
q_{\text{crit}} = \frac{q_v}{q_{\text{crit}}}
\]

where \( p_a \) is the atmospheric pressure.

These data were plotted in this way by Robertson to indicate "anomalies" soils - i.e. soils that fall outside the broad band indicated on the Figure. However, they might also be interpreted as indicating different ratios of small-strain stiffness \( G_0 \) to larger strain stiffness, or different modulus degradation rates.

![Figure 15. Ratio of "working strain" stiffness \( E' \) to \( q_v \) versus \( q_v/q_{\text{crit}} \) for sands with different stress histories (Baldi et al., 1989).](image-url)
This is clearly only a very tentative interpretation at this stage. This is therefore an area that requires much further research. Similarly, for stiff clays, the link between $G_vq_v$ and the shape of the modulus degradation curve needs to be explored if progress is to be made in using this approach for foundation settlement in this type of soil.

For other penetration tests, notably the SPT test, the situation is no different, though the establishing any theoretical basis for a link between $G_v$, SPT $N$ value and a large-strain stiffness (or a modulus degradation curve) is obviously even more difficult than for the CPT. Thus, it is more difficult to see how the Wroth (1984) criteria can be met for this application of this test.

4.6 Direct correlation methods for stiffness determination

Most of the traditional settlement prediction methods based on in situ tests used in the past rely on correlations either between measured parameters and settlement, or between the same parameters and stiffness parameters ($E$, $G$ etc) which are then used in the settlement calculations. The theme of this paper so far has been that since the behaviour of practically all soils is non-linear, even in the "elastic" (ie. prefailure) part of the stress-strain curve, any method of deformation prediction that does not take this into account is bound to be of only limited applicability. A review of these methods is outside the scope of this paper, but a suggestion that arises from the previous discussion is that these methods should be re-examined in the context of non-linear elasticity, to determine if better correlations might be possible if the strain levels of interest in particular problems, or in different zones for the same problem, are taken into account.

Thus, for soil layers in which the strain levels are low, the ratio of $E$ to $q_v$ or $E$ to $N'$ would be greater than in layers where the strain levels are high. It certainly is not logical to use the same correlation for all layers irrespective of expected strain level.

5. MEASUREMENT OF IN SITU STRESS

Unlike the area of deformation prediction, it appears that much slower progress is being made in the area of determination of in situ horizontal effective stress ($\sigma_{zh}^e$) in soils (or in determination of $K_v$, the ratio of $\sigma_{zh}^e$ to $\sigma_{xx}^e$). Jamieson and Lo Presti (1994) summarised the situation for cohesionless soils by stating that "the present state of the art in evaluation of $\sigma_{zh}^e$ is far from being satisfactory". This echoes the comment by Simpson et al. (1979), who, after reviewing the laboratory and in situ methods of determining $\sigma_{zh}^e$ for stiff clays, concluded that "...of all the approaches available, none stands out as exceptionally promising".

One of the major problems with measurement of in situ horizontal stress is that generally there is no independent way of determining the correct value, and that there is no way of verifying that the measured value is correct. Furthermore, in overconsolidated soils, even when there is a well-understood geological history, there is no guarantee that $K_v$ varies smoothly with depth, or is constant laterally even within the same local area. That, when measurements show scatter in $K_v$ values, this may be a reflection of true scatter in this parameter, or may be a reflection of the inadequacy of the measurement technique.

5.1 Direct measurement of $\sigma_{zh}^e$

Of the in situ methods, the SBB test still appears to be the most accurate "direct" method of determining $\sigma_{zh}^e$. In this context, "direct" means that an attempt is made in this test to measure $\sigma_{zh}^e$ directly, rather than inferring it from other observations. In his review of geocomposite testing in clays, Bhonsle (1995) states that "of all the methods available to evaluate the in situ horizontal stress in clays, the SBB remains the tool of reference".

Some of the poor results obtained with this method in stiffer material may have been due to problems with instrumentation and system compliance. Some partial explanation of what happens with the SBB with Cambridge-type "tulip arms" for measuring displacement was provided by...
Whittle et al. (1995). The instrument they used had six displacement transducers placed around the circumference at the mid-height of the expanding section, instead of the usual three. Figure 17(a) shows the output from each of the six arms plotted as pressure versus radial displacement, with the displacement zero being shifted for each arm for the sake of clarity. The arms are numbered from 1 to 6 sequentially around the circumference, so that arm 4 is opposite arm 1, arm 5 is opposite arm 2 and arm 6 is opposite arm 3. Clearly, interpretation of individual arms would not give a very clear indication of the horizontal total stress, but when opposite pairs of arms are averaged, the situation is much clearer, as shown in Figure 17(b). This shows that the reference point for measurement of movement – i.e., the body of the instrument – can move, but by averaging opposing arms, this is allowed for and a clearer picture emerges.

For soft clays, the standard SBP, with a reasonable amount of care during insertion, can give what appear to be very good measurements of in situ horizontal total stress from the “lift-off” pressure. The “quality” of the insertion process can be judged to some extent by the pore pressures generated during insertion (Fahey and Lee Goh, 1993), and this should be done in soft or even stiff clays if measurement of horizontal stress is a primary aim of the test. At very least, it will indicate what value of pore pressure to subtract from the lift-off pressure to calculate the horizontal effective stress.

However, in spite of the apparent success with measurement of $\sigma_{ho}$ in soft clays, there is no consensus about the reliability or accuracy of the SBP for this purpose in stiff clays, or sand. From a theoretical point of view, some scepticism is warranted, particularly in coarser grain materials (sands). The idea that we can insert any instrument into a stiff material without causing any stress change in that material is very difficult to accept.

As an illustration of the magnitude of the problem, consider a soil with a $G$ value of 100 MPa and a hyperbolic stress-strain curve. Using the Fahey and Carter (1993) model described earlier, a pressuremeter test was modelled in which the insertion process resulted in a reduction in horizontal stress from the assumed in situ value of 100 kPa to 60 kPa. The expansion from this point was then modelled. Figure 18 shows the output (plotted as cavity pressure versus radial movement of the cavity wall in $\mu$m). The important points to note are:

- an inward movement (“disturbance”) of only 11 $\mu$m (assuming an 80 mm diameter pressuremeter) is all that is required to allow the pressure to drop by 40 kPa;
- on reloading, the “lift-off” pressure would indicate a horizontal stress that is much lower than the true value;
- there is no way of determining from the reloading curve when the initial in situ horizontal stress has been reached.

Thus, in this case, even with a radial disturbance of only 11 $\mu$m, any possibility of identifying the in situ horizontal stress has been lost. Since this movement is about an order of magnitude smaller than the typical particle size in a fine sand deposit, it is impossible to imagine that the disturbance could be any less than this. The opposite type of disturbance – causing a radial movement outwards due to the drilling process – is equally likely, and this would also prevent the identification of in situ horizontal stress from the lift-off pressure.

Figure 17. Lift-off behaviour for 6-arm SBP plotted as (a) individual arm outputs, and (b) pairs of arms averaged

Figure 18. FEM modelling of drilling “disturbance” (unward movement of 11 $\mu$m) followed by cavity expansion.
Most other in situ test methods that provide a value of $\sigma_0$ do so on the basis of correlation. Jamiołkowski and Lo Presti (1994) briefly reviewed the DMT method of measuring $K_0$ and concluded that there were many uncertainties in the method.

For cohesive soils, there are a number of methods that rely on laboratory tests (measurement of suction for example), and these certainly provide some additional useful information.

The conclusion is that the best that can be done in this area with conventional methods is to use as many methods as are available, including knowledge of the geological history of the deposit, and derive an estimate of $K_0$ on the basis of all of this information.

5.2 Deriving $\sigma_0$ using seismic methods

Among others, Roesler (1979) showed that the shear wave velocity in soil depends on the normal effective stresses in the direction of wave propagation and the direction of particle motion, and hence the deduced value of $C_s$ is also dependent on these stresses. Thus,

$$V_s = C(c_i^{\sigma})^{-\alpha} (\sigma_i^{\sigma})^{-\beta}$$  \hspace{1cm} (12)

where $C$ is a constant, and $c_i^{\sigma}$ and $\sigma_i^{\sigma}$ are the normal effective stresses in the direction of wave travel and the direction of particle motion, respectively, and $\alpha$ and $\beta$ are exponents, the values of which must be determined experimentally.

For the case where the horizontal stress is isotropic but is not equal to the vertical stress, we can therefore expect to have three different values of $V_s$:

$V_s^{ha}$, the velocity of shear waves travelling vertically, with particle motion in any horizontal direction;  
$V_s^{ha}$, the velocity of shear waves travelling horizontally, with particle motion in the vertical direction; and  
$V_s^{ha}$, the velocity of shear waves travelling horizontally, with particle motion in the orthogonal horizontal direction.

Since techniques are available for generating all three types of shear waves in situ, this anisotropy of $V_s$ offers the possibility of investigating the anisotropy of in situ stress. Assuming that the constant $C$ in Equation 12 is the same for all three types of shear waves, then:

$$\frac{V_s^{ha}}{V_s^{h}} = \left( \frac{\sigma_i^{\sigma}}{\sigma_i^{\sigma}} \right)^{\alpha} = \left( \frac{\sigma_i^{\sigma}}{\sigma_i^{\sigma}} \right)^{\alpha} = (K_0)^{\alpha} \hspace{1cm} (13)$$

which gives

$$K_0 = \left( \frac{V_s^{ha}}{V_s^{h}} \right)^{\frac{1}{\alpha}}$$  \hspace{1cm} (14)

However, this assumption about $C$ may not be valid, due to the possible influence of soil "fabric" on the shear wave velocity — i.e. even with an isotropic stress state, the various shear wave velocities could vary due to anisotropic soil "fabric". Jamiołkowski and Minassian (1995) therefore gave a more general form of Equation 12:

$$K_0 = \left( \frac{C_{h\sigma}}{C_{h\sigma}} \right)^{\frac{1}{\alpha}}$$  \hspace{1cm} (15)

where $C_{h\sigma}$ and $C_{h\sigma}$ are the constants in Equation 12 corresponding to $V_s^{ha}$ and $V_s^{h}$ respectively. Similarly,

$$\frac{V_s^{ha}}{V_s^{h}} = C_{h\sigma} (c_i^{\sigma})^{\alpha} (\sigma_i^{\sigma})^{\beta}$$

$$\frac{V_s^{ha}}{V_s^{h}} = C_{h\sigma} (c_i^{\sigma})^{\alpha} (\sigma_i^{\sigma})^{\beta} = \frac{C_{h\sigma}}{C_{h\sigma}} (K_0)^{\alpha} \hspace{1cm} (16)$$

which gives:

$$K_0 = \left( \frac{V_s^{ha}}{V_s^{h}} \right)^{\frac{1}{\alpha}}$$  \hspace{1cm} (17)

Thus, this approach for determination of $K_0$ is not as straightforward as it appears, since it requires an independent method of determining the values of $\alpha$, $\beta$, $C_i$, and $C_{h\sigma}$.

Becher and Powel (1955a, 1955b) report the results of tests at various sites in the UK and Norway, in which various techniques were used to determine shear wave velocities in situ. At the overconsolidated clay sites at Madingley and Chatenden, they used a seismic CPT and vertically and horizontally polarised shear waves in cross-hole tests, to derive the various values of shear wave velocity. (They also used surface Rayleigh waves to determine the Rayleigh wave velocity, denoting this $V_{R}$. The results are shown in Figure 19.

For both of these sites, the values of $K_0$ were claimed to be known with reasonable certainty from other tests. This allowed the various constants in the shear wave velocity equations to be investigated.

Firstly, by plotting $V_s^{ha}$ against $\sigma_0$ on logarithmic axes, it was found that a linear relationship applied for each site. Since

$$V_s^{ha} = C_{h\sigma} (\sigma_0)^{\alpha \beta}$$  \hspace{1cm} (18)

the gradient of these lines gives the value of $2\alpha$ for 0.62 and 0.50 for $\sigma_0$ for the Madingley and Chatenden sites, respectively. Using these values, and the relationship $2\alpha = 3\beta$ obtained by Roesler (1979)
for granular soils, gave a reasonably good normalisation of all of the $V_p$ data for each site, but with different relationships evident for the two sites. A better normalisation for each site was obtained using $n_a = 3 n_b$, as shown in Figure 20(a), but still the data for the two sites were different. However, by using the value of $n_a/n_b = 0.5$ for the two sites, they found that the data for the two sites were in reasonable agreement, as shown in Figure 20(b).

From all of this, it is clear that in situ
measurement of shear wave velocities $v'_s$, $v'_p$, and $v'_n$ offers very exciting potential to investigate the in situ stress state and fabric effects in soils. However, much more work is clearly required in this area before a workable method of deriving $K_s$ can be derived, particularly to determine how to separate out the effects of stress state and fabric.

6. SUMMARY AND CONCLUSION

Predictions of soil deformations under the influence of foundation (and other) loads have been generally found to be of very limited accuracy, even for stiff soils in the “elastic” or pre-failure range of strain. A major reason for this has been that the non-linearity of the stress-strain response in this strain range has not been taken into account until recently.

There is now the opportunity to make a dramatic improvement in methods of deformation prediction. This arises because of a general acceptance of the importance of non-linearity at small strains, and the ability to determine the initial tangent stiffness $G_t$ by relatively simple in situ tests, particularly the seismic CPT (and also the non-intrusive surface wave tests, such as the SASW method).

This paper has outlined a framework for methods of deriving the parameters required for settlement prediction. These parameters describe the rate at which the normalised secant stiffness $(G/G_s)$ reduces with increasing shear stress. The methods are based on using some larger-strain test in conjunction with a direct measurement of $G_t$ to derive these parameters. The most suitable tests are those that directly measure a larger strain stiffness. Unload-reload modulus values $(G_s)$ from SPT tests, or other pressuremeter tests, or from a modified DMT test meet this requirement.

While more indirect methods may also be suitable, though the success of the process requires clearly-definable links between the measured quantity and the larger strain stiffness.

For materials that can be sampled without disturbance, all of the parameters required for non-linear analyses can be obtained from very careful laboratory tests, using appropriate instrumentation to measure the sample strains. Nevertheless, even for such soils, the approach described in the paper could be an alternative (or a complement) to the laboratory methods.

The topic of in situ stress measurement (i.e. determination of $K_s$) is also dealt with briefly in the paper. The conclusion is that even the best direct methods (including the SPT) have not proved to be universally successful in this application, particularly in stiff clays and sands. Some new methods that are based on the fact that shear wave velocity depends on the stresses in the direction of travel and in the direction of particle motion appear to be a very useful addition to the available techniques. However, there remain a number of questions to be answered with the seismic methods, particularly how the effect of soil “fabric” on shear wave velocity can be differentiated from the effects of the various principal stresses.

7. ACKNOWLEDGEMENTS

The pressuremeter and seismic CPT data presented for the two sites in Perth were obtained by Mr A.A. Soliman as part of his research work at the University of Western Australia. This work was carried out with the support of Main Roads Department of Western Australia (now MRWA) and the Water Authority of Western Australia. This support is gratefully acknowledged.

8. REFERENCES


Bellotti, R., Benoli, J., Fretti, C. and Jamiołkowski, M. (1998). Stiffness of Toyoura sand from very careful laboratory tests, using appropriate instrumentation to measure the sample strains. Nevertheless, even for such soils, the approach described in the paper could be an alternative (or a complement) to the laboratory methods.

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boring pressuremeter tests in Po River sand. Proc. 2nd Int. Symp. on the Pressuremeter and its Marine Applications, College Station, USA, 57–74, ASTM STP 950.


Marchetti, S. (1980). In situ tests by flat


Siddon, G. (1972). Sense of Place, a Response to an Environment: The Swan Coastal Plains, Western Australia. The University of Western Australia Press, Perth, Western Australia.


Analysis and use of CPT in earthquake and environmental engineering

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Charles E. Vai, Jr., Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, Va., USA

ABSTRACT: Of the widely used penetration tests, the Cone Penetration Test (CPT) has undergone the most significant development. There have been important improvements in instrumentation, data acquisition, and data processing capability, and in the analytical interpretation of CPT results. Predictions of cone penetration resistance have been based on bearing capacity theory, cavity expansion theory, steady state deformation analysis, finite element analysis, and the results of calibration chamber tests. Cavity expansion theory yields the most accurate and generally applicable results. Applications of the CPT in geotechnical earthquake engineering have included soil profiling, determination of properties for use in ground response and liquefaction analyses, and the design and evaluation of ground improvement. Liquefaction potential assessment curves based on cone resistance are supported by field data and theoretical analysis. In geoenvironmental projects, data are provided by several types of new sensors, and the CPT is used for site characterization, determination of groundwater flow conditions, assessment of contaminant types and distributions, measurement of hydraulic conductivity, and in the design of waste containment and remediation systems.

1 INTRODUCTION

Penetration tests of various types have provided the backbone of geotechnical site investigations for centuries. Probing and sampling are done by drilling, pushing, hammering, and projecting rods, tubes, and instrumented devices of different types into the ground to obtain samples and data that are interpreted for identification of soil types and their boundaries, the location of groundwater and the depth to rock, hydrostatic pressures, soil engineering properties, and, more recently the location and characterization of wastes, buried objects, and pollutants of various types.

1.1 Test types and their interpretation

Current penetration testing technology includes the Standard Penetration Test (SPT), for use in many soil types; the Becker Hammer Penetration Test (BPT), for use in gravelly and cobbly soils; the "static" Cone Penetration Test (CPT), for use in sands, silts, and clays; dynamic cone penetration test of various types; and other probes that may be used for special purposes. Some aspects of the SPT, CPT, and BPT are compared in Table 1.

The specific details of the testing apparatus and the procedures that are used for penetration tests can

<table>
<thead>
<tr>
<th>Soil Types</th>
<th>CPT</th>
<th>SPT</th>
<th>BPT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuous profile</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Soil sample</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Repeatability</td>
<td>Very good</td>
<td>Good</td>
<td>?</td>
</tr>
<tr>
<td>Sensitivity to profile changes</td>
<td>Very good</td>
<td>Fair</td>
<td>Fair</td>
</tr>
<tr>
<td>Empirical Correlations for properties</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Table 1. Comparisons of the CPT, SPT, and BPT advantages and limitations.

have significant influences on the responses that are measured. How the measured quantities; e.g., penetration resistance under a static or dynamic load, a sleeve friction, or a water pressure; are interpreted has a major impact on the conclusions concerning properties and probable engineering behaviour. Ac-
cordingly, recent research on penetration testing has led to development of new theoretical and empirical interpretation methods and innovative and improved apparatus and test procedures.

1.2 Scope of this paper

We focus on the cone penetration test for five reasons: (1) the CPT has, at least in North America, become the in-situ test of choice for those soil types and profiles that are suitable for its use, (2) alone among the penetration tests, the CPT provides a continuous record with depth, (3) the cone penetrometer has been improved continuously over time, in terms of both the probe and the ways in which data are obtained and interpreted, (4) cone penetrometer systems can incorporate additional sensors and test devices to provide useful information in addition to the penetration resistance, and (5) much of the data obtained by cone penetration testing is amenable to rational interpretation using both analytical methods and empirical correlations. An up-to-date, very comprehensive, and detailed compilation of information on CPT apparatus, test procedures, data analysis and interpretation methods, and applications of CPT results is contained in Lunne et al. (1997).

Two of the most important and more recently developed applications of the CPT are in geotechnical earthquake engineering and geoenvironmental engineering. Following a discussion of current capability for rational prediction of cone penetration resistance from known soil properties and the inverse problem of deducing soil properties from measured penetration resistance, we review and evaluate these two applications.

2 PREDICTION OF CONE PENETRATION RESISTANCE

Methods for the analysis of the cone penetration resistance of soils have been reviewed and evaluated recently by Yu & Mitchell (1996, 1998). Undrained penetration in clays and fully drained penetration of sands were considered. Five methods were examined: (1) bearing capacity theory, (2) cavity expansion theory, (3) steady state deformation, (4) incremental finite element analysis, and (5) calibration chamber testing. Details of the methods and a comprehensive discussion are given in Yu & Mitchell (1996), and a more concise presentation is given in Yu & Mitchell (1998). A considerably condensed version of the latter is given here, supplemented by some new results that provide additional insights.

2.1 Bearing capacity theory

In this approach the cone penetration resistance is considered to be equal to the collapse load of a deep, circular foundation. Both limit equilibrium analysis and slip line analysis have been used for determination of this load.

With limit equilibrium, a failure mechanism is assumed and global equilibrium of the soil mass is evaluated to determine the failure load. Some of the failure mechanisms that have been assumed are shown in Figure 1. In the slip line method, a yield criterion is combined with the equations of equilibrium to give a set of differential equations of plastic equilibrium. A limitation of bearing capacity approaches is that they do not account explicitly for the soil stress-strain and volume change behavior. Some bearing capacity solutions are compared in Table 2.

These solutions yield bearing capacity factors $N_q$ for clay, and $N_q$ for sand to use in

$$q_c = N_q q_s + p_o \quad \text{or} \quad q_c = N_q q_s + \sigma_u$$  \hspace{1cm} (1)

![Figure 1. Assumed failure mechanisms for deep penetration (adapted from Durgunoglu & Mitchell 1975).](image-url)
for clays, where \( \sigma_a \) is the undrained strength, \( p_a \) is the in-situ total mean stress, and \( \sigma_v \) is the in-situ total vertical stress, and

\[
q = N_v \sigma_v
\]

for sands, where \( \sigma_{vo} \) is the in-situ vertical effective stress. The cone factor \( N_v \) is a function of the failure zone geometry, and the factor \( N_v \) depends on the sand friction angle, soil-cone interface friction angle, and geometry.

### Table 2. Summary of some bearing capacity solutions for cone penetration resistance (adapted from Yu & Mitchell 1998). The notation used is given in Appendix A.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Authors</th>
<th>Methods and Assumptions</th>
<th>Cone Factor or Main Conclusions</th>
</tr>
</thead>
</table>
| Cohesive Soils      | Meyerhof (1961)           | Limit equilibrium analysis with the failure mechanism shown in Figure 1 (b). Plane strain solutions with an empirical shape factor to account for axisymmetric geometry. | \[
N_v = 1.15 \times (6.28 + \alpha + \cot \frac{\alpha}{2})^2
\]

| Cohesive Soils      | Durgunoglu & Mitchell (1975) | Limit equilibrium analysis with the failure mechanism shown in Figure 1 (d). Plane strain solutions with an empirical shape factor to account for axisymmetric geometry. | \[
N_v = 1.2 \times (2.443 + 3.303 \lambda + \sin \left[ (1 - \lambda) \frac{\pi}{2} \right])
\]


| Cohesive Soils      | Kounouo & Kuku (1982)      | Slip-line analysis with a non-standard cone in which the shaft diameter is smaller than that of the cone base. Axysymmetric analysis. | The cone resistance approaches a constant value once the depth of penetration is greater than the cone diameter. |

| Cohesite Soils      | Janbu & B. S. S. (1974)    | Limit equilibrium analysis with the failure mechanism shown in Figure 1 (c). Plane strain solutions. | \[
N_v = \frac{1 + \sin \theta'}{1 - \sin \theta'} \exp \left[ (\pi - 2\theta) \tan \theta' \right]
\]

| Cohesite Soils      | Durgunoglu & Mitchell (1975) & Chen & Huang (1996) | Limit equilibrium analysis with the failure mechanism shown in Figure 1 (d). Plane strain solutions with an empirical shape factor to account for axisymmetric geometry. | \[
N_v = 0.194 \exp(7.629 \tan \theta')
\]

| Cohesite Soils      | Sekoulevski (1965)         | Slip line analysis. Plane strain conditions. Axysymmetric and plane strain solutions. | \[
N_v = \frac{K \cos \delta_1 \cos (\alpha_2 + \delta_2) \left( 1 + \sin \theta' \cos (\Delta_2 + \delta_2) \right)}{\cos \delta_1 \cos (\alpha_2 + \delta_2) \left( 1 - \sin \theta' \cos (\Delta_2 - \delta_2) \right)}
\]

where \[
K = \exp \left[ (\pi - 2\theta) - (\alpha + \delta) - 2\theta - (\alpha - \delta) \right] \tan \theta'
\]

\[
\sin \Delta_2 = \frac{\sin \delta_2}{\sin \theta'}, \quad \delta = 1.2
\]

| Cohesite Soils      | De Sainne & Gola (1988)    | Slip line analysis. Axysymmetric and plane strain solutions. The cone factors for plane strain cases are much less than those for axisymmetric cases; the cone roughness has a significant effect on the value of the cone factor. | |


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2.2 Cavity expansion theory

The pressure required to produce a deep hole in an elastic-plastic medium is proportional to the pressure needed to expand a cavity of the same volume under the same conditions (Bishop et al. 1945). To use cavity expansion as a basis for prediction of cone resistance requires (1) development of limit pressure solutions for cavity expansion in a soil and (2) relating the expansion limit pressure to cone resistance. Some of the solutions that have been developed using this approach for both cohesive and cohesionless soils are summarized in Table 3. These solutions differ mainly in (1) the assumptions concerning the constitutive behavior of the soil in the elastic and plastic zones surrounding the expanding cavity, (2) assumptions concerning the contributions of spherical and cylindrical cavity expansion, and (3) the stress rotation from cylindrical cavity expansion to vertical penetration resistance.

The cavity expansion approach is more realistic than the bearing capacity method because (1) both elastic and plastic deformation of the soil during penetration can be taken into account using cavity expansion theory and (2) the cavity expansion approach can consider, at least in an approximate manner, both the influence of the cone penetration on initial stress states and the effect of the stress rotations that occur around the cone tip.

The solutions developed by Salgado (Salgado 1993; Salgado et al. 1997a) are based on the observation that a penetrometer advances by creating and expanding a cylindrical cavity in the soil. In the vicinity of the cone tip there are rotations of the stress, displacement, and strain fields. Penetration resistance is determined by calculating first the cylindrical cavity pressure and then performing a stress rotation to obtain $q_c$. The theory computes the penetration resistance in terms of the relative density of the soil, the lateral and vertical stress at the depth of interest, and the intrinsic strength and deformation parameters of the soil. The latter include the small-strain shear modulus, peak friction angle, and dilatancy angle of the sand.

2.3 Steady state approach

When viewed as a steady state of deformation process, cone penetration is analyzed as a steady state flow of soil past a fixed penetrometer. Solutions based on this approach are given by Baligh (1985). Hodesky et al. (1985), Teh (1987). Whittle (1992), and Yu et al. (1996a). Some of the solutions are summarized by Yu & Mitchell (1998). To date, this approach has been restricted to undrained penetration of clays.

2.4 Incremental finite element analysis

Both small strain (de Borst & Vermeer 1982; Griffiths 1982) and large strain (Budhu & Wu 1991, 1992, and Kousis & al. 1988 for clays; and Civitini & Gioda 1988 for sands) finite element models have been proposed. A more comprehensive model for both sands and clays has been given by van den Berg (1994). Because of numerical difficulties, there are uncertainties concerning the accuracy of cone factors derived by finite element analysis (Yu & Mitchell 1996, 1998).

2.5 Calibration chamber testing

Large calibration chambers have been used extensively in recent years to develop empirical correlations between cone resistance and sand properties. Some of the correlations are given in Table 4, where penetration resistance is shown as a function of relative density, friction angle, or state parameter. Because calibration chambers are necessarily limited in size, penetration test results are influenced by both size and boundary condition effects, which lead to different measured values of penetration resistance than measured in the field. The difference between the calibration chamber values and those determined in the field can be quite large, especially for small chamber to cone diameter ratios and dense sands. The cavity expansion solution developed by Salgado (Salgado 1993; Salgado et al. 1997b) provides a basis for quantification of chamber size effects.

2.6 Comparisons and evaluations for cohesive soils

Theoretical cone factors for clay derived from bearing capacity theory, cavity expansion theory, the strain path method, and the large strain finite element method were compared by Yu & Mitchell (1996, 1998). Values determined by bearing capacity theory show no influence of soil stiffness because bearing capacity theory does not take deformations into account.

Using the vane strength as a reasonable measure of the shear strength of normally consolidated clays, Lumme & Kleven (1981) found that the average value of the cone factor is about 15, with a range from about 11 to 19 for different clays in the field. Cone factors for overconsolidated clays are generally somewhat higher than for normally consolidated clays. The cone factors that agree most closely with these measured values are obtained using the Baligh (1975) cavity expansion theory, the Yu (1993) cavity expansion theory, and the van den Berg (1994) rough cone finite element analysis.
Results obtained by Kurup et al. (1994) from calibration chamber penetration tests in clay yielded cone factors from 14 to 16. These values agree with the values predicted by the Yu (1993) cavity expansion theory within 10 percent. The strain path factors obtained by Baligh (1985) are in the range of 16.3 - 17.5, and the van den Berg (1994) finite element method gives factors from 15.3 to 16.7, depending on the clay stiffness. The bearing capacity and strain path methods both under-predict the measured values.

### 2.7 Comparisons and evaluations for cohesionless soils

Evaluations of theories for penetration resistance of cohesionless soils using field test data are difficult owing to the heterogeneity of most natural deposits and the many uncertainties about their actual properties and stress states. Accordingly, chamber test results may provide a better basis for comparison; however, here, too, there are difficulties associated with chamber size and boundary condition effects, as discussed by Yu & Mitchell (1996, 1998).

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Authors</th>
<th>Methods and Assumptions</th>
<th>Cone Factor or Main Conclusions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesive Soils</td>
<td>Ludmiły &amp; Johnston (1974)</td>
<td>The normal stress on the cone face is assumed to be equal to that required to expand a spherical cavity from zero radius.</td>
<td>( N_c = 3.06 + 1.33 \ln \frac{G}{s_e} )</td>
</tr>
<tr>
<td></td>
<td>Vesic (1972, 1977)</td>
<td>The cone resistance is related to the spherical cavity limit pressure by a failure mechanism.</td>
<td>( N_c = 3.90 + 1.33 \ln \frac{G}{s_e} )</td>
</tr>
<tr>
<td></td>
<td>Baligh (1975)</td>
<td>The cone resistance is related to a combination of the work required to give the cone tip a virtual vertical displacement and the work done to expand a cylindrical cavity around the cone shaft.</td>
<td>( N_c = 12.0 + \ln \frac{G}{s_e} )</td>
</tr>
<tr>
<td></td>
<td>Yu (1993)</td>
<td>The cone resistance is obtained by combining the cylindrical cavity limit pressure with a rigorous plasticity solution for the steady penetration of an infinite rigid cone.</td>
<td>( N_c = 4.18 + 1.95 \ln \left( \sqrt{\frac{G}{2s_e}} \right) ) (smooth cone) ( N_c = 9.4 + 1.95 \ln \left( \sqrt{\frac{G}{2s_e}} \right) ) (rough cone)</td>
</tr>
<tr>
<td></td>
<td>Ludmiły &amp; Johnston (1974)</td>
<td>The normal stress on the cone face is assumed to be equal to that required to expand a spherical cavity from zero radius.</td>
<td>( N_c = \frac{1 + 2K_s}{3} \left( 1 + \sqrt{3} \tan^{-1}(\lambda \phi) \right) )</td>
</tr>
<tr>
<td></td>
<td>Vesic (1972, 1977)</td>
<td>The cone resistance is related to the spherical cavity limit pressure by a failure mechanism.</td>
<td>( N_c = \frac{1 + 2K_s}{3} \exp \left( \frac{\pi - \phi_e}{2} \tan \phi_e \right) \times \tan^{-1} \left( \frac{45 + \frac{1}{3} \lambda \phi_e}{\phi_e} \right) )</td>
</tr>
<tr>
<td></td>
<td>Chen &amp; Juang (1996)</td>
<td>The cone resistance is related to the cylindrical cavity limit pressure by an approximate slip line analysis.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Salgado (1993)</td>
<td>The cone resistance is related to the cylindrical cavity limit pressure by an approximate slip line analysis.</td>
<td>The cone factor cannot be expressed analytically and a numerical procedure must be used.</td>
</tr>
<tr>
<td></td>
<td>Salgado et al. (1997)</td>
<td>The cone resistance is related to the cylindrical cavity limit pressure by a simple failure mechanism.</td>
<td>( N_c = \frac{1 + 2K_s}{3} \frac{4}{3} (1 - \sin \phi_e) )</td>
</tr>
<tr>
<td></td>
<td>Yaoushka &amp; Hyde (1995)</td>
<td>The cone resistance is related to the cylindrical cavity limit pressure by a simple failure mechanism.</td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Summary of some cavity expansion solutions for cone penetration resistance (adapted from Yu & Mitchell 1998). The notation used is given in Appendix A.
Chamber test data for Ticino sand show that the bearing capacity solution of Durgunoglu & Mitchell (1975) predicts cone resistance well at low stress levels (shallow penetration), but seriously over-predicts the penetration resistance at higher stress levels, probably because the theory fails to account for sand compressibility. The cavity expansion correlation proposed by Yasufuku & Hyde (1995) over-predicts the cone factor for all values of friction angle and mean stress.

On the other hand, predictions of cone tip resistance using the Salgado et al. (1997a) cavity expansion theory, as incorporated in computer program CONPOINT, for four sands were found to be within ±30 percent of the values measured in about 400 tests on different silica sands tested over a range of relative densities and confining pressures in several different calibration chambers. Figure 2, from Salgado et al. (1997b), shows predicted and measured variations of \( q_c \) with lateral effective stress and relative density \( D_r \) for clean Hokkaido, Monterey, Ticino, and Toyoura sands. The experimental values of \( q_c \) were corrected to free-field conditions using the approach of Salgado et al. (1997a).

Penetration resistance is shown in Figure 2 as a function of lateral effective stress, because cylindrical cavity expansion pressure, and, therefore cone resistance, is more dependent on horizontal pressure than on vertical pressure. In practice, however, cone resistance is usually related to vertical effective stress, \( \alpha \). Accordingly, in-situ lateral stress, or the \( K_d \) conditions at a site can be expected to influence the results and interpretations of CPT tests, as shown by Salgado et al. (1997b).

Data points are shown in Figure 2 for the four test sands separated into four relative density ranges. Also shown are the theoretical lines for Ticino and Monterey sands for the lower end and the higher end, respectively, of each relative density range. Ticino sand, with the lowest friction angle of the four sands tested (34.8 degrees), yields the lowest \( q_c \) values, whereas, Monterey sand, with a friction angle of 37.0 degrees and a high shear modulus, gives the highest values. The theoretical lines are not expected to bound all the data, because if all data were to one side or the other of the lines, then the theory would be significantly and consistently over- or under-predicting the penetration resistance. That most of the experimental data fall between the lines corresponding to 80 percent of the theoretical values for Ticino sand and 120 percent of the theoretical values for Monterey sand is taken as evidence for good agreement between theory and measurement.

<table>
<thead>
<tr>
<th>Correlation Type</th>
<th>Authors</th>
<th>Methods and Assumptions</th>
<th>Cone Resistance or Cone Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative Density</td>
<td>Schmertmann (1976)</td>
<td>The cone resistance is assumed to be primarily dependent on the vertical stress and the soil density.</td>
<td>( q_c = C_0 (\sigma'_d)^{0.5} \exp(C_1 D_r) )</td>
</tr>
<tr>
<td>Relative Density</td>
<td>Hoehndy &amp; Hitchman (1988)</td>
<td>The cone resistance is assumed to be primarily dependent on the horizontal stress and soil density.</td>
<td>( q_c = C_0 (\sigma'_h)^{0.5} \exp(C_1 D_r) )</td>
</tr>
<tr>
<td>Relative Density</td>
<td>Jumahkowki et al. (1988)</td>
<td>The cone resistance is assumed to be primarily dependent on the mean stress and soil density.</td>
<td>( q_c = C_0 \sigma_m \exp(0.166 \phi' - 9) )</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>Hoehndy &amp; Hitchman (1988)</td>
<td>The cone resistance is assumed to be primarily dependent on the horizontal stress and the soil friction angle.</td>
<td>( q_c = \sigma'_h \exp(0.166 \phi' - 9) )</td>
</tr>
<tr>
<td>State Parameter</td>
<td>Been et al. (1987)</td>
<td>The cone resistance is assumed to be primarily dependent on the mean stress and soil state parameter.</td>
<td>( q_c = \frac{1 + 2 K_d}{3} \left[ 1 + k \exp\left( -u_d \right) \right] )</td>
</tr>
<tr>
<td>State Parameter</td>
<td>Yu et al. (1996a)</td>
<td>The cone resistance is assumed to be primarily dependent on the pressure-meter limit pressure (i.e., cylindrical cavity limit pressure) and soil state parameter.</td>
<td>( \frac{q_c}{\psi} = \exp(1.542 - 3.372 \psi) )</td>
</tr>
</tbody>
</table>
An additional example of the successful use of cavity expansion theory for prediction of penetration resistance is given in the subsequent section of this paper on CPT-based liquefaction triggering analysis. Detailed analysis by Sladen (1989) has shown that there is no unique correlation between the cone factor for sand and the state parameter proposed by Fellenz et al. (1987). However, the dependence of the cone factor-state parameter correlation on mean stress can be taken into account using the cavity expansion solution of Collins et al. (1992). Used in conjunction with the correlation suggested by Ladynyj & Johnston (1974), Table 3, there is good agreement with the measured cone resistance, except in very dense sand at very low mean stress (shallow cone penetration).

2.8 Conclusions about prediction of cone penetration resistance

Of the several approaches developed for prediction of cone penetration resistance, cavity expansion provides the most satisfactory and consistent results. It can be used for both sand and clay, it accounts for soil compressibility (or dilatancy); and it accounts for stress build-up around the penetrometer shaft during penetration.

![Graphs showing normalized lateral ESE stress versus normalized tip resistance](image)

- Data from calibration chamber tests on four sands
- $q_c$ from CONPOINT with parameters of Ticino (T) or Monterey No. 0 (M) sands
- $0.8 q_c$ for Ticino or $1.2 q_c$ for Monterey

Figure 2. Normalized tip resistance versus normalized lateral stress and $D_r$ based on both calibration chamber test results and CONPOINT. Intrinsic parameters of Ticino Sand (T), give lowest $q_c$ values on average, and Monterey No. 6 Sand (M), give highest $q_c$ values on average for all sands tested (from Salgado et al. 1997b).

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3.0 CPT TESTING AND INTERPRETATION FOR EARTHQUAKE ENGINEERING APPLICATIONS

Important applications of the CPT in geotechnical earthquake engineering include site characterization and soil profiling, assessment of liquefaction triggering potential, estimation of residual strength, acquisition of data needed for analysis of earthquake-induced ground deformations, and design and evaluation of ground improvement projects. Some special considerations in the use of cone penetration testing for these applications are discussed in this section.

Although not covered in detail in this paper owing to space limitation, we would like to call attention to the usefulness of shear wave velocity measurements that can be economically and reliably made by incorporation of a seismometer near the cone and friction sleeve assembly (Campanella et al., 1986). Seismic cone equipment and measurement procedures, examples of results and interpretations are given by Larson et al. (1997). Both cross-hole and down-hole measurements can be made, although the down-hole method is simpler and quicker. Shear wave velocity profiles obtained by seismic cone measurements can be used for several important purposes in geotechnical earthquake engineering including (1) soil profiling, (2) location of critical strata for sampling or further in-situ testing, (3) determination of small strain shear modulus profiles for use in ground response analyses, (4) liquefaction potential assessment, and (5) evaluation of the effectiveness of ground improvement.

3.1 Site profiling, soil identification and classification

Of the available penetration tests in common use today, the CPT is the only one that provides a continuous, detailed profile of the soil strata penetrated. The detection of relatively thin layers, that might otherwise be missed by SPT, but which may contribute significantly to earthquake-induced ground deformations is possible, as shown in Figure 4 from Boulanger et al. (1997). Unfortunately, however, the very significant advantages of continuous profiling and detection of thin layers are partially offset by the fact that no sample cores are obtained in the CPT for direct examination and soil identification.

To help overcome this limitation, a number of soil identification and classification charts have been proposed that are based on actual or normalized values of tip resistance, friction ratio, and pore water pressure; e.g. Douglas & Olsen (1981), Robertson et al. (1986), Robertson (1990), Jefferies & Davies (1991), and Olsen & Mitchell (1995). An example of these charts is given in Figure 5.

However, we argue most strongly against reliance on these charts in the absence of site specific validation of soil types by samples or by prior experience with similar soil conditions. There have been many cases wherein reliance on the classifications given by the charts in the absence of ground truth would have yielded incorrect results.

Although it has been widely recognized for many years that the cone friction ratio increases with soil fines content and plasticity, strong, direct correlations suitable for reliable determination of soil type and/or liquefaction potential assessment do not exist. This is well illustrated by Figure 6 from Suzuki et al. (1995) and by Figure 7 from Arango (1997).

The Suzuki et al. (1995) data show that for friction ratios between 0.25 and 1 percent, values that are typical for sandy soils, fines contents ranging from 2 to 50 percent were measured. Arango’s (1997) data are from sites in Indonesia, Dubai, San Francisco, New Mexico, and Trinidad, and they show a range of measured fines contents from 8.5 percent to more than 90 percent for the same range of friction ratios. Furthermore, close examination of the data points in Figure 7 shows that in one case fines contents in the narrow range of about 20 to 30 percent were associated with friction ratios ranging from about 0.25 to 6.0 percent, and in another, fines contents from about 6 to 95 percent were associated with friction ratios varying only from about 1.4 to 1.3 percent.
Figure 5. Soil classification based on cone penetration resistance (from Robertson 1990).

The lack of a strong fines content-friction ratio correlation is not surprising when the many factors in addition to fines content that are likely to influence the measured sleeve friction are considered. These factors include in-situ lateral stress, nature of the fines (e.g., no to high plasticity), soil structure, cone sleeve surface properties, and test procedural details.

3.2 Limitations of $q_c/N$ correlations

A number of very useful soil property and soil behavior correlations to the SPT $N$-value has been made over the many years that the SPT has been in routine use. As the CPT is a relatively “new kid on the block,” a suitable database for development of independent correlations based on the CPT tip resistance $q_t$ has not been available. Fortunately, this situation is now changing rapidly, as shown, for example, by the extensive data and correlations summarized by Luone et al. (1997).

A logical development, in the absence of independent correlations between $q_t$ and soil properties, was to try to relate $q_t$ to $N$-value, thus enabling the use of equivalent $N$-values and the SPT-based soil property correlations. In some cases, strong correlations between $q_c/N$ and mean particle size $D_{50}$ were obtained; e.g., as developed by Robertson & Cam-
Figure 8. $q_c/N$ correlations based on mean particle size (from Kulhawy & Mayne 1990).

Panella (1985) and Seed & DeAlba (1986). When a large number of data points from tests at many sites are examined, however, as shown by Figure 8 from Kulhawy & Mayne (1990), it may be seen that a rather broad band is defined. Similarly, Arango (1997) compared data for four sites with some proposed correlations and obtained the results shown in Figure 9.

It is evident from these comparisons that $q_c/N$ correlations based on mean particle size may involve significant uncertainty. Specific comparisons of pairs of $q_c$ and $N$ values can be erratic. Site specific $q_c/N$ values based on median values for relatively thick homogeneous layers may be more reliable. It is, nonetheless, very fortunate, as shown later, that the $q_c/N$ correlations developed by Robertson & Campanella (1985) and Seed & DeAlba (1986), when used to derive CPT-based liquefaction potential assessment curves from SPT $N$-value based curves, gave results that agree surprisingly well with curves derived from both field liquefaction data and theoretical methods that are independent of SPT penetration resistance measurements.

3.3 Influences of thin layers

The failure zone that develops around an advancing cone penetrometer extends to some distance radially and beneath the cone itself; therefore, the penetration resistance that is measured will be influenced by the strength and deformation properties of the soil at some distance from the cone. Thus, the measured value of $q_c$ may not give a true measure of the actual state of the soil at the elevation of the cone.

Both theoretical considerations (e.g., Vreugdenhil et al. 1994), and the results of experimental measurements (e.g., Trendwell 1976), show that both the depth above a layer boundary at which a cone "senses" an underlying layer of different properties and the magnitude to which the measured cone resistance differs from what would be measured in a homogeneous layer depend on at least the layer thickness to cone diameter ratio and the relative strengths and stiffnesses of the successive layers.

Figure 9. Relationship between mean particle diameter, cone tip resistance, and friction ratio for four sites (adapted from Arango 1997).
The results of penetration tests by Trendwell (1976) into layered samples of Monterey No. 0 sand are illustrative. Samples were placed in layers of uniform density by piling of gravel into a box having plan dimensions of 0.76 m by 0.76 m. The successive layers alternated between loose and dense. Penetration resistance versus depth is shown in Figure 10a for a loose layer sandwiched between two dense layers, and in Figure 10b for a dense layer sandwiched between two loose layers. Also shown for comparison are penetration resistance curves for tests into homogeneous samples of loose and dense sand for the full depth. The cone penetrometer used for these tests had a base diameter, B, of 20.3 mm.

From tests of this type, it was learned that the advancing cone sensed the layer of different density when it reached within about three to four cone diameters of the layer boundary. It was found also that the penetration resistance fell approximately to the loose layer value when passing from a dense layer to a loose layer. On the other hand, the higher penetration resistance of the dense layer was not reached when passing from a loose into a dense layer before the resistance fell again because of the influence of an underlying loose layer.

The results of tests using two different sizes of penetrometer in a five-layer system composed of alternating dense and loose layers are shown in Figures 11a and 11b. It is evident that the penetration resistance variation with depth determined by the penetrometer having a 10.1 mm base diameter was much more sensitive to the layer boundaries than obtained using a larger (20.3 mm base diameter) penetrometer. Furthermore, for the smaller penetrometer, the resistance measured in the dense layer reached a value much closer to that for a deep sand layer than obtained using the larger diameter penetrometer.

These results show again that in a stratified system, (1) the resistance measured in a loose or soft layer is likely to be reasonably close to what would be measured in a homogeneous loose or soft layer, but (2) the penetration resistance measured in a dense layer of limited thickness may be too low. Lunae et al (1979) note that the sphere of influence may be as much as 10 to 20 cone diameters in stiff materials, which means that a dense sand layer might have to be up to 0.75 m or more thick before full resistance is recorded. Even this could be insufficient if there is an underlying soft or loose layer.

Although it might be conservative for many applications to use the lower penetration resistance measured in a sand layer of limited thickness, there could be situations in liquefaction studies where this would not be true. For example, consider the soil classification chart previously shown in Figure 5. If the recorded value of \( q_u \) is too low, the friction ratio will
be correspondingly high, and, together with the low
\( u_c \), an incorrect soil classification could result sug-
gest that the soil is finer grained than it really is. This could lead to an unwarranted conclusion con-
cerning the soil behavior type (see next section) used
for assessment of liquefaction potential.
Figure 12. Suggested correction to CPT cone resistance in thin sand layers (from Lunne et al. 1997 based on Vreugdenhil et al. 1994).

A method, based on elastic theory, for correcting thin stiff layer data was developed by Vreugdenhil et al. (1994). This method is summarized by Lunne et al. (1997) using Figure 12. The variation of q_c with depth for a dense sand layer sandwiched between two softer layers is shown in Figure 12(b). The corrected value of penetration resistance q_c is obtained by multiplying the cone resistance q_c measured in the dense layer by the layer correction factor shown in Figure 12(a). The curve corresponding to q_c/q_f = 2 is recommended for conservatism.

3.4 Liquefaction potential assessment

One of the most significant recent advances in the application of cone penetration testing has been the development and acceptance of CPT-based charts for liquefaction potential assessment.

Owing to the lack of a suitable field database to enable direct correlations between CPT penetration resistance and liquefaction resistance, some early charts were developed based on SPT to CPT conversions, i.e., q_c/\bar{N}_60 ratios, that were a function of mean particle size. E.g., Robertson & Campanella (1985), Seed & DeAlba (1986). In these relationships q_c is the measured tip resistance and \bar{N}_60 is the SPT blow count, adjusted to a hammer efficiency of 60 percent. As noted earlier, these q_c/\bar{N}_60 ratios were relatively well defined, and the resulting liquefaction resistance curves have agreed surprisingly well with subsequent curves that have been proposed based on a greatly expanded number of field case histories and directly measured values of q_c.

For example, Figure 13, adapted from Ishihara (1993), shows the liquefaction resistance curves proposed by Robertson & Campanella (1985) and Seed & DeAlba (1996), as well as field data-based curves (Ishihara & Tepasakis 1988), for clean and silty sands. The average cyclic resistance ratio (CRR) on the right vertical axis is given by

\[
CRR = \left( \frac{q_c}{\sigma_0'} \right) = 0.65(\sigma_{om} / g)(\sigma_c / \sigma_0') \rho
\]

where \( q_c \) is average cyclic shear stress, \( \sigma_0' \) is the total overburden pressure, \( \sigma_0' \) is the effective overburden pressure, \( \sigma_{om} \) is the peak horizontal ground surface acceleration, \( g \) is the acceleration of gravity, and \( \rho \) is a stress reduction factor that decreases approximately linearly from 1.0 at the ground surface to a minimum of 0.5 at a depth of 30 m. The values for maximum cyclic resistance ratio on the left vertical axis are for maximum, rather than averaged, input ground motion and equal the values computed by equation (3) divided by 0.65.

To use Figure 13, the measured value of cone resistance is corrected and normalized. The normalization procedure recommended by the 1996 National Center for Earthquake Engineering Research (NCEER) Workshop for corrected penetration resistance and vertical stress is to use a dimensionless penetration resistance according to

Figure 13. Comparison between four CPT based charts for estimating cyclic resistance ratio (CRR) for clean sands (adapted from Ishihara 1993).
where:

\[ C_Q = \left( \frac{P_n}{\sigma_v' C} \right)^3 \]  

and \( P_n \) is atmospheric pressure in the same units as the cone resistance and overburden pressure.

An additional curve is shown on Figure 13 that was developed from the results of laboratory cyclic loading tests on four sands and corresponding penetration resistance values that were computed using cavity expansion theory and the known sand properties (Tseng 1989; Mitchell & Tseng 1990). This liquefaction resistance curve is completely independent of any CPT-SPT correlations or the uncertainties associated with interpretations of liquefaction observations in the field. It is considered that the reasonably close agreement among the derived and empirical curves in Figure 13 adds strength to CPT-based methods for liquefaction potential assessment, as well as providing an alternative method for evaluation of liquefaction potential.

Much additional field data has become available since the development of the curves shown in Figure 13, so it has been possible to develop improved curves. In particular, Stark & Olson (1995), Suzuki et al. (1995), and Suzuki et al. (1997) have compiled extensive databases, with results summarized in Figure 14, from Olson (1997). The earlier curves of Robertson & Campanella (1985) and Shibata & Topper's (1988) are also shown, as is the curve derived by Tseng (1989) using the results of laboratory cyclic loading tests and penetration resistance values computed using cavity expansion theory (Mitchell & Tseng 1990). The consensus recommendation for clean sand from a workshop held in January, 1996, under the sponsorship of NCEER is also superimposed on Figure 14. Experts at this workshop reviewed the “simplified procedure” for liquefaction potential evaluation and concluded that the CPT should be adopted as a primary tool for determining soil stratigraphy and liquefaction resistance.

The chart in Figure 14 is for magnitude 7.5 earthquakes and level ground conditions. Evaluation of liquefaction potential for other earthquake magnitudes is done using magnitude scaling factors, with recommendations in the NCEER Workshop consensus report (in press). Suzuki et al. (1997) accounted for magnitude differences by replacing the coefficient 0.65 in equation (3) by the quantity 0.14(M-1), where M is the earthquake magnitude.

As may be seen in Figure 14, the liquefaction curves for clean sands that have been developed by several investigators and data sets are in reasonable agreement. However, there is less consensus on the best procedures to account for the effects of fines, although it is generally agreed that soils containing fines have a lower penetration resistance for a given resistance to liquefaction than do clean sands.

One approach is to increase the normalized penetration resistance, \( q_{cf} \), of a sand with fines content greater than 5 percent by a factor, \( K_{CF}q_{sdr} \), to give an equivalent clean sand value (NCEER 1997; Robertson & Wilde 1997)

\[ q_{cf} = q_{sdr} + \Delta q_{sdr} \]  

where

\[ \Delta q_{sdr} = K_{CF}(q_{sdr} - q_{sdr_{clean}}) \]  

which may be written as

\[ \Delta q_{sdr} = K_{CF}(1-K_{CF})(q_{sdr} - q_{sdr_{clean}}) \]  

The correction factor \( K_{CF} \) is given by

- \( K_{CF} = 0 \) for AFC≤5%
- \( K_{CF} = 0.0257(AFC - 5) \) for 5%<AFC≤35%
- \( K_{CF} = 0.80 \) for AFC>35%

AFC is an "apparent fines content" that is intended to account for the fact that the measured penetration resistance is influenced by factors in addition to the actual fines content, such as the nature of the fines (mineralogy, plasticity), stress history, and soil structure. Robertson & Wilde (1997) propose that the combined effects of all these factors
be accounted for through use of a “soil behavior type index”, \( I_c \). The index \( I_c \) was proposed by Jeffers & Davies (1993) as a basis for quantitative description of the soil behavior types in the CPT chart developed by Robertson (1990) previously shown in Figure 5.

\[
I_c = \left(3.347 - \log(t)\right)^3 + (\log F + 1.23)^4
\]  

(9)

where \( t \) and \( F \) are the normalized penetration resistance and friction ratio;

\[ Q = \left(\frac{q_s - \sigma'_{w}}{P_2} \right)^n \]  

(10)

\[ F = \left(\frac{f_s}{q_s - \sigma'_{w}}\right) \times 100\% \]  

(11)

in which \( f_s \) is the sleeve friction, \( n \) is typically equal to 1, \( P_2 \) is a reference pressure in the same units as \( \sigma'_{w} \), e.g., 100 kPa if \( \sigma'_{w} \) is in kPa, and \( P_{eq} \) is a reference pressure in the same units as \( q_s \) and \( \sigma'_{w} \).

Robertson & Wride (1997) propose the following correlation between AFC and \( I_c \):

\[
I_c < 1.26 \quad AFC = 0 \\
1.26 < I_c < 3.5 \quad AFC (\%) = 1.75 I_c^{0.25} - 3.7 \\
I_c > 3.5 \quad AFC = 100\%
\]

Based on CPT data from four earthquakes and 68 sites in Japan, Suzuki et al. (1997) were able to show a clear boundary between liquefaction and no liquefaction if the soil characteristics are taken into account using the soil behavior type index. The clear separation of data points into liquefaction and no liquefaction is shown Figure 15. The adjusted tip resistance \((q_t)_a\) is defined by

\[
(q_t)_a = q_t \times F(I_c)
\]  

(12)

where \( F(I_c) \) is given by:

\[
\begin{array}{c|c}
I_c & F(I_c) \\
\hline
<1.65 & 1.0 \\
1.8 & 1.2 \\
1.9 & 1.3 \\
2.0 & 1.5 \\
2.1 & 1.7 \\
2.2 & 2.1 \\
2.3 & 2.6 \\
2.4 & 3.5 \\
\end{array}
\]

Suzuki et al.’s (1997) curve from Figure 15 has been superimposed on the other proposed curves in Figure 14, and it may be seen that this curve is slightly more conservative than the NCEER recommended curve.

An advantage of both Robertson & Wride’s (1997) and Suzuki et al.’s (1997) methods is that liquefaction potential can be estimated on the basis of CPT data only, so the need for determination of fines content is eliminated. Ohtsu (1997) has developed an equation for computation of the cyclic resistance ratio (\( CR_{10} \)) directly from the CPT data without the need for reference to charts similar to Figure 14. His equation, for M=7.5 earthquakes, is

\[
CR_{10} = (0.0128q_s)(\sigma'_{w})^{0.7} - 0.025 + 0.17R_f \\
- 0.028R_f^2 + 0.0016R_f^3
\]  

(13)

in which \( R_f \) is the friction ratio in percent, and \( q_s \) and \( \sigma'_{w} \) are in atmospheres (1 atm = 1 tsf = 100 kPa).

We reiterate, however, that to use only CPT measurements without obtaining samples for verification of soil types and to ensure that the measured values of tip and sleeve resistance are consistent with these soil types is seldom warranted.

3.5 Post-liquefaction residual strength

Reliable estimation of the post liquefaction residual strength of sandy soils and the cyclic load degradation of the strength of cohesive soils remain among the most difficult determinations in geotechnical earthquake engineering. The widely used SPT N-value based correlation of Seed & Harser (1990) has

Figure 15. Field correlation between shear stress ratio and adjusted tip resistance. Data are from 68 sites and four earthquakes in Japan (adapted from Suzuki et al., 1997).
ducting research on the development of a vibratory plezocone. Among the planned measurements is the comparison of the penetration resistances of partially or fully liquefied sands with the undrained post-liquefaction strength measured separately.

3.6 CPT testing and site remediation

The CPT offers one of the most reliable methods for use in design and for evaluating pre- and post-ground improvement conditions at sites where mitigation of liquefaction or other types of earthquake-induced ground failure are likely. Because of the relatively low cost of CPT testing and the continuous records that are obtained, it is possible to use a large number of tests and to assess the uniformity of treatment, especially deep densification, in considerably more detail than practical by direct sampling or other in-situ test methods. Numerous case history descriptions are available in the literature.

A recent example that illustrates clearly the improvement in properties is shown in Figure 16. Vibrocompaction was used to densify the sand hydraulically fill at the new Chek Lap Kok Airport in Hong Kong (Ng et al. 1996). The initial relative density of the fill was 30 to 40 percent, and the target relative density following treatment was 60 to 80 percent, as indicated by a cone tip resistance of 15 MPa. The substantial increases in cone resistance that developed as a result of treatment, and the continuing increase with time, are easily seen.

When interpreting the results of deep densification or other forms of ground treatment, it is necessary to keep in mind that CPT soil classification charts and liquefaction potential charts may not be directly applicable to post-treatment conditions owing to the changed ground conditions, especially the in-situ lateral stress. Salgado et al. (1997b) give a simple procedure for quantifying the effect of an increase in $K_c$, on the liquefaction correlation between the cyclic resistance ratio and the CPT penetration resistance of clean, unconfined sand.

Careful examination of pre- and post-treatment tip and sleeve resistance values (and pore pressures, if available) may allow evaluation of low penetration resistance zones to determine if they are caused by soil type differences or inadequate improvement.

The results of seismic cone measurements may be particularly useful for obtaining values for the dynamic properties of improved zones for use in dynamic response and deformation analyses.

4 CPT TESTING AND INTERPRETATION FOR GEOENVIRONMENTAL APPLICATIONS

The principal uses of the CPT for geoenvironmental purposes are for (1) site characterization, (2) deter-
mination of groundwater flow conditions, (3) assessment of contaminant types and distributions, (4) measurement of hydraulic conductivity, and (5) design of waste containment and site remediation methods. Each of these applications is discussed in this section.

4.1 Site characterization

The demonstrated usefulness of the cone penetrometer to characterize sites for conventional geotechnical engineering projects has been extended to site characterization for geoenvironmental applications. The major factors to be determined in environmental site characterization include:

- Soil types
- Thickness and lateral extent of soil layers
- Location of the groundwater table
- Location of bedrock

Issues in the determination of soil types and soil classification based on the results of CPT for applications in geotechnical engineering were discussed in Section 3.1. Classifications of the type shown in Figure 5 that are based on tip resistance, friction ratio, and pore pressure values, are useful, but require ground truth validation by samples, other identification methods, or prior experience at every site.

Recent developments in CPT technology may increase the accuracy of soil type determination using the CPT alone. The “vision cone penetrometer” or VisCPT allows real-time observation of the soils being penetrated (Raschke et al. 1997). The VisCPT has high and low magnification miniature CCD cameras located in a module behind the cone penetrometer. Illumination is provided for the cameras by light emitting diodes (LED’s). The VisCPT allows particle sizes from coarse sand to silt to be discerned. The use of the VisCPT along with new methods of image analysis may provide a means to remove some of the uncertainty associated with the lack of a sample with the CPT.

Another device that may aid in the classification ability of the CPT is the Soil Moisture Probe (SMP) (Pfitzner et al. 1995). This cone is equipped with a dielectric sensor to measure the electrical capacitance and conductivity of soils in-situ. Data collected using a miniature three-frequency sensor located in the cone has shown promise in the distinction of major soil types.

The ability of the CPT to determine the presence of thin soil layers, because of the continuous penetration record that is obtained, makes the test particularly useful for environmental applications. Location of sand layers at contaminated sites is especially important, since high permeability layers can be preferential pathways for pollutant transport.

The location of the water table can be estimated based on the pore pressures measured with the piezo-element located at the tip of the cone. Location of bedrock can be inferred by cone refusal. Use of the CPT for site characterization has certain advantages over conventional drilling and sampling techniques. The detection of the presence of thin layers using drilling and sampling techniques is only possible if continuous sampling procedures are used. Continuous sampling is often much more time consuming and expensive than using the CPT. In general, the CPT test is much faster than drilling and sampling; therefore, more soundings can be conducted with the CPT. If the site is contaminated or is considered to be potentially contaminated, the CPT has additional advantages. A CPT does not generate cuttings like those that are generated in conventional rotary boring methods. Contaminated cuttings generated by rotary borings must be disposed of in a proper manner, which can greatly increase the cost of site characterization (Bratton et al. 1995). The absence of cuttings in CPT tests also means a safer working environment for drilling crews, since their exposure to hazardous materials is reduced (Malone et al. 1992).

If site conditions mandate that the drilling equipment be decontaminated, the smaller diameter of the cone rods, compared to the larger diameter of conventional augers, results in less disposable waste water (Bratton & Timian 1995). Some commercial cones have sacrificial tips that can be left in the hole after penetration, and grout can be injected to fill the hole as the cone is withdrawn. The small hole made by CPT tests requires less grout to fill than the larger holes left by conventional rotary borings.

It is important to note that if penetration tests are being conducted at contaminated sites, usual methods for soil classification based on penetration test results may not be applicable. Soils contaminated with hydrocarbons, for example, may produce different tip and sleeve resistances than uncontaminated soils having the same classification (Lambson & Jacobs 1995, Malone et al. 1992).

A disadvantage of the CPT is the inability to penetrate certain types of materials that may have important significance in geoenvironmental projects. Gravel layers can serve as pathways for pollutant transport, and these layers are difficult to penetrate using normal cone penetrometers. Also, fractured bedrock zones may have high permeabilities, and the CPT cannot provide useful information for this type of material.

4.2 Determination of Hydraulic Conductivity

Efforts to determine the in-situ hydraulic conductivity of soil deposits from cone data can be separated into two groups: correlations to soil classification methods and pore pressure response methods.
4.2.1 Correlations to soil classification

Many empirical relationships exist which correlate the hydraulic conductivity of a soil to the soil type or grain size; e.g., Hazen (1911); Munassero (1994); Olsen (1994). If a CPT can determine the type of soil, then the hydraulic conductivity of the soil can be approximated.

Munassero (1994) compared the results of field and laboratory hydraulic conductivity tests with CPT tests conducted in cement-bentonite slurry trenches. Based on the soil classification chart developed by Robertson et al. (1986), he assigned hydraulic conductivity values to the different soil classification numbers in that chart. His tentative hydraulic conductivity assignments are shown in Figure 17.

Based on judgement and experience, Olsen (1994) overlaid hydraulic conductivity contours on a soil classification chart that he developed (Olsen 1988), and this is shown in Figure 18. He used the resulting plot to obtain the estimated distribution of hydraulic conductivity at a layered site. By calculating the resultant vertical and horizontal hydraulic conductivity values for the site, he obtained values that bracketed hydraulic conductivity data from field tests.

Chuang et al. (1992) and Smythe et al. (1989) present a procedure to estimate the hydraulic conductivity based on a direct correlation of the cone tip resistance with hydraulic conductivity. For a friction ratio of 0.5, their method simplifies to the empirical equation:

\[ k = 10^{-4} c^{-0.88} \]

(14)

with \( k \) in cm/sec and \( c \) in t/sf. A different empirical relationship was given for soils with a friction ratio of 1.0. It should be noted that this method is based on very limited data and given results that do not agree well with published hydraulic conductivity data.

4.2.2 Pore Pressure Response Methods

The rate that pore pressures are generated and dissipated during cone penetration is governed to a large degree by the hydraulic conductivity of the soil. If a penetration test is paused in a saturated soil layer, the time required for the pore pressures to return to hydrostatic values can be measured. This allows a pore pressure dissipation curve to be developed.

Both empirical and theoretical approaches have been used to relate hydraulic conductivity to the pore pressure dissipation rate for fine-grained soils (Baligh & Levadoux 1980; Torstensson 1982; Gupta & Davidson 1986; Teh & Housh 1991; Robertson et al. 1992; Runt et al. 1993). The pore pressure dissipation curve can be used to calculate the coefficient of consolidation for the soil (c_v), and the hydraulic conductivity can be determined based on a measured or estimated value of the bulk modulus (M) of the soil.

Robertson et al. (1992) state that for dissipation times (t) less than 0.3 min, the penetration process appears to be partially drained, and no correlation or data exist from which to determine the value of permeability. This limits the use of dissipation tests to silt and clay soils. Although knowledge of the permeability for fine-grained soils is still important, the majority of contaminant transport takes place in layers of higher permeability (i.e. silty sand and gravel). Measurement of the permeability of sand and gravel layers is of the greatest importance in modeling transport of contaminants, and dissipation tests are not applicable in these materials (Axel & Wright 1995).

Other penetrometers, such as the BDT probe, involve inserting a modified probe that contains an evacuated glass vial into the ground (Hogenstogler 1994; Zeno et al. 1994; Chuang et al. 1992; Stienstra & van Deen 1994). At a certain depth, a hypodermic needle is inserted through the bottom of the vial to allow pore water to fill the vial through a porous element located on the shaft of the probe. By measuring the time required to fill the tube and the final pressure in the tube, the hydraulic conductivity can be calculated. The vial can be removed and replaced, the probe can be inserted to a new depth, and another test can be conducted.

Another type of “cone” device can be used to in-

![SOL CLASSIFICATION by ROBERTSON et al. 1986](Figure 17. Hydraulic conductivity measurement based on the Robertson et al. (1986) classification chart (Munassero 1994).)
stall a PVC mini-well (Aust & Wright 1995). This device involves installing a PVC well pipe using a cone penetration insertion system. After the well is installed, conventional pump tests or falling head tests can be conducted.

These types of probes are not true cone penetrometers in the sense that they cannot measure tip and sleeve resistance, nor can they measure the dynamic pore pressures produced during penetration.

The information collected from conventional cone penetration tests that is of use for site characterization, such as soil classification, cannot be obtained from these probes. The results cannot be used with current methods based on correlations of hydraulic conductivity with soil classifications. In addition, these types of tests require considerable time, and may not be cost-effective for hydraulic conductivity assessment at large sites.

Figure 18. Hydraulic conductivity assessment based on Olsen's (1994) classification chart.
There may be uncertainties in measuring the permeability in a zone of disturbed soil. During the advancement of the penetrometer, the grain size distribution may change due to particle crushing, and the void ratio and structure of the soil near the penetrometer may be altered, resulting in lower than expected permeability values (Kosmad & Frechette 1995).

Elsworth (1993) developed a theoretical method to determine permeability based on dynamic pore pressure readings measured during cone penetration. He performed an analysis of cone penetration using dislocation theory, through which the hydraulic conductivity and coefficient of consolidation can be calculated. Elsworth's theory allows variables such as the penetration rate and the location of the pore pressure element to be examined. This theory differs from the other pore pressure theories in that the hydraulic conductivity can be calculated from the dynamic pore pressures measured during penetration. In other theories, the penetration must stop to allow full or partial pore pressure dissipation.

Elsworth's theory allows promise in that it addresses many of the important aspects of cone penetration testing that can influence hydraulic conductivity measurement. The main limitations of his theory are that it assumes stress-strain linearity of the soil and only small strains are assumed in the soil during penetration. However, this approach for the determination of hydraulic conductivity has great merit in that the hydraulic conductivity can be determined "on-the-fly" instead of stopping penetration and waiting for the pore pressures to dissipate.

In summary, cone penetrometers can be used to estimate the permeability of soil deposits in the same fashion as they can be used for estimation of soil type, but there are problems associated with directly "measuring" the permeability of sandy deposits.

4.3 Assessment of Contaminant Types and Distributions

The assessment of contaminant types and distributions is an important, but expensive, aspect of geoenvironmental projects. Special cone penetrometers equipped with new sensors have augmented the conventional methods of taking soil, gas, and water samples for field and laboratory analyses. Some cones allow gas, soil, and water samples to be taken at depth. These new probes have found use in performing initial site screenings, but have not supplanted the conventional methods of site assessment.

There has been a rapid increase in the development of sensors incorporated into cones that test gas, soil, and water samples down-hole. Table 5 lists some of the sensors that have been used to date. The following sections briefly discuss the more common sensor types and their use in site assessment.

<table>
<thead>
<tr>
<th>Sensor</th>
<th>Measured Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>Alkalinity of saturated soil environment</td>
</tr>
<tr>
<td>Resistivity</td>
<td>Bulk electrical resistivity</td>
</tr>
<tr>
<td>Temperature</td>
<td>Soil temperature</td>
</tr>
<tr>
<td>Gamma radiation</td>
<td>Gamma radiation level in soil</td>
</tr>
<tr>
<td>Fluorescence</td>
<td>Hydrocarbon presence in soil</td>
</tr>
<tr>
<td>ORP (oxygen-reduction potential)</td>
<td>Redox potential</td>
</tr>
</tbody>
</table>

4.3.1 Resistivity

One of the earliest and most common types of cone sensors used to indicate the presence of contaminants was the resistivity cone (Campanella & Weevers, 1990; Aust & Wright 1995; Malone et al. 1992; Jacobs & Hasan 1995). The electrical resistivity of soil deposits depends on the resistivity of the sand grains, dry density, pore water resistivity, and clay content (Malone et al. 1992). The presence of contamination can alter the pore water resistivity, and, therefore, the bulk resistivity measured in situ. Resistivity probes can determine ionic (salt) contamination in low levels, but may have difficulty in accurately discerning low concentrations of hydrocarbon and chlorinated solvents contamination (Malone et al. 1992). These probes have been successfully used to determine the presence of acid tails (Jacobs & Hasan 1995) and creosote (Okoye et al. 1995). The bulk resistivity of a soil depends on many factors. For example, it may be difficult to determine the presence of pollutants that are lighter than water, since these float on top of the water table. Since the bulk resistivity can vary according to soil moisture content, changes in the normal soil resistivity can obscure detection of the contaminants (Okoye et al. 1995). The resistivity value alone cannot provide a clear indication of contaminant concentration. Other cone parameters (tip, sleeve, and pore pressure) should be considered when assessing if a site is contaminated (Malone et al. 1992).

4.3.2 Temperature

Temperature transducers have long been incorporated into cone penetrometers. The original reason for employing these sensors was to monitor the soil temperature and provide a means of calculating the soil temperature at various depths. Temperature increases due to the friction developed during penetrometer advancement.
4.3.3 pH and oxygen-reduction potential (ORP)

Recently sensors to measure pH and redox potential have been used downhole in cone penetration systems. In the earliest applications, the cone is penetrated to the desired depth of testing. A water sample is collected from the cone and then reinserted for the next measurement. Alternatively, a fluid flushes water or another decontaminating fluid while remaining in the ground. In newer systems, the pH and redox sensors are designed to work at the interface between the cone and the soil, thus allowing continuous measurements during penetration (Ptumma et al. 1995).

4.3.4 Gamma radiation

Gamma radiation probes are an example of matching specific sensors for the contaminant present at the site. Branton & Timian (1995) report using a probe containing a gamma radiation sensor to examine the extent of contamination at a nuclear fuel production site.

4.3.5 Fluorescence

Much effort has been expended in developing advanced cones that measure the fluorescence emitted by a soil when illuminated by laser light (Lieberman et al. 1991; Shin & Branton 1995; Lambson & Jacobs 1995). These cones are often called laser-induced fluorescence (LIF) cones. The primary purpose of these cones is to determine the presence and concentration of petroleum-based hydrocarbons. In the earliest versions, the light from a pulsed nitrogen laser located at the ground surface was transmitted through a fiber optics cable to a sapphire window located at the soil-penetrator interface. The laser light causes the illuminated material to emit photons (fluorescence). The fluorescence is transmitted back up to the ground surface via a separate fiber optics cable, and recorded using a photodiode array detector (Lieberman et al. 1991). The raw parameters of interest are the fluorescence intensity, wavelength, and the lifetime or decay of the photons (Lambson & Jacobs 1995). These data can provide a “signature,” which depends on the contaminant type and concentration.

Newer developments have included using tunable or variable wavelength lasers to maximize the fluorescence, and to incorporate the source of the laser light and the detection device downhole (Shin & Branton 1995). These are intended to make the LIF cone more robust and cheaper.

These new devices show great promise in measuring the presence of certain types of contaminants; however, more research is necessary before these devices can be used for broader applications. A large database of tests conducted in contaminated soils is needed to allow better test interpretation. The results obtained from the LIF cone are not just dependent on contaminant type and concentration, but are also influenced by soil type, grain size, existing organic compounds in the soil, and other factors (Branton et al. 1993). The presence of contaminants in the soil can influence the measured fluorescence due to scattering effects at the sapphire window (Lambson & Jacobs 1995). Certain contaminants also influence the tip and sleeve resistances. Lambson & Jacobs (1993) suggest using the sleeve and tip resistances to augment the fluorescence data to develop a more complete “signature” for contaminated soil. Another possible drawback of the LIF probe, as well as other CPT systems described, is that pollutants may be carried down the hole with the probe, and erroneous readings may result at the lower elevations (Branton et al. 1995).

4.3.6 Sampling Probes

Some probes allow water or gas samples to be taken and analyzed within the cone by internal sensors, or a conveyance system can transport the sample up to the ground surface for testing. An example of this is the BAT probe described in section 4.2 of this paper. These probes normally are not considered to be cone penetrometers in the sense that they do not provide tip, sleeve, and dynamic pore pressure measurements. However, a new cone penetrometer developed at ISMES does provide conventional cone penetration data, as well as gas, water, and soil samples. This is the Envirosensor®, or the Multifunctional Envirocone Test System (MEETS) (Baldé & O'Neill 1999; Piccoli & Benoit 1995). The cone contains an internal pump to transmit gas and water samples up through the drill rod where pH, redox, dissolved oxygen, and temperature are measured using a flow cell at the ground surface. Special provisions are made to purge the contaminants from the sample conveyance system and to quantify the success of the purging procedure by measuring the electrical conductivity of the fluid. In addition, a tubercule sampler can be pushed ahead of the cone tip for retrieval of a soil sample.
4.3.7 Other CPT systems for Contaminant Assessment

There are many other new cone penetration systems that are being developed but space constraints prevent the authors from discussing these in detail in this paper. These include probes that use frequency domain and time domain reflectometry to measure the dielectric properties of soils, which may give some insight into pollutant presence (Silicona & van Deen 1994; Plümgräaff et al. 1995). CPT systems making use of fiber optics with other types of non-contact systems that use scattered light Raman spectroscopy, infrared reflectance, and absorbance are under development (Baldl & O’Neill 1995; Braton & Tinnin 1985; Branton et al. 1995). It is reasonable to assume that the development of even more advanced CPT systems will continue to parallel the rapid development of miniature sensors and advanced data analysis systems.

4.4 Design of site remediation methods

The CPT can be used as an aid in designing site remediation methods and evaluating the success of these methods. Certain methods of bioremediation of wastes make use of biological processes of microorganisms to reduce the level of concentration of contaminants in the soil. The success of these methods can depend on if the in-situ environment can sustain the growth of these organisms. Two different probes have been developed that measure in-situ parameters which can aid in this assessment.

The Chemoprobe is a penetrometer that combines a combination of sensors to measure the in-situ temperature, pH, electrical conductivity, and redox potential of water samples (Silicona & van Deen 1994). The probe allows a 15 ml specimen of water to enter the probe, and the water comes in direct contact with the sensors. The system must then be flushed for subsequent testing.

A newer probe, termed the PhOReP probe, is in development by Plümgräaff et al. (1995). Similar to the Chemoprobe, this CPT device measures the soil pH, temperature, and redox potential. This probe, however, uses a sensor-mounted at the soil-probe interface and allows the measurements to be made while the core is being pushed. These new devices can aid in determining if the in-situ environment is compatible with certain bioremediation techniques.

The CPT devices previously described for determining the presence of contaminants can also be used for assessing the performance of a chosen remediation procedure. Sounding conducted before and after the remediation efforts can give an indication if the remediation effort is successful. As an example, shown in Figure 19 is a plot of the sounding made with Vertek’s patented Fuel Fluorescence Detector (FFD) cone system (an LIF system). Figure 19 shows the Total Petroleum Hydrocarbon (in ppm) initially at a site and during air sparging, which was employed as a remediation procedure. A reduction in the amount of petroleum product is evident at depths below about 10 ft.

4.5 Summary of CPT use in geoenvironmental applications

The CPT has made rapid technical advances in the past few years to become a valuable tool in geoenviron-

![Graph showing TPH (ppm) x 100 before and during air sparging](image-url)
environmental applications. These applications include site assessment, determination of the in-situ permeability, determination of contamination type and extent, and assessment of remediation methods and the success of these methods. In addition to the many advantages and disadvantages of the CPT for these applications compared to the conventional methods of drilling, sampling, and installation of wells that have been discussed, a few general observations should be noted.

1. The cone has become a much more complex device than the device that has been used successfully in geotechnical applications for over twenty years. This results in a higher cost, which also increases the financial consequences if the penetrometer is lost.

2. The associated equipment necessary for monitoring the sensors has become much more complex, and the education and training of the field technicians and engineers will become much more important.

3. Field maintenance of the equipment will become much more difficult. The interpretation and presentation of the data will become a more specialized field.

4. The calibration of conventional cone penetrometers has always been important. The calibration of the more advanced and complex sensors in the newer cones will be even more important to ensure validity of the data.

5. The older, more conventional, correlations used with CPT tests, such as soil classification, relative density, permeability, and strength, need to be critically assessed if they are employed at contaminated sites.

More technological advances are needed and additional field experience must be gained before the CPT assumes a primary role in site assessment for geoenvironmental purposes. Currently, there has not been the regulatory acceptance required for using it in lieu of more conventional and proven methods (Bratton et al. 1995; Malone et al. 1992). Conventional methods of drilling, sampling, laboratory testing, and installation of monitoring wells will continue to play an important part in geoenvironmental engineering practice.

Nonetheless, the advanced CPT systems described can be used to great advantage for the following purposes:

1. To help in the overall site characterization, including identification of soil strata and their boundaries, and the location of the water table.

2. To help delineate the extent and depth of contaminants, which may be used to more accurately calculate the volume of material that has to be treated.

3. To determine some properties of the subsurface materials.

4. To aid in the design of a monitoring well system. The initial screening tests that can be performed with these devices can optimize the placement of monitoring wells, which may reduce the costs of projects and increase the usefulness of monitoring wells.

5.0 ASSESSMENT AND CONCLUSION

Theories for the quantification of cone penetration resistance in terms of soil properties have been evaluated. Cavity expansion theory stands out as the most consistent and accurate of the methods that have been developed. Several important applications for the CPT in geotechnical earthquake engineering and geoenvironmental engineering have been described. Important issues in CPT testing and data interpretation include sensitivity, reliability, and interpretation methods for identification of soil types, soil state, in-situ stresses, and groundwater conditions.

The two most important limitations of the CPT for site characterization and in-situ property determination are (1) the static cone cannot be used in all soil types, and (2) no samples are obtained for positive soil identification. Although soil classification charts have been developed in an attempt to overcome the second of these limitations, care must be exercised in their use, and soil samples should always be taken to validate the chart interpretations, unless prior experience is available for the site and soil conditions under investigation.

Several specific conclusions may be drawn from the topics reviewed in this paper, as follows:

1. Theoretical predictions of the cone penetration resistance of sands in terms of the sand properties are possible within an accuracy of about ± 30 percent. A higher degree of accuracy should be possible when deducing soil properties from cone resistance.

2. The CPT can be used reliably and economically for several phases of geotechnical earthquake engineering, including soil profiling, identification of critical strata, determination of small strain shear modulus from measurements using a seismic cone penetrometer, liquefaction potential assessment, and the design and evaluation of ground improvement for mitigation of ground failure risk. No techniques are yet available for direct determination of post-liquefaction residual strength by the CPT.

3. The presence of layers of different density, stiffness, and strength within a soil profile may have a significant influence on the measured values of cone penetration resistance. The cone "errors" the presence of a different underlying layer when
it reaches within a few diameters of the layer boundary. The layer effect is likely to be conservative in that the measured resistance of stiff or dense layers will be low, whereas that measured in soft or loose layers will be reasonably close to the correct value. On the other hand, the layer effect could lead to incorrect soil identification if tip and friction ratio based classification charts are used.

4. Determinations of fines content from measurements of friction ratio are not reliable.

5. The ratio q_{tip}/N of CPT tip resistance to SPT N-value does not correlate uniquely with mean grain size (D_{50}). Site specific determinations should be developed if a q_{tip}/N value is needed so that the results of CPT tests can be used with N-value property correlations, or vice versa.

6. The soil behavior index I_{s}, computed using the normalized tip resistance and normalized sleeve friction and equation (9), appears to provide a means for taking the effects of fines, plasticity, and other soil characteristics into account in the assessment of liquefaction potential.

7. CPT-based liquefaction potential assessment charts have now been developed that are supported by both field data and theoretical considerations.

8. The CPT provides a good method for assessing the magnitude and uniformity of in-situ soil densification by various methods.

9. Specific uses of the CPT in geoenvironmental projects include site characterization, determination of groundwater flow conditions, assessment of contaminant types and distributions, measurement of hydraulic conductivity, and in the design of waste containment and site remediation methods.

10. The recently developed “vision cone penetrometer” may help significantly in overcoming the major disadvantage of the CPT; namely, no sample for positive soil identification.

11. Some progress has been made in the development of methods for reliable assessment of hydraulic conductivity from CPT data. Although good success is possible using dissipation tests and in-flow or out-flow tests while the penetrometer is held stationary at a given depth, techniques for accurate determination of hydraulic conductivity during penetration need further development.

12. Sensors can be incorporated into the cone penetrometer system to measure pH, temperature, electrical resistivity, fluorescence, gamma radiation, and oxygen-reduction potential, all of which can be useful for detection of pollutants, their concentrations, and their distributions.

13. New probes have been developed for use in conjunction with the CPT that allow water and gas samples to be taken and analyzed by internal sensors.

14. As with most new techniques, regulatory acceptance is slow in coming, but must be obtained before widespread use of the new sensors can be expected.

6.0 ACKNOWLEDGMENTS

Dr. Ignacio Arango provided data and information useful for the assessment of CPT-SPT correlations and fines content determination by CPT measurements. Dr. Richard S. Olsen contributed the results of his recent analyses and correlation studies. Professor Rodrigo Salgado assisted through identification of relevant publications and analyses of the accuracy of CPT measurements. Dr. Ross W. Boulander provided valuable insights on interpretation of CPT data. Ms. Keesi Perkins assisted in preparing the figures and in the formatting of the text. We thank these colleagues for their valuable input to this paper.

7.0 REFERENCES


Hazen, A. 1911. Discussion of "Dams on Sand Foundations" by A.C. Keong. Transactions. 73: 199. ASCE.


Olsen, R.S. & J.K. Mitchell 1995. CPT stress nor-
Of Geotechnical Engineering. 121(12): 856-869.
Silenstra, P. & J.K. von Dienen 1994. Field data col-
lection techniques – unconventional sounding and
sampling techniques. In N. Rengers (ed), Engi-
neering Geology of Quaternary Sediments. Proce.
20th Anniversary Symposium of the Ingangi:
41-56. A.A. Balkema: Rotterdam.
Prediction of liquefaction resistance based on
CPT tip resistance and device friction. Proc. XIV
Int. Conf. Soil Mech. And Found. Engn. Han-
burg, Germany.
Suzuki, Y., K. Nishizaka, K. Koyamada, Y. Taya, &
Y. Kubota 1995. Field correlation of soil lique-
faction based on CPT data Proceedings. CPT'95
Suzuki, Y., K. Tokimatsu, Y. Taya, & Y. Kubota
1995. Correlation between CPT data and dynamic
properties of in situ frozen samples. Proc. 3rd Int.
Conf. On Recent Advances in Geotech, Earth-
quake Engineering and Soil Dynamics. 1: 249-
252. St. Louis, Univ. of Missouri, Rolla.
of the cone penetration test in clay. Geotechnique.
41: 17-34.
Teh, C.J. 1987. An analytical study of the cone
U.K.
Wiley, New York.
Tokimatsu, K. 1988. Penetration tests for dynamic
problems. Proc. 1st Int. Symp. on Penetration
Torstensson, B.A. 1982. A combined pore pressure
and point resistance probe. Proceedings of the
Second European Symposium on Penetration
Treadwell, D.D. 1976. The influence of gravity,
pressur, compressibility, and layering on soil re-
stance to static penetration. Ph.D. thesis. Uni-
versity of California, Berkeley.
Tseng, D.-J. 1989. Prediction of cone penetrometry re-
istance and its application to liquefaction as-
essment. Ph.D. thesis. University of California,
Berkeley.
Van den Berg 1994. Analysis of soil penetra-
Veise, A.S. 1961. Bearing capacity of deep founda-
tions in sand. Highway Research Record 39: 112-
153.
98(SM3): 265-290. ASCE.
Veise, A.S. 1977. Design of pile foundations, Na-
tional Cooperative Highway Research Program
Report No 47 Transportation Research Board,
Washington, DC.
Vreugdenhil, R., R. Davis, & J. Berrill 1994. Inter-
pretation of cone penetration results in multi-
layered soils. Int. Journ. For Num. and Analytical
Methods In Geomechanics. 18(9): 585-589.
penetration problems in clay. Proc. 1st Int. Conf.
On Computational Plasticity: Fundamentals and
Applications: 2: 883-894.
capacity in crushable sand. Geotechnique. 45(4):
663-676.
Yu, H.S. & J.K. Mitchell 1996. Analysis of cone re-
stance: a review of the literature. Internal Report
No. 142.09.1996. Department of Civil Engineer-
ing, University of Newcastle, Australia.
Yu, H.S. & J.K. Mitchell 1998. Analysis of cone re-
stance: a brief review of methods. To appear in
the Journal of Geotechnical and Geoenviron-
mental Engineering. ASCE.
Yu, H.S., F. Schmied, & L. Collins 1996b. Analysis of
cone pressuremeter tests in sands. Journal of
Geotechnical Engineering. 122(8): 625-632.
ASCE.
Advanced numerical methods for the analysis of
cone penetration in soils. Internal Report, De-
partment of Civil Engineering, University of
Newcastle, Australia.
Cone pressuremeter testing and discrete-depth
ground water sampling techniques: a cost-
effective method of site characterization in a
multiple-aquifer setting. Ground Water Moni-
toring Review. 13(1): 176-182.
Zervaigianis, C.S. & N.A. Kaltezios 1988. Experi-
nences and Relationships from Penetration Test-
ing in Greece. Proc. 1st Int. Symp. on Penetration

8.0 NOTATION

The following symbols are used in Tables 2, 3, and
4:

\[ A = \frac{q_n}{p_{0}} \]  - ratio of effective spherical cavity limit
pressure to the initial mean effective stress

\[ C_D, C_s \]  - constants in chamber correlations be-
tween cone resistance and relative density

\[ D_v = \text{relative density of soil} \]
\[ G = \text{shear modulus of soil} \]
\[ I_r = \frac{G}{\sigma_0 \text{, rigidindex of clay} \]
\[ I_n = \text{reduced rigidity index of sand} \]
\[ K = \text{variable used in the slip-line solution of} \]
Sokolovskii (1965) \]
\[ N_s = \text{coefficient of earth pressure at rest} \]
$k, m =$ constants in chamber correlations between cone factor and state parameter

$m_1, m_2, m_3, m_c =$ constants in Collins et al.’s (1992) cavity expansion solution for spherical limit pressure

$N_i = (q_i - p_i) / \alpha_i =$ cone factor for clay based on cavity expansion solutions

$N_i = (q_i - \alpha_i) / \nu_i =$ cone factor for clay based on all other methods

$N_h = q_i / \nu_i =$ cone factor for sand based on all other methods

$\nu =$ function of friction angle used in the solution by Vesic (1977)

$p_i =$ in-situ total mean stress

$p_i' =$ in-situ effective mean stress

$q_i =$ effective cone tip resistance

$\alpha_i =$ unconfined shear strength of clay

$\sigma_{n0} =$ initial soil specific volume

$\sigma_{n0} =$ in-situ total vertical stress

$\sigma_{n0}' =$ in-situ effective horizontal stress

$\sigma_{n0}' =$ in-situ effective vertical stress

$\alpha =$ cone apex angle

$\beta =$ friction angle at the soil-cone interface

$\beta =$ angle used in the solution Janksu and Sennacet (1974)

$\lambda =$ cone roughness indicator (1 for a rough cone and 0 for a smooth cone)

$\phi'$ = drained angle of soil friction

$\phi_0, \phi_0, \phi_0, \phi_0, \phi_0 =$ angles used in the slip line analysis by Solodovnikov (1965)

$\psi =$ dilation angle of soil

$\psi_i' =$ effective cylindrical cavity limit pressure

$\psi_i' =$ effective pressuremeter limit pressure

$\phi_i' =$ total spherical cavity limit pressure

$\phi_i' =$ effective spherical cavity limit pressure

$\zeta =$ state parameter (Bec et al. 1987)
Advanced interpretation of field tests

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ABSTRACT: A selective review is made of some of the advanced techniques that are available for the interpretation of in situ tests. Soil classification from CPT tests has in the past been based principally on use of charts, and the use of Neural Network classification systems offers a powerful and general alternative. An important feature of the interpretation of cone or pressuremeter tests is that the strength, stiffness and horizontal stress contribute to give a particular test result. Advanced interpretation methods must take into account this interaction, so that factors used to derive one parameter may depend on the value of another. In the analysis of pressuremeter tests two developments are highlighted: the analysis of the unloading phase of the tests, and the realisation that the failure length of the pressuremeter has a substantial effect on strength measurements. Interpretation of the cone pressuremeter requires use of some advanced techniques (and in particular requires use of large strain analysis), but this is repaid by the benefit that it combines many of the advantages of both the CPT and the pressuremeter.

1 INTRODUCTION

The purpose of this paper is to conduct a rather general review of the circumstances in which advanced interpretation of field tests is justified and of some of the techniques that can then be used. Many of those involved in research on field testing are already using or developing advanced interpretation methods. Practising engineers are often, however, more sceptical about the merits of some of the more advanced methods. This paper is therefore primarily aimed at practising engineers, with the intent of encouraging them to adopt new methods when they are appropriate.

What constitutes “advanced” interpretation of field tests? This question has no simple answer, and can only be interpreted within the context of the currently accepted state-of-the-art. What is at present regarded as an advanced technique may in the future be regarded as commonplace. For the purpose of this paper “advanced” methods will be taken as any that fall into one or more of the following categories:

- methods which involve corrections to current approaches, based on improved understanding of the mechanics of the test,
- methods which are intrinsically complex (by present standards), e.g. those which rely heavily on computational techniques,
- new methods which have not yet gained current acceptance in practice, yet potentially offer improvements over conventional approaches.

1.1 Field testing or laboratory testing?

A discussion of the advanced interpretation of field tests must first address the long-standing debate on the relative merits and applications of laboratory and field testing. The interpretation of tests must also be set in the context of the design methods that will be used in connection with the test results.

The way in which in situ test results have been used has depended very much on the history of the development of geotechnical engineering in different countries. For instance, in the United Kingdom since about the mid-1950’s, geotechnical engineers have relied heavily on the triaxial test as the primary means of measuring the strength and deformation properties of soils. In, for example, slope stability analysis, the method described by Bishop (1955) would usually be used. The choice of a safety factor would be based on the combination of use of triaxial testing and Bishop’s method. If either of these were to be changed, then new experience on appropriate safety factors would need to be obtained.
The above position meant that, as in situ testing has become established in the U.K., much effort has been put into use of field tests to measure the same engineering parameters as are measured by the triaxial test. During this process it has been recognised that, for instance, the mode of shearing affects the undrained shear strength, so that experience has been obtained in converting measurements made with field tests back to equivalent values from triaxial tests.

The development of field testing in the U.K. can be contrasted with, say, the history of the pressuremeter test in France. At the time that the Ménard pressuremeter was developed, the use of laboratory testing as the primary tool for measuring soil properties was less well established in France. As a result, design methods were developed in which the results of the pressuremeter tests were used directly in, for instance, foundation design. Although correlations were later established between the parameters measured by the Ménard pressuremeter and more fundamental engineering parameters, it is not necessary to know the latter to use the pressuremeter “design rules”.

The “direct method” of use of in situ test results (exemplified by the French application of the pressuremeter) and the “indirect method” (exemplified by the approach commoner in the U.K.) are illustrated in Figure 1.

The direct approach has advantages, principally in that it is straightforward to apply, since it tends to rely on a series of rather well-defined procedures. Provided that these have been well thought out, the method can be quick and economical. It has, however, a number of disadvantages in that:

- it relies heavily on the collection of large numbers of case records for calibration,
- it does not provide the engineer with a sense of the importance of particular features of the soil behaviour on the design,
- it is not readily generalised to new soils, new types of construction or new tests.

As a result of these constraints, the direct approach does not lend itself to rapid change, and so there are few developments of “advanced” interpretation in this area. The examples given below will therefore be within the context of the indirect approach.

When in situ tests are employed using the direct approach, then the relative merits of in situ and laboratory tests for a particular engineering application may need to be discussed. However, when using the indirect method, such a debate is perhaps less important, since a combination of laboratory and in situ tests can be used. In the second phase of the design (B in Figure 1) the engineer will need to select the most important parameters needed. The decision then becomes one of a choice of the most appropriate method to measure these parameters. The issues in choosing a method will usually include:

- the relative simplicity and accuracy of laboratory and field methods,
- problems of sample disturbance,
- how well the field test may reproduce the design conditions,
- possible benefits of measuring a quantity by more than one means,
- cost.

1.2 Chemical and environmental testing

Many sophisticated new testing methods are being introduced in the area of environmental testing. A prime example is the use of laser-fluorescence techniques to identify hydrocarbons contamination at polluted sites (Lamborn and Jacobs, 1995). The development of such tests has involved a major investment both in equipment, and in the interpretation techniques for the tests. The author has no direct experience of use of in situ tests for environmental purposes, so these applications are not covered here. They are, however, addressed elsewhere at the ISCE conference. The author acknowledges that many of the “advanced” techniques currently under development are in the geo-environmental area.

1.3 Geophysical testing

Another area where rapid advances are being made is that of geophysical testing. In particular the development of tomography and other imaging
1.4 Examples

The examples given below all involve the use of advanced techniques to determine the physical characteristics of soils. They are drawn principally from the use of the cone penetrometer (CPT) and the pressuremeter, since these are probably the commonest in situ devices for measuring the mechanical properties of soil. The two tests are complementary, in that the principal application of the CPT is as a profiling tool, with a supplementary use for estimating soil property values, while the pressuremeter is less appropriate for profiling, but is used primarily for property measurement.

The examples are drawn principally from the Author’s own experience, but there are many others who are also working on the development of advanced interpretation methods.

2 PREREQUISITES

There are certain prerequisites that have to be satisfied for any interpretation of in situ tests, and these are of course even more important if the interpretation is to be of a sophisticated nature. In the following it will be assumed that the following minimal criteria will be satisfied as a matter of good practice:

- All tests will be carried out using equipment that is in good working order, properly maintained and calibrated, and suitable for testing the particular soil encountered.
- Test procedures will adhere to accepted standards (where these are published), including proper record-keeping of all relevant data.
- All necessary corrections will be applied to raw data so that the results properly represent the soil response (examples of corrections are those for the membrane stiffness in a pressuremeter test, or of the correction from \( q_s \) to \( q_l \) in the CPT).
- Data should be presented in an appropriate way, where possible making use of properly defined dimensionless groups. For example, use of \( (q_s - \sigma_{wo})/\sigma_{wo} \) is acceptable, but use of \( q_l/\sigma_{wo} \) is not, since, in an undrained CPT test \( \sigma_{wo} \) simply results in an additive term on \( q_l \).

3 SOIL CLASSIFICATION

Soil classification is here taken as the qualitative description of the soil (e.g. sand/silt/clay), together with qualifying comments (loose/dense, normally consolidated/overconsolidated, soft/stiff etc.), but not involving quantitative measurement of parameters.

The two primary in situ devices for soil classification are the CPT (and especially the piezocone) and the Marchetti dilatometer. One advantage of the CPT is that it gives a continuous profile, and the dilatometer too gives quite a detailed profile (with data usually at 100mm intervals). The interpretation as far as soil classification and stratigraphy is concerned is almost entirely empirical, and has principally been expressed in the form of charts. Those published by Robertson et al. (1986) for the interpretation of the piezocone are a typical example.

This approach is undoubtedly valuable, principally because it allows practitioners to gain a quick estimate of the sorts of soil present, without the need for any sophisticated calculation. It does, however, have drawbacks. CPT classification charts were originally presented in terms of two variables, usually cone resistance \( q_c \) and friction ratio \( f_r \) (see e.g. Douglas and Olsen, 1991). In this case the classification can be represented in a simple way on a two-dimensional chart. There is inevitably some overlap of the zones, but this can be reduced by normalising the parameters in a suitable way. Wroth (1984, 1988) suggested use of \( Q = \frac{q_c}{\sigma_{wo}^{0.5}} \) and

\[ F_r = \frac{q_c - \sigma_{wo}}{q_c} \times \frac{\sigma_{wo}}{\sigma_{wo}} \]

both of which can easily be provided determined that estimates of in situ vertical stress and pore water pressure can be made. Robertson (1990) adopted this normalised form. Houlsby and Hitchman (1988) showed that the cone resistance is more closely related to the horizontal stress than the vertical stress, so that the use of modified factors such as \( Q_h = \frac{q_c}{\sigma_{ho}} \times \frac{\sigma_{wo}}{\sigma_{wo}} \) would result in charts with less overlap of different regions. The problem is that, to use such a chart, an estimate of \( K_p \) must be made. Since this is often not possible with any accuracy, authors have preferred the normalisation with respect to vertical stress. This practice is, however, misleading, since the dependence on \( K_p \) is effectively hidden in the inaccuracies in the chart rather than being explicitly
apparent to the engineer. The charts therefore work well for soils with typical $K_p$ values, and not so well for other soils. It would be better to use normalisation with respect to horizontal stress, and provide engineers with guidance on the estimation of $K_p$.

The major problem with the use of charts for classification arises when they are extended from two variables to three or more. Robertson (1990) presents charts involving $Q_2$ against $F_2$ and $Q_3$ against $B_3$. The charts represent projections of classification zones in the three-dimensional space onto two-dimensional planes. The problem lies not with the choice of variables, but the limitations inherent in presenting three-dimensional information in two-dimensional form. If the zone is to be represented have anything other than the simplest of shapes, this approach rapidly becomes unsatisfactory.

A variety of additional sensors (further pore pressure measurements, acoustic sensors etc.) have been added to the CPT, and if these are to be exploited for classification, the problem becomes one of combining data from four, five or even more sensors. Such a task is daunting, and it is unlikely that simple two-dimensional charts will offer a solution. It is possible that ingenious correlations could be devised (either empirically or with some theoretical input), but this process becomes increasingly difficult the more variables are involved.

An alternative approach, in which the Author has been involved, is the use of techniques developed in the IT area to assist in classification. Specifically, neural networks have been used to classify soils. The procedure is as follows, and is illustrated in Figure 2. A number of input quantities are chosen for the Neural Network, in the case illustrated these are the normalised cone resistance, friction ratio and pore pressure measurement. The values of the "hidden layer" units are each a weighted sum of the values of the input units, and the values of the output units are in turn weighted sums of the values of the hidden layer units. Most importantly, nonlinearities are introduced in both these summation processes (usually in the form of a function that effectively saturates at a particular value). The values of the output units can represent either numerical values of quantities such as density, horizontal stress or undrained strength, or particular values can be assigned to mean for instance "clay" or "silt" or "sand". Neural Networks are therefore suitable both for determining numerical values of engineering parameters, or for classification of soils into categories. When used to determine numerical values the method fulfills a similar role to multiple regression analysis, but has the advantage that the form of the function fitted to the data does not have to be chosen in advance.

The Neural Network is first "trained" on a large set of data for which both the input and output values are known (in the same way that classification charts are based on databases of tests from known sites). During this phase the weighting factors are found by an optimisation process which minimises the errors in the prediction of the output variables.

Once the network has been "trained", i.e. the weighting factors are known, it is then tested by applying it to a second set of data for which both input and output are known. The accuracy of the predictions of the output quantities is measured, and provided that this is acceptable, the trained network is then of use for application to data where the output is unknown.

Henshaw and Ruck (1998) give an example of the use of such a system for the identification of both soil type (in this case distinguishing between three types of sand) and engineering properties of sands. Both conventional CPT data, and the data from an acoustic sensor were used. The method proved to be particularly effective for identifying soil type, but was less effective as far as quantitative estimation of soil properties is concerned.

The Neural Network approach clearly has certain disadvantages when compared to the use of charts:

- it requires use of computer software rather than a sheet of paper,
- the classification process is not as obvious to the engineer, since it is hidden within the weighting factors of the network, rather than being transparently obvious as lines drawn on a chart.
The first of the drawbacks is rapidly becoming less important. The second can be offset by the fact that, in a properly designed system, an indication of the confidence with which a network is able to make a classification is available as well as the classification itself. (This is equivalent to the answer to the question "How close is the point to the boundary between two classifications?"). What are the direct advantages of the new approach? The most important are:

- The method can be extended simply to any number of inputs, and so gets away entirely from the limitations of two-dimensional charts,
- The classification process can be carried out in a rigorous mathematical way, and is not biased by subjective judgement. (Some may of course regard this as a disadvantage, since it leaves little room for engineering judgement).

The use of modern IT techniques certainly has a role to play in the identification of soil types from in situ data, especially where several measured variables may affect the classification. Engineers’ quite justified suspicion that such methods represent “black boxes” over which they have little control should be allayed by clear presentation of the principles on which any method is based.

4 MEASUREMENT OF THE ENGINEERING PROPERTIES OF SOILS

A key feature of developments in the understanding of the interpretation of in situ tests has been the realisation that the results of the tests are affected by a multiplicity of factors. The strength, stiffness and in situ stresses interact to produce a particular measurement in a test. There are exceptions, such as the vane test, which provides a direct measurement of the undrained strength, and it is accepted that the vane strength is relatively unaffected by other factors such as the soil stiffness. The general rule is, however, that the results obtained represent the combined effect of several factors.

The following discussion is therefore organised in terms of different tests, rather than in terms of different measured quantities.

5 THE CONE PENETROMETER

5.1 The CPT in clay

The undrained strength of a clay is derived from the CPT results from a formula of the well-known form:

$$q_c = N_{c1} \sigma_u + \sigma_v$$  

(1)

A significant advance was the recognition that the total cone resistance $q_c$ (which is corrected for the pore pressure acting in the groove behind the cone tip) should be used, not simply the measured cone resistance $q_c$. This practice has now, fortunately, become almost universal, but it means that some early databases that use $q_c$ are no longer of value.

“Advanced” interpretation of the cone here relates entirely to the determination of the factor $N_{c1}$. The main variables that affect $N_{c1}$ are:

- the soil stiffness,
- the horizontal stresses in the ground.

This reveals immediately one of the key features of the in situ test: that the engineering parameters for the soil cannot be measured separately, but that strength, stiffness and horizontal stresses all combine to affect the results of the tests.

Despite the apparent simplicity of the cone test, it is not straightforward to analyse. Housby and Teh (1988) analysed the CPT test in clay using a combination of the strain path method and finite element methods, and arrived at the following empirical expression which fitted their calculated $N_{c1}$ values:

$$N_{c1} = N_{c1} \left[ 1 + \frac{1}{2000} \frac{G}{s_u} \right] + \frac{2.2 + 1.5 \frac{\sigma_{um} - \sigma_{u1}}{2s_u}}{5 \left( 1 + \ln \left( \frac{G}{s_u} \right) \right)}$$

(2)

where $N_{c1}$ is the spherical cavity expansion pressure, $s_u$ is the undrained strength, and the above expression is a simplification of Housby and Teh’s expression in which an intermediate value of cone roughness has been assumed.

The advantage of an expression such as the above is that it explicitly recognises the role of the horizontal stress and the stiffness in affecting the cone resistance. The engineer can assess the impact of different assumptions about (for instance) the horizontal stress on the calculated undrained strength.

Equation (2) was derived theoretically, and because of the shortcomings of the analysis (which, for instance, did not take into account pre-failure changes of stiffness, or the possibility of any sensitivity) it probably will not agree with field data at a given site. Locally established correlations would usually provide superior estimates of $N_{c1}$.  

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but equation (2) could nevertheless be used to estimate the sorts of variation of \( N_d \) that might be expected at locations with different horizontal stress and stiffness values.

Although the measurement of the strength with the CPT is affected by the stiffness and horizontal stress, the CPT provides no independent measurement that can provide values of these quantities: they must be estimated by other means.

This means that, as far as measuring soil properties is concerned, the CPT is best used in conjunction with other devices.

5.2 The CPT in sand

Interpretation of the CPT in sand is a more challenging task than interpretation in clay. Analysis of the cone penetration process is much more difficult in a frictional, dilating material than it is in undrained clay. Some analyses have been made, but even the most successful of these (see for example Durganoglu and Mitchell, 1975, and Last, 1982) must only be regarded as approximate.

As a result, the interpretation of cone data in sands is based more on empirical evidence. The difficulty of obtaining site data where the properties of sands can be well established by independent means has in turn led to the development of calibration chamber testing. Most of the “advanced” interpretation of the CPT in sand is based on results from calibration chamber tests. Note that this is in contrast with clays, for which calibration chamber tests are time-taking and expensive, and calibration of analytical procedures at well-documented test sites is a more advantageous option.

Large calibration chambers for sand are widely available. Lanne et al. (1997) provide a useful compendium of work completed at NGI, ENEIL CRIS, ISMES and Southampton. Much other work has been carried out at Oxford University and at Monash University.

The main disadvantage of calibration chamber tests is that even if the chamber is very large, there is still some measurable influence of the boundaries in dense sands (Schmidt and Holstey, 1991). Appropriate corrections can, however, be made, so that correlations established in chambers can be used with some confidence in the field. Another problem is that there is evidence that the properties of naturally aged sands differ significantly from those of sand prepared in the laboratory.

Nevertheless, data from well-planned series of calibration chamber tests can be used to establish correlations for field interpretation. The correlations are principally empirical, although some include some physical insight to the problem.

It is difficult to control the stiffness of a sand separately from the values of other variables. This means that, although the influence of the stiffness on the measured cone resistance is recognised, the precise effect cannot be quantified in calibration chamber tests. The principal quantities which can be controlled are:

- sand type,
- horizontal and vertical stresses,
- density,
- stress history (typically expressed as overconsolidation ratio).

In most of the work collected by Lanne et al. (1997) the lateral strain was kept as zero during sample preparation, and so the horizontal stress is largely controlled by the value of the overconsolidation ratio. Furthermore, a very high proportion of the tests are at an OCR of 1.0. This aspect of the tests means that they do not provide a means of separating out the influence of OCR and of horizontal stress.

In the field the OCR and \( K_o \) values are of course also correlated for many deposits in which the \( K_o \) value has arisen as a result of a simple deposition and erosion process. The tests from Italy/NGI/ Southampton will be relevant to these sites, and represent an impressive database of considerable value. They will not, however, be so relevant to sites with more unusual stress histories, which have resulted in an unusual combination of OCR and horizontal stress. Using the results for such sites would therefore be misleading. With this problem in mind, the focus of calibration chamber work at Oxford University has been on tests in which the stresses are controlled independently of the overconsolidation ratio.
The tests are carried out in sands that have been stressed to one of seven standard stress values, as shown on Figure 3. By comparing different combinations of tests the influence of horizontal stress, vertical stress and overconsolidation ratio can be examined separately. Early work by Houlshy and Hitchman examining the behaviour of the Marchetti dilatometer in sand (see Smith, 1993) led to the conclusion that the influence of the horizontal stress was much more important than the influence of overconsolidation. The conjecture is therefore that the differences observed in the data reported by Lummis et al (1997) for tests at different OCR values, are in fact principally due to the differences in horizontal stress, and not in the OCR itself.

A correlation which has been found to fit a large body of calibration chamber data reasonably well is:

\[
\log_{10} \left( \frac{q_f - q_{fr}}{\sigma_{fo}} \right) = 1.51 + 1.23D_R
\]

where \(D_R\) is the Relative Density (as a ratio). It is assumed that Relative Density can be readily converted to an indication of the friction angle by, for instance, the correlation published by Bolton (1980). The correlation is shown in Figure 4, where it can be seen that even well-controlled calibration chamber tests lead to quite a considerable scatter for different soils and conditions. Even more scatter can of course be expected for field tests. Correlations such as equation 3 (and the many others that have been published in the literature on interpretation of in situ tests) should be used with caution, and as approximate indicators only of soil properties.

6 THE SELF-BORING PRESSUREMETER

6.1 Approaches to Interpretation of the pressuremeter test

The pressure-expansion curve from self-boring pressuremeter test can be derived using simplified theories for either clay (Gibson and Anderson, 1981) or sand (Hughes, Wroth, and Wride, 1977). In each case the shape of the curve explicitly depends on the strength parameters (undrained strength for clay, angles of friction and dilation for sand) the shear modulus and the in situ horizontal stress.

The commonest way of interpreting experimental data is to plot it in ways that single parameters can be extracted from the experimental curves. Different practitioners use slightly different methods, but a common approach would be:

1. obtain the horizontal stress from an estimate of the "lift-off" pressure at which straining of the soil begins.
2. obtain the undrained strength by measuring the
slope of a reploted pressure-expansion curve as \( \psi \) against \( \ln(\psi) \) (Gibson and Anderson, 1961). There is an analogous procedure for sands, which requires also an estimate of the angle of friction at constant volume (Hughes, Wroth and Windle, 1977).

3. Estimate the shear modulus from the slopes of unload-reload loops. Whilst the above approach is well established, it has some drawbacks. The estimation of in situ horizontal stress from lift off pressures is, for instance, notoriously dependent on (a) any tendency to over-drill or under-drill the hole and (b) the engineer’s judgement.

An obvious alternative is to construct the theoretical curve for a pressuremeter test, and then tune the parameters used to define the curve so that the best fit is obtained. The curve can be obtained either from a simple formula, or perhaps from a numerical analysis. Shuttle and Jeffries (1995) term this process “iterative forward modelling”, and have used it with some success to fit pressuremeter test results. As theories for analysing pressuremeter tests become more sophisticated this approach becomes increasingly attractive. The danger is that, if the model used involves a large number of parameters, then equally good fits to the data (in practical terms) may be achieved by different combinations of parameter values. Some additional “intelligence” needs to provided during the fitting process so that unlikely values of parameters are avoided.

6.2 Unloading curves

An important development in the understanding of pressuremeter tests was the realisation that useful information could be extracted from the unloading curve as well as the loading curve. An analysis of the unloading sections of pressuremeter curves in sand was published by Houltsby, Clarke and Wroth (1986), and an equivalent analysis for tests in clay by Jeffries (1988). The importance of these analyses is that the unloading curves are insensitive to any imperfections in the drilling process, which affect the shape of the loading curve but not the unloading. The analyses are certainly “advanced” in that they involve quite complex mathematics. A careful track has to be kept of the stress history of elements of soil around the pressuremeter as they are (a) loaded elastically, (b) loaded plastically, (c) unloaded elastically and finally (d) unloaded plastically.

In spite of the complexity of the analyses, the final results are relatively straightforward. Given the expense of conducting a pressuremeter test, it should be routine practice always to obtain data from the unloading curve as this can provide useful additional data for interpretation.

6.3 Effect of length to diameter ratio

The pressuremeter is usually analysed on the assumption of plane strain conditions in the axial direction. This is equivalent to the assumption that the pressuremeter is infinitely long. This assumption is clearly questionable, since a typical self-boring pressuremeter has a length-to-diameter ratio of only about 6. The common assumption that the simplification of infinite length introduces only a small error is probably rooted in some early work in which elastic analyses of the pressuremeter were carried out. It is true that the stiffness measured by a short pressuremeter is only marginally higher than for an infinitely long pressuremeter (the difference is about 1.5% for \( L/D = 6 \)). The same is not true, however, once plastic deformation begins.

Yenag and Carter (1990) reported a study using finite element analysis in which the effects of pressuremeter length were taken into account. This study was extended by Houltsby and Carter (1994). Further work has been carried out on the effects of finite length by Yu (1990), Yao (1996) and Shuttle and Jeffries (1995).

The principal conclusions from the above studies are that in clay the measured stiffness from the pressuremeter test needs to be reduced by a factor which depends on (a) the stiffness of the clay and (b) the strain range over which the strength is measured (if the slope of the Gibson and Anderson (1961) plot is used). In sand the picture is slightly more complex, since the simplifications inherent in the Hughes, Wroth and Windle (1977) analysis tend to counteract the effects of the finite length. Yu (1990) gives details of corrections which can be applied, and these again depend on the soil stiffness.

6.4 Stiffness measurement with the pressuremeter test

The single most important issue in the measurement of the stiffness of soils that has become recognised in recent years is the strong dependence of stiffness on the amplitude of the strain. The characteristic “S-shaped” curve in the plot of \( G'/p' \) against \( \ln(\psi) \) is by now well known to geotechnical engineers. It should be recognised that the existence of this curve is itself proof that soil is not “elastic” except at
extremely low strain amplitudes (less than say $\Delta y = 10^{-5}$). The use of the terminology of an "elastic modulus" $G$ is therefore strictly incorrect, but has become common in practice.

In a triaxial test the shear strain amplitude is easy to calculate, using the usual simplifying assumption that the soil deforms as a right cylinder. Even if the reorientation of the specimen is taken into account it is found that the shear strain throughout most of the specimen correspond with closely the nominal calculated value. The stress conditions are also well defined in the triaxial test, so that the mean stress will be known. Thus it is possible to plot the whole of the $G^2/p'$ against $\ln(\Delta y)$ curve with considerable confidence about each data point (although the resolution of stiffness at small strain amplitudes requires particular instrumentation).

Consider now the measurement of the stiffness of a soil from unload-reload loops on a pressuremeter test. In principle, this is an excellent method of measuring stiffness since it avoids all the problems of sample disturbance that usually reduce the measured stiffness in the laboratory. An estimate of the mean effective stress during the cycle must be made so that the $G$ value can be reduced to a normalised value $G^2/p'$. The problem is twofold:

- The radial stress at the pressuremeter surface is known, but the hoop and axial stresses are not measured, and can only be estimated.
- All three stresses are varying with distance from the surface of the pressuremeter, so that (even if the full stress system could be estimated) a representative value of $p'$ has to be chosen from a range of possible values.

Of more importance is the fact that the strains undergone by elements of soil at different distances from the pressuremeter vary strongly with the radius. In an undrained test the strains are inversely proportional to the square of the radius (from the centreline of the pressuremeter). This means that for a typical SBSM test, with a pressuremeter diameter of 80mm, the strains in the soil about 85mm from the surface of the pressuremeter are only 1/10 of the value at the pressuremeter surface.

Figure 5 shows the results of some laboratory measurements of stiffness of clays, and shows that a tenfold change in the strain can have an enormous effect on the stiffness, especially in the range of strains from about 0.01% to 1%, which is typical of the strains used in pressuremeter testing. Unless proper account is taken of the variation of strain, then it is impossible to put measurements of stiffness in context with other results. Two approaches are considered here.

In the first approach we simply try to identify a representative strain for a pressuremeter test, in terms of the strain applied at the pressuremeter surface. There will be no unique solution, but the following analysis helps to resolve whether the measured stiffness is dominated by the material close to the pressuremeter or distant from it.

Consider the problem shown in Figure 6, which represents a highly idealised test. A pressuremeter of radius $a$ is surrounded by elastic soil with stiffnesses $G_1$ out to radius $r_1$, and outside that the soil has modulus $G_2$. It is straightforward to show that the measured shear modulus (for an undrained test in which $v = 0.5$) will be given by:

$$G_m = G_1 + \left( G_0 - G_1 \right) \frac{a^2}{r_1^2}$$

(4)

Taking the change of stiffness at the radius at which the shear strain will have dropped to only 1/10 of that at the pressuremeter surface, the factor $a^2/r_1^2$ at this radius is also 1/10. The measured shear
modulus would therefore be \( G_m = 0.5G_1 + 0.3G_0 \). This demonstrates that the measured modulus is very much dominated by the stiffness of the material close to the pressurerometer. The shear strain at the pressurerometer surface is therefore a reasonable estimate of an appropriate shear strain for interpretation of the modulus values.

The second approach is to investigate the way that moduli defined in different ways can be transformed. In a laboratory test we can define a secant modulus \( G_s = \frac{\psi}{\gamma} \), and a tangent modulus \( G_t = \frac{d\psi}{d\gamma} \). Similarly in a pressurerometer test in which pressure \( \psi \) is plotted against cavity strain \( \epsilon \), one could define a secant modulus \( G_{ps} = \frac{\psi - \sigma_{ps}}{2\epsilon} \) and a tangent modulus \( G_{pt} = \frac{1}{2} \frac{d\psi}{d\epsilon} \). The definitions of the moduli can be used to show that:

\[
G_s = G_t + \frac{dG_t}{d\gamma}
\]

\[
G_{ps} = G_{pt} + 2\epsilon \frac{dG_{pt}}{d\epsilon}
\]

Muir Wood (1990) showed that, for an undrained pressurerometer test, the Fahrner (1972) "subtangent" analysis leads to the result:

\[
G_s = G_{ps} = G_{pt} + 2\epsilon \frac{dG_{pt}}{d\epsilon}
\]

Thus the tangent modulus measured from the pressurerometer curve is equal to the secant modulus from a conventional laboratory test.

Muir Wood (1990) pursues the implications of the above relationships when particular forms of variation of shear modulus with strain are assumed. Here we explore the more general relationships. It is common to plot modulus against logarithm of strain (as in Figure 5), and it is useful to see how the moduli are related in this plot. Define \( x = \ln \gamma \) for a laboratory test and \( x = \ln (2\epsilon) \) for a pressurerometer test (it is straightforward to show that the maximum shear strain in the soil in a pressurerometer test is \( 2\epsilon \)). It can then be shown that:

\[
G_t = G_s + \frac{dG_s}{dx}
\]

\[
G_{pt} = G_{ps} + \frac{dG_{ps}}{dx}
\]

Figure 7: Links between definitions of the shear modulus

So that the relationships between the moduli are as shown on Figure 7 (note that the horizontal scale uses natural logarithms, not logarithms to base 10 as is commonly used). The different definitions of the modulus give rise to different curves on this plot. The values only coincide if the shear modulus is constant, in which case all the definitions reduce to the same value. This will only be the case at very low strains (typically \( \gamma < 10^{-5} \)).

For a substantial range of intermediate strains, the shear modulus (whatever the definition) falls approximately linearly with \( \ln (\gamma) \), so that each of the \( dG/dx \) terms is approximately constant, and the (approximately) straight sections of the three curves shown in Figure 7 will be parallel and equally spaced.

The importance of the above observations is that, while it must be recognized that the different definitions of the modulus give rise to different \( G - \ln (\gamma) \) relationships, these can be interrelated in a rational way. The results of pressurerometer tests can therefore be properly related to those of other tests.

7 THE CONE-PRESSUREMETER

The great advantage of the CPT is that it provides a detailed profile of properties with depth. The pressurerometer is better suited to accurate measurement of properties at spot locations. These complementary functions naturally led to the development of the cone-pressurerometer, which combines all the features of a CPT with some of those of a pressurerometer. The cone-pressurerometer simply consists of a pressurerometer mounted behind a standard 15cm3 cone.

The main obstacle to the understanding of the cone-pressurerometer test is that the pressurerometer test
is carried out not in undisturbed ground, but in soil which has been displaced by the cone. This means that an understanding of the cone penetration process is needed, so that the analysis of the pressuremeter phase of the test is started at the appropriate initial conditions. This exercise is not trivial, and has been one of the catalysts for the development of advanced interpretation methods.

### 7.1 Clays

The interpretation of the Cone Pressuremeter in clays needs to take into account the installation of the cone, and requires the use of large strain analysis. The analysis was made by Housby and Withers (1988), and concentrates principally on the interpretation of the unloading section of the test. This is in contrast with self-boring pressuremeter tests, where most information is obtained from the loading section.

The analysis gives rise to a simple geometric construction to determine the undrained shear strength, the shear modulus and the in situ horizontal stress. Studies of this procedure (Housby and Withers, 1988, Housby and Natt, 1990, Powell and Shields, 1995) indicate that: (a) the strength measurements correspond quite closely to those measured by other means, (b) the stiffness values are broadly comparable to those measured from unloading loops (although uncertainty about the appropriate strain range makes interpretation difficult), but (c) the implied horizontal stresses bear little resemblance to site values. Even when the effects of length-to-diameter ratio are taken into account (Yao, 1996) there is little improvement in the estimation of horizontal stress. One conclusion has to be that a full understanding of the mechanical processes involved in the test has yet to be achieved.

### 7.2 Sands

The analysis of the cone pressuremeter in sands is significantly more complex than the equivalent analysis in clay. This is principally because of the difficulties of large strain analysis in frictional, dilative materials. The study of this problem was, however, the catalyst for the solution obtained by Yu (1990) for the complete expansion and contraction of a cylindrical or spherical cavity in a cohesive/frictional material with dilation (see also Yu and Housby, 1991, 1995). The analysis suggests that again the unloading section of the test will yield most information, but comparisons between the analysis and test results are not entirely satisfactory. Work by Yu (1994) using models based on the state parameter approach is proving to be a more promising avenue, and the more advanced analysis in which changes of the angle of friction with stress level and density are taken into account appears to be amply justified.

At present the interpretation of the cone pressuremeter in sand is, like the interpretation of the CPT, largely empirically based. Schneid (1990) and Natt (1993) studied the cone pressuremeter in sand. They derived empirical correlations which allow the relative density and the horizontal stress to be estimated from the cone tip resistance \( q_t \) and the limit pressure \( \sigma_{hl} \) from the pressuremeter test. The basis of the correlations is that both the cone resistance and the limit pressure depend on two variables: the horizontal stress and the relative density. Approximate empirical expressions for relationships are (Natt, 1993):

\[
\frac{q_t - \sigma_{ho}}{\sigma_{ho}} = A + B D_R = 1.98 + 19.1 D_R
\]

\[
\frac{q_t - \sigma_{ho}}{\sigma_{ho}} = C + D D_R = 3.39 + 10 D_R
\]

The above equations can be solved simultaneously to give a quadratic in the horizontal stress:

\[
D(q_t - \sigma_{ho})(q_t - \sigma_{ho}) - A(\sigma_{ho} - a_p) = B(\sigma_{ho} - a_p)(q_t - \sigma_{ho}) - C(q_t - \sigma_{ho})
\]

which can be solved for \( \sigma_{ho} \). A simple back substitution then gives the value of \( D_R \).

Figure 8 shows a comparison between the measured horizontal stress in calibration chamber tests, with the horizontal stress deduced from the cone pressuremeter results using the above method. This figure shows that a reasonable estimate of the horizontal stress can be made with the cone pressuremeter. This position should be contrasted with the interpretation of the CPT, where one of the obstacles to interpretation was the fact that the horizontal stress was unknown.

Figure 9 shows the comparison of measured relative density with the estimate from the above procedure, and demonstrates that reasonable estimates of the relative density can also be obtained.

Manassero (1991) used a procedure similar to the above (although differing in detail) to combine the results of the CPT and the conventional self-boring pressuremeter to obtain estimates of horizontal stress in the field, and reported some success.
enigineering, allowing some engineering properties of soils to be measured that either
cannot be determined from laboratory tests, or are
less well determined by laboratory tests.

* Whilst some field tests can be interpreted by
  simple methods, others require more advanced
  methods for proper interpretation. Although
  simplicity has many merits, advanced methods
  (where necessary) should not be avoided.

* Soil classification from in situ tests has in the past
  been based primarily on the use of charts. Whilst
  this method is useful when only two quantities
  are measured, it becomes cumbersome when
  three or more measurements are made. Use of
  Neural Networks is a promising technique in
  which classification can be carried out using
  several input quantities. It has already been
  proven as a useful technique for distinguishing
  between different sands.

* Interpretation of the CPT in clay to determine
  undrained strength should take into account the
  value of the stiffness and the horizontal stress.

* Interpretation of the CPT in sand is based
  principally on the results of calibration chamber
  tests rather than analysis. Again the value of the
  horizontal stress should be taken into account.

* The unloading curve from a pressuremeter test
  provides useful information and is amenable to
  analysis.

* Pressuremeter test results should be corrected to
  take into account the finite length of the
  pressuremeter if overestimates of strength
  parameters are to be avoided.

* Stiffness measurements from the pressuremeter
  can be related to those of other tests, but
  appropriate transformations between different
  stiffness measurements must be used.

* The cone pressuremeter can be used to give good
  estimates of the undrained strength of a clay
  (based on a theoretical analysis) and for the
  density and horizontal stress of a sand (based on
  calibration chamber test results).

* The effects of length-to-diameter ratio are
  significant for the finite pressure measured with
  the cone pressuremeter.

9 ACKNOWLEDGEMENTS

The content of this paper is based to a large extent
on the work of many research students and assistants
at Oxford University, in particular those of Teh Cee
Ing, Fernando Schmid, Hai-Sui Yu, Nigel Nutt,
Brendan Rock and Mitsudaka Yao.
10 NOTATION

$D_R$  Relative Density
$f_f$  friction ratio
$f_s$  friction sleeve measurement
$G$  shear modulus
$\rho'$  mean normal stress ($\sigma_1' + \sigma_2' + \sigma_3' / 3$)
$q_c$  uncorrected cone resistance
$q_I$  total cone resistance
$q_t$  $(q_I - \sigma_0') / \sigma_0$
$q_u$  $(q_I - \sigma_{uo}) / \sigma_{uo}$
$s_u$  undrained strength
$\gamma$  shear strain
$\epsilon$  pressuremeter strain (tensile hoop strain at the pressuremeter surface)
$\nu$  Poisson’s ratio
$\psi$  pressuremeter pressure
$\psi_L$  Cone pressuremeter limit pressure

11 REFERENCES

Bolton, M.D. 1986 The strength and dilatancy of sands, Géotechnique, Vol. 36, No. 1, 63-78

Figure 10: Results of cone-pressuremeter tests with different length-to-diameter ratio (after Schnaid, 1990)
Houlshby, G.T. and Ruck, B.M. 1998 Interpretation of Signals from an Acoustic Cone Penetrometer, Proc. ISC '98
Hughes, J.M.O., Wroth, C.P. and Windle, D. 1977 Pressuremeter tests in sand, Géotechnique, Vol. 27, No. 4, 455-472
Last, N.C. 1982 The Cone Penetration test in Granular Soils, PhD Thesis, King's College, London University
Robertson, P.K. 1990 Soil classification using the cone penetration test, Canadian Geotechnical Journal, Vol. 27, No. 1
Robertson, P.K., Campahtna, R.G., Gillespie, D and Groig, J. 1986 Use of piezometer cone data, Proc. ASCE Specialty Conf. InSitu '86, Use of In Situ Tests in Geotechnical Engineering, Blacksburg, 1263-1280
Shuttle, D.A. and Jeffries, M.G. 1995 A practical geometry correction for interpreting pressuremeter tests in clay, Géotechnique, Vol. 45, No. 3, 549-553
Smith, M.G. 1993 A laboratory study of the Marchetti dilatometer, DPhil Thesis, Oxford University
Wroth C.P. 1984 The interpretation of in situ soil tests, 24th Rankine Lecture, Géotechnique, Vol. 34, No. 4, 449-489
Yao, M. 1996 A study of the effect of length to diameter ratio on the results of pressuremeter tests, MSc Thesis, Oxford University
Yu, H.S. 1994 Interpretation of pressuremeter unloading tests in sands, Report 100.07.1994, Dept. of Civil Eng. and Surveying, The University of Newcastle NSW
Objectives and planning for site investigations
QuickSite™, the Argonne expedited site characterization methodology

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ABSTRACT: Expedited site characterization (ESC), developed by Argonne National Laboratory, is an interactive, integrated process emphasizing the use of existing data of sufficient quality, multiple complementary characterization methods, and on-site decision making to optimize site investigations. The Argonne ESC is the basis for the provisional ESC standard of the ASTM (American Society for Testing and Materials). QuickSite™ is the implementation package developed by Argonne to facilitate ESC of sites contaminated with hazardous wastes. At various sites, Argonne has successfully implemented QuickSite™ and demonstrated the technical superiority of the ESC process over traditional methodologies guided by statistics and random-sampling approaches. For example, in a QuickSite™ characterization of a perch, a kafir at the Pantex Plant in Texas, past data and geochemical analysis of existing wells were used to develop a model for recharge and contaminant movement. With the model as a guide, closure was achieved with minimal field work.

INTRODUCTION

Argonne National Laboratory has developed a package of implementation tools to facilitate ESC of sites contaminated with hazardous wastes. The expertise (intellectual property, methods, software, etc.) gained by Argonne during the development and practice of ESC since 1989 (Burton et al., 1993; Burton 1994) has been consolidated into the QuickSite™ package being commercialized by Argonne. ("SM" signifies a service mark.) The commercialization will permit corporations with environmental problems and organizations within the environmental industry to gain access to Argonne's intellectual property without the time and expense of developing equivalent expertise and implementation tools. The ESC methodology is an interactive, integrated process emphasizing the use of existing data of sufficient quality, multiple complementary characterization methods, and on-site decision making to optimize site investigations. Throughout this paper, the terms "QuickSite™" and "Argonne ESC" are used interchangeably.

On March 7, 1996, President Clinton signed into law the National Technology Transfer and Advancement Act of 1995 (PL 104:113). Section 12 of this Act requires federal agencies to adopt and use, to the extent practicable, technical standards developed by voluntary, private-sector, industry-led, consensual bodies and to work closely with those organizations to ensure that the standards are consistent with agency needs. Federal agencies must report to the Office of Management and Budget their reasons if they do not use such standards. As a result, the U.S. Department of Energy funded the development of a provisional ASTM standard for ESC (ASTM 1997), with the Argonne ESC as the basis.

The QuickSite™ process ensures cost minimization and rapid closure for site characterizations, leading to correct remedial action decisions. For sites of the U.S. Department of Agriculture (USDA), Interior (DOI), Energy (DOE), and Defense (DOD), Argonne has successfully demonstrated the technical superiority of the QuickSite™ process over traditional methodologies guided by statistics and random-sampling approaches. At former facilities of the USDA, QuickSite™ reduced site characterization costs by 80-90% and time by 70-80%, compared to traditional methods (Burton 1994). DOE estimated that QuickSite™ saved $14 million and four years in a remedial site investigation at the Pantex Plant in Texas (Ferguson 1995).

Argonne's ESC is a flexible process that is neither site nor contaminant dependent. ESC can be tailored to fit the unique characteristics distinguishing one site from the next, in contrast to the traditional approach of making all sites conform to the same rigid, inflexible investigation regimen. QuickSite™ has
been applied successfully to remedial site investigations of landfills with multiple contaminants in the southwestern United States for the DOE, former grain storage facilities in the Midwest for the USDA, a weapons production facility in Texas for DOE, and closing and active military bases in several locations for DOE. The process can be applied both at sites that have seen little investigation and at sites subjected to many previous site characterizations without closure. In the latter case (e.g., at many DOE and DOD sites), QuickSite® offers a rapid solution, frequently with little additional field work.

QUICKSITE® FUNDAMENTALS

The QuickSite® approach to ESC emphasizes good scientific investigation principles and expert judgment. Key features are as follows: (1) The technical team leader has broad expertise in geosciences, and the multidisciplinary, geoscience-based team has strong field experience. (2) The team leader and team remain constant throughout the work and participate in all phases of the program, including all field activities. (3) The process uses multiple, complementary technical methodologies and emphasizes nonintrusive and minimally intrusive investigation methods. (4) High-quality data are required throughout the program for accurate decision making; screening techniques of lower quality are not used. (5) A dynamic work plan allows adjustments to the program in response to on-site data analysis and decision making.

Processes and costs of ESC and conventional site characterization methods have been compared in detail (ASTM 1997). The example below of the application of QuickSite® at the DOE Pantex Plant demonstrates the use of the approach within a large facility with a prior history of environmental investigations.

QUICKSITE® AT THE PANTEX PLANT

In 1993, DOE selected the Pantex Plant for the first application of the Argonne ESC to a DOE environmental problem. This project was to serve as a model for future implementation of the QuickSite® process at DOE sites by private and government contractors.

Site Setting and History: Pantex is located at an elevation of 1,148 m (3,750 ft) above sea level in the Southern High Plains region of west Texas in Carson County, approximately 27 km (17 mi) northeast of Amarillo (Figure 1). The regional climate is semiarid, with annual precipitation of about 51 cm (20 in.). Dry air, high temperatures, and moderate winds produce an evaporation rate as high as 178 cm/yr (70 in./yr) over exposed water areas. The area's principal topographic features are numerous shallow depressions called playas.

The Pantex Plant encompasses approximately 6,475 ha (16,000 acres). The plant has been operated as a weapons facility since 1942 and is now used primarily for the disassembly of nuclear weapons. The specific task set for the QuickSite® work at Pantex was characterization of a perched aquifer within Zone 12, in the southeastern portion of the facility (Figure 1). The objective was to determine the nature and extent of groundwater contamination beneath Zone 12 and to characterize the source and migration rate of the hazardous constituents. Numerous solvents, metals, pesticides, polychlorinated biphenyls, petroleum hydrocarbons, acids, inorganic compounds, and high explosives are used in operations and in near Zone 12, and discharges of these chemicals have occurred. In the past, surface drainage ditches in and around Zone 12 carried untreated process water to the nearby playas.

At Pantex, groundwater occurs directly above bedrock and in a shallower perched zone. Although both water zones occur in the unconsolidated Tertiary Ogallala Formation, the convention at the Pantex site is to refer to the shallow perched zone as the "perched aquifer" and the deeper zone as the "Ogallala aquifer." The aquifers are unconfined. The saturated thickness of the main Ogallala aquifer is 65.6-131.2 m (200-420 ft) in the vicinity of the Pantex Plant and exceeds 98.4 m (320 ft) to the northeast, where the aquifer is a primary source of

Figure 1. Location and plan of the Pantex Plant.
water for the city of Amarillo. The municipal well field is located within 1.6 km (1 mi) of the northeast corner of the Pantex Plant.

The upper part of the Ogallala Formation contains an apparently persistent zone of low-permeability, fine-grained material called the fine-grained zone (FGZ). This unit serves to percolate water above the main Ogallala aquifer in the vicinity of the Pantex Plant and more specifically in the area of Zone 12. Although water from the perched aquifer is not used, this groundwater zone is significant because it contains both organic and inorganic contaminants that could migrate downward into the Ogallala aquifer.

THE QUICKSITE™ PROCESS AT PANTEX

Existing Data: Argonne’s first task in the QuickSite™ process is assimilation, quality evaluation, and integration of all technically acceptable existing site data. Before Argonne’s QuickSite™ investigation on Pantex began, the large volume of data generated by previous investigations had not been evaluated or integrated to formulate hydrogeologic models consistent with all data for the site. After preliminary examination of the results of previous investigations, Argonne concluded that the existing database was mostly usable, with some modification and reinterpretation. Gaps and data of questionable quality were identified, but minimal field work was needed to fill the gaps. Therefore, no major field data acquisition activity was required for Argonne’s Phase 1 study.

Results of Previous Studies: A significant amount of background information on contamination of the perched groundwater in the vicinity of Zone 12 was accumulated in conjunction with ongoing and previous studies. Included were data from 25 monitoring wells installed in various Zone 12 studies and from other monitoring and domestic wells in the area. The conclusions from prior studies were as follows: (1) Recharge and hence contaminant transport were thought to occur mainly through the floors of the playa lakes. (2) The direction of groundwater flow in the perched aquifer was believed to be controlled by focused recharge through the playas (i.e., groundwater flowed radially from the playa centers). (3) Groundwater in the perched aquifer was thought to leak through the FGZ into the underlying Ogallala Aquifer. (4) The continuity of the perched aquifer was unclear; possibilities included segmentation of the perched aquifer and hydraulic isolation of the segments. (5) The toxic and isotopic concentrations in the perched and Ogallala aquifers were not considered particularly useful for determining the continuity of the perched aquifer or evaluating potential hydraulic connections between the two aquifers.

Evaluation of Existing Data: The QuickSite™ review of the contaminant and geochemical data from Pantex indicated that these data were largely inconsistent with prior conclusions. For example, the concentration of the high explosive HMX in the perched aquifer, based on the June 1993 sampling results, is shown in Figure 2. The pattern of higher concentrations directly under the Zone 12 area was observed for all contaminants. In addition, concentrations failed to decrease with distance from Playa 1 (the playa lake nearest Zone 12, considered to be the principal source of recharge for the perched aquifer in this area). Argonne’s review of contaminant sources and waste disposal practices revealed that the explosives wastes (e.g., HMX) were placed mainly in ditches in the eastern part of Zone 12. The HMX distribution suggested that recharge in the Zone 12 area occurred largely between the playas, by downward infiltration. Thus, Argonne found the existing contaminant distribution to be inconsistent with the previous model of playa recharge for the perched aquifer beneath Zone 12.

Argonne’s detailed geologic analysis, based on existing well logs, suggested that a gravel-filled channel crosses Zone 12 and opens to the southeast. Data from previous studies were insufficient to evaluate the effectiveness of the FGZ as a barrier to vertical water movement or to determine whether the absence of water at the perched level resulted from...
the absence of the FGZ or from lateral lithologic changes within the FGZ. Alternatively, Argonne believed that the water might be absent because the top of the FGZ occurs locally above the top of the perched water.

Argonne's review of 27 km (17 mi) of existing seismic reflection data for Zone 12 and adjacent areas uncovered several problems and inconsistencies in data acquisition, processing, and interpretation. Little information of geologic significance could be extracted from the survey. However, Argonne's work with a short section of one of the lines demonstrated that reprocessing of the original data could enhance the geophysical expression of the Zone 12 perched aquifer, possibly providing independent data to support a technically sound model for that aquifer. Hence, Argonne was able to identify specific areas for data collection and evaluation that would minimize field work and costs and would maximize the use of existing wells and data to answer questions about the groundwater.

PHASE I QUICKSITE\textsuperscript{SM} PROCESS AT PANTEX

The Phase I goal was to develop a working model for answering technical questions about the perched aquifer by using available data and existing monitoring wells, without an extensive field program. The basic QuickSite\textsuperscript{SM} multidisciplinary approach was followed, however. The program emphasized integration of all data (geologic, geophysical, and geochemical) within a dynamic work plan to begin answering questions and to constrain the system.

Phase I Geophysics Results: Reprocessed seismic lines showed strong, continuous reflections at arrival times approximately corresponding to the depth to the top of the perched aquifer, consistent with the seismic response predicted for this unconsolidated geologic section, where acoustic impedance is controlled largely by pore fluids. The perched-water reflection was seen on all seismic lines reprocessed by Argonne. However, in some segments the reflection was partially or completely attenuated, indicating that the perched water was too thin to be resolved in the seismic data or was absent. Integration of these data with the geologic constraints indicated locations for field testing to determine the extent of the perched aquifer.

Phase I Hydrogeology and Geochemistry Results: Geochemical and water level data collected by Argonne from existing wells suggested that the perched aquifer at Pantex might be present as at least two geochemically distinct and hydraulically separate aquifers. The perched aquifer under Playa 1 and north of Zone 12 appeared to be hydraulically separate from that under Zone 12. This conclusion was based on the presence of a dry hole at the location of well PTX06-1009 and differences in the stable oxygen, hydrogen, and carbon isotope compositions of groundwater samples. The location of well PTX06-1009 and the distribution of oxygen isotope compositions in the perched aquifer are shown in Figure 3. The isotopically heavier samples were all located near Playa 1 and between Playas 1 and 12. The apparent boundary between the two isotopically distinct perched aquifers coincided with the absence of the perched aquifer at well PTX06-1009.

The geochemical data indicate that recharge and hence contaminant transport to the perched aquifer in the vicinity of Zone 12 occur by downward percolation between playas. Recharge to the perched aquifer under Playa 1 occurs mainly from infiltration of slightly evaporated water through the playas. Differences in the isotopic geochemistry also indicate that groundwater recharged through the playas does not flow under Zone 12. These data entirely contradict previous models identifying the playas as the main source of water for the perched aquifer beneath Zone 12.

PHASE II QUICKSITE\textsuperscript{SM} PROCESS AT PANTEX

On the basis of the model developed by evaluating existing data and the multidisciplinary Phase I program, Argonne carried out a Phase II field program of soil boring, coring, and HydroPunch drilling.
sampling at locations delineated by the Phase 1 data integration. Argonne's geologic hypothesis was that the thickness and extent of the perched aquifer were controlled largely by significant relief on the upper surface of the perching layer (the FGZ) and by the distribution of gravels above the FGZ, suggesting the presence of a paleodrainage channel crossing beneath Zone 12 and emptying to the southeast. In Phase 1, seismic reprocessing showed that perched water is present beneath the Zone 12 vicinity, except for an area immediately south of Zone 12 and at two other locations to the west and northwest. The interpreted absence of the perched aquifer to the south of Zone 12 was consistent with the postulated location of the southern boundary of the channel.

The four soil borings drilled and cored to the FGZ in Phase II were placed to test this hypothesis and to define the extent of contamination to the southeast of Zone 12. Soil boring 1 was near the southern edge of the postulated channel, hole 2 was close to the center of the channel, and hole 3 was south of the channel. Drill site 4 was at the eastern boundary of the plant, on the southeast extension of the postulated channel. The locations of Phase II drill sites and geologic cross section A-A' are shown in Figure 4. The conventional Phase II approach planned prior to Argonne's involvement called for the installation of 13 perched-aquifer monitoring wells in a ring around Zone 12. However, all of Argonne's subsurface data were obtained from lithologic cores, geophysical logs, and groundwater samples without installing wells.

Phase II Results: Argonne's drilling showed that the FGZ is a dry, fine-grained sandstone with a variable clay content. Polygonsite is the most abundant clay mineral, followed by illite-smectite, a swelling clay. Polygonsite is known to form in acid environments characterized by high evaporation rates, such as floodplains, mud flats, playa lakes, and soil horizons developing under acidic conditions. The unsorted character of the core samples in thin sections and the presence of abundant polygonsite with calcite cement suggest that the FGZ formed at a paleoland surface under desert conditions. Depressions in the land surface deepened by erosion and subsidence could have localized the paleochannel that crosses beneath Zone 12. The relief on the top of the FGZ is at least 19.7 m (60 ft) in this area.

The dips in the top of the FGZ are the primary control on distribution of the perched aquifer and on the thickness of the water present (Figure 5). The thickest saturated zones occur where the top of the FGZ is lowest. The ridge on the top of the FGZ northeast of the main channel is structurally high enough to be above the perched water table at well PTX90-1009, which is dry. The existence of this ridge tending northwest to southeast was also suggested by hydrologic and geochemical differences between the segments of the perched aquifer to the north and south of the ridge. The constraint on the south side of the main channel is provided by the high on the FGZ at Argonne's drill site 3. Argonne postulated that the channel system continues off the plant site to the southeast.

Boreholes in the Argonne Phase II EOC field program were placed to test the hypothesis that the principal contaminants (largely high explosives) originated from plant activities on the eastern flank of Zone 12, migrated vertically to the perched aquifer in the interplaya area, then followed a southeasterly course dictated by the local hydraulic gradient, the configuration of the FGZ perching layer, and the distribution of hydraulically conductive gravels within paleochannels trending southeastward. Results of Argonne's HydroPunch sampling of the perched aquifer confirmed that high-explosive contamination extends to the eastern boundary of the Pantex Plant at drill site 4. At the request of the DOE, this site was converted to a monitoring well because of its critical location at the plant boundary dowgradient from Zone 12.

**DISCUSSION**

The QuickSite™ process at Pantex relied heavily on preexisting data and existing monitoring wells. This
The approach of maximizing the use of existing resources is a cornerstone of the Argonne ESC process. The multidisciplinary scientific team integrated data from many sources and site studies to develop a technically defensible model of the perched aquifer in the vicinity of Zone 12. This model was used to guide and focus characterization of the aquifer in Phase II. The minimization of Argonne's Phase I and Phase II field activities reduced DOE's projected schedules and budget for the Zone 12 field investigation significantly from the previous plan (Starke 1996). Additional savings were realized by canceling planned monitoring well installations, along with quarterly sampling and analysis. Reducing costs (time and money) while generating technically defensible data is the main goal of QuickSiteSM.

ACKNOWLEDGMENT

This work was supported by the U.S. Department of Energy (Assistant Secretary for Environmental Management) and by the U.S. Department of Agriculture (through interagency agreement) under contract W-31-109-Eng-38.

REFERENCES


Ferguson, D.J. 1995. The successful application of Argonne's expedited site characterization to DOE Pantex groundwater investigations. Oral presentation at Seventh National Technology Information Exchange (TIE) Workshop, Cincinnati, Ohio.

Site investigation and characterization for deep excavation

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ABSTRACT: As a site characterization for a deep excavation of 45 m, 35 boreholes were drilled. Among the boreholes, 3 were drilled down to 80 m deep and the others 60 m. The site consists of fill, deposit, residual soil and bedrock. The bedrock is stratified in highly weathered, slightly weathered, partly weathered and fresh sound rock. Major concern of this project was to decide proper supporting systems for excavation. To do this, the rock properties were very important so that a series of comprehensive tests was carried out in both field and laboratory. From analyses of the test results, it was decided to apply soldier beam and lagging systems with soil nailing in soil layers and highly to slightly weathered rock layers. In partly weathered to fresh sound rock layers, it was recommended to put a beam at the toe of the soldier beam system and to excavate directly to the bottom with rock-holing.

1 INTRODUCTION

The demand for the utilization of underground space in urban areas has been increased recently because of the high land price of urban areas in Korea. Therefore, it is a general trend to design a building with the maximum utilization of underground space. To meet this end, excavation is inevitably getting deeper and deeper. This leads to a question like what support system is the most appropriate in terms of construction safety as well as economy. To make a better decision, it is necessary to investigate sites thoroughly. This paper presents site investigation scheme and geotechnical characteristics of a deep and wide excavated area for the high-rise building complex that is being constructed in the southern part of Seoul.

Samsung Group initiated to construct the building complex named as Togok Synergy Park (TSP) located at Togok-Dong, Kangnam-Gu, Seoul. It will serve as offices, department stores, convention centers, hotels, recreational facilities, etc. The high-rise building will be supposedly 102 stories and its foundation is planned to be constructed at 45 meters below the ground level. The site is composed of two parts: the northern site (Site 1) with 100 by 200 meters and the southern site (Site 2) with 140 by 400 meters. Two subway lines are located at the vicinity of the complex site. The east-west bound subway is now in service and the north-south bound subway is still under construction. A department store is located at the east of the north and southbound subway line. A 100-meter wide stream, Yangjae-Chan, is also located at the south of the study area. The subsurface materials are fill, deposit, residual soil, and rock with various degree of weathering. In general, rock appears about 15 meters below the ground level. Our main concern is to estimate mechanical and geological properties of the rock masses, because the rock properties determine supporting systems, the excavation sequence, and the stability of adjacent structures.

As a site investigation, 35 boreholes as shown in Figure 1 were drilled to identify subsurface structures and to get soil and rock samples. Three boreholes in the high-rise building area were drilled down to 80 meters and the other boreholes were drilled down to 60 meters.

Georadarographic study was carried out to locate utility pipes such as electric power, communication lines, gas, water, etc. A borehole televizor (borehole camera) was also used in some boreholes to know the geology, conditions of intact rocks, the degree of weathering, the extent of fracture zones, and the characteristics of discontinuities. Pressuremeter tests were performed with a single packer to estimate the in-situ stresses of rock masses. Also, Lugeon tests were carried out in the field to get the hydraulic properties of various rocks. Laboratory tests such as uniaxial compression tests, triaxial compression tests and Brazilian tests were
fulfilled for the estimation of rock strength. Permeability, uniaxial compression and triaxial compression test for soil as well as the basic tests such as Atterberg limits, specific gravity, grain size analysis, etc. were performed.

2 GEOLOGY

2.1 General geology

The bedrock underlying a proposed site is classified as banded biotite gneiss, which belongs to the Pre-Cambrian. Biotite is locally changed into chlorite by metamorphism at low temperature, resulting in chlorite gneiss. The bedrock is generally dark-gray and consists of fine-grained particles. Major minerals are quartz, feldspar and biotite. The dip angles of the discontinuities of bedrock show a wide variation of 10° to 80°. It is, however, mostly around 50 to 70°.

Although joint sets of bedrock show wide local variations, joint surfaces are generally undulating (smooth to slightly rough) and tight or locally coated with calcite and/or chlorite. Many numbers of joints such as fracture zones and faults in a large or small scale were observed in fresh sound rock as well as partly weathered rock in the most of boreholes, except borehole B-8, 12, 13, 17, 18, 19 and 24 whose rock masses were relatively in a good state. Fracture zones contain a considerable amount of breccia, while the shear zone detected in borehole B-5 contains both clay and breccia. Even though clay and breccia were locally found, it can be concluded that most of the faults have a slickenside with little infilling materials.

2.2 Subsurface stratigraphy

The preliminary investigation reveals that subsoils are divided into 4 main strata according to their origins; fill, deposit, residual soil and bedrock. The configuration and properties of each layer are shown in Table 1 prepared based on the drill logs. A representative subsoil profile is shown in Figure 2 to illustrate the subsurface stratigraphy.

<table>
<thead>
<tr>
<th>Layer</th>
<th>USCSC</th>
<th>Thickness (m)</th>
<th>SPT N value</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>S/SM</td>
<td>2.3 - 4.4</td>
<td>2-33</td>
<td>all boreholes</td>
</tr>
<tr>
<td>Deposit (fine grained)</td>
<td>SC/AL</td>
<td>1.3 - 4.5</td>
<td>2-10</td>
<td>all boreholes except B-1, 6, 18, 22, 23, 24</td>
</tr>
</tbody>
</table>
| Deposit (coarse grained)  | SM/SW- 
SP/SM/GM/GM | 2.7 - 7.4     | 6-50°       | all boreholes |
| Residual soil             | ML/SM/GM | 0.9 - 9.2     | 25-50°      | all boreholes except B-6, 19 |
| Bedrock                   |       |               |             |         |

* USCSC: Unified Soil Classification System

![Figure 2. A subsoil profile of Section A-A](image-url)
3 LABORATORY AND IN-SITU TESTS

3.1 Borehole televiewer survey

Televiewer survey was performed in some boreholes at site 2; southwestern part (ZONE A) and northeastern part (ZONE B). Stereoplot projection and Rose diagram analyses were carried out and summarized as follows;

1) ZONE A (boreholes B-1, 2, 5 and 6)
   - fault: N70°E/80°SE is dominant.
   - clear fracture: similar with faults, N50°E/70°SE is dominant.
   - normal fracture: irregular, N40°E/70°SE is dominant.
   - minor fracture: N65°E/70°SE is dominant.

2) ZONE B (boreholes B-10, 11 and 22)
   - fault: N40°-50°E/60°-70°SE is dominant.
   - clear fracture: similar with faults, N45°E/70°SE is dominant.
   - normal fracture: N50°E/60°-70°SE is dominant.
   - minor fracture: strike of NE and NNE are dominant.

3.2 Ground water conditions

The ground water table was measured several times inside each borehole after the completion of drilling. Among 35 values, 18 representative ground water table values are given in Table 2. The ground water table was detected at depth ranging from 7.8m to 20.5m below the ground surface.

3.3 Uniaxial compression and point load tests

119 rock core samples from 35 boreholes were tested for uniaxial compression strength (UCS). The compressive strength of rock at Site 1 ranges from 165 to 955 kg/cm². On the other hand, the compressive strength of rock at Site 2 varies from 102 to 1,410 kg/cm², which shows wider variation than that of Site 1. Young's modulus and Poisson's ratio of rock in Site 2 scatters between 3.30 and 9.89 kg/cm², and between 0.18 and 0.28, respectively.

For heavily jointed rock whose samples were not appropriate for uniaxial compression test, point load tests were performed instead and their results were converted to uniaxial compressive strength values.

3.4 Brazilian tests and triaxial compression tests

In order to get rock strength parameters, 13 Brazilian tests and 13 triaxial compression tests were performed with rock cores from 7 boreholes. Table 3 shows the part of test results. As shown in Table 3, tensile strength of a rock is approximately half of cohesion strength of the rock.

3.5 Field permeability tests

Coefficients of permeability for deposit and residual soil were measured by pour-in method. Both constant head and falling head tests were adopted at 17 boreholes. The coefficient of permeability ranges approximately from 7.7×10⁻⁵ to 8.6×10⁻⁵ cm/sec for deposit layer and from 1.2×10⁻⁴ to 2.1×10⁻⁴ cm/sec for fill layer.

A total of 153 Lugeon tests were performed to estimate the coefficient of permeability in bedrock. A single packer was used in the tests and test interval was within 5 to 10 m in depth. The pressure was raised stepwise from 2 to 10 kg/cm² with the increment of 2 kg/cm² per a step and fell down in the reverse order. Discharge for each pressure step was measured and both coefficient of permeability and Lugeon value were estimated by the following equations.

Table 2. Ground water table

<table>
<thead>
<tr>
<th>hole no.</th>
<th>depth (m)</th>
<th>elevation (m)</th>
<th>borehole</th>
<th>depth (m)</th>
<th>elevation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>14.4</td>
<td>11.1</td>
<td>B-19</td>
<td>12.6</td>
<td>2.0</td>
</tr>
<tr>
<td>B-4</td>
<td>11.7</td>
<td>3.7</td>
<td>B-23</td>
<td>9.4</td>
<td>6.6</td>
</tr>
<tr>
<td>B-5</td>
<td>12.2</td>
<td>4.9</td>
<td>B-23</td>
<td>11.8</td>
<td>1.0</td>
</tr>
<tr>
<td>B-7</td>
<td>11.5</td>
<td>4.5</td>
<td>B-23</td>
<td>9.5</td>
<td>3.6</td>
</tr>
<tr>
<td>B-9</td>
<td>9.7</td>
<td>6.2</td>
<td>B-14</td>
<td>16.8</td>
<td>-2.7</td>
</tr>
<tr>
<td>B-11</td>
<td>9.5</td>
<td>2.4</td>
<td>B-19</td>
<td>7.8</td>
<td>6.1</td>
</tr>
<tr>
<td>B-12</td>
<td>16.7</td>
<td>-0.7</td>
<td>B-19</td>
<td>20.5</td>
<td>-6.9</td>
</tr>
<tr>
<td>B-13</td>
<td>9.4</td>
<td>6.5</td>
<td>B-15</td>
<td>8.9</td>
<td>6.0</td>
</tr>
<tr>
<td>B-17</td>
<td>17.9</td>
<td>-2.3</td>
<td>B-16</td>
<td>10.2</td>
<td>6.9</td>
</tr>
</tbody>
</table>

Table 3. Triaxial compression test results

<table>
<thead>
<tr>
<th>hole no.</th>
<th>Young's modulus (GPa)</th>
<th>Poisson's ratio</th>
<th>Tensile strength (kg/cm²)</th>
<th>Friction length (kg/cm²)</th>
<th>Cohesion length (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>35.1</td>
<td>-</td>
<td>90</td>
<td>47</td>
<td>170</td>
</tr>
<tr>
<td>B-4</td>
<td>45.4</td>
<td>-</td>
<td>89</td>
<td>46</td>
<td>150</td>
</tr>
<tr>
<td>B-5</td>
<td>21.7</td>
<td>5.37</td>
<td>90</td>
<td>43</td>
<td>160</td>
</tr>
<tr>
<td>B-7</td>
<td>42.8</td>
<td>6.30</td>
<td>70</td>
<td>43</td>
<td>130</td>
</tr>
<tr>
<td>B-9</td>
<td>34.6</td>
<td>6.57</td>
<td>50</td>
<td>44</td>
<td>90</td>
</tr>
<tr>
<td>B-11</td>
<td>46.7</td>
<td>-</td>
<td>100</td>
<td>44</td>
<td>210</td>
</tr>
<tr>
<td>B-13</td>
<td>50.5</td>
<td>-</td>
<td>100</td>
<td>42</td>
<td>180</td>
</tr>
<tr>
<td>B-15</td>
<td>45.1</td>
<td>7.85</td>
<td>50</td>
<td>33</td>
<td>90</td>
</tr>
<tr>
<td>B-17</td>
<td>78.9</td>
<td>5.24</td>
<td>80</td>
<td>47</td>
<td>140</td>
</tr>
</tbody>
</table>

from 165 to 955 kg/cm². On the other hand, the compressive strength of rock at Site 2 varies from 102 to 1,410 kg/cm², which shows wider variation than that of Site 1. Young's modulus and Poisson's ratio of rock in Site 2 scatters between 3.30 and 9.89 kg/cm², and between 0.18 and 0.28, respectively.
\[ K = \frac{2.3 \times Q}{2\pi L} \log_{10} \frac{L}{r} \]  

where \( K \) is coefficient of permeability (cm/sec), 
\( Q \) is discharge (cm^3/sec), 
\( L \) is test interval (cm), 
\( H \) is head loss (cm) and 
\( r \) is radius of casing (cm).

\[ Lu = \frac{10 \times Q'}{P \times L'} \]  

where \( Lu \) is Lugeon value, 
\( Q' \) is discharge (l/min), 
\( P \) is pressure (kg/cm²) and 
\( L' \) is test interval (m).

It turned out that the coefficient of permeability varied from \( 1.0 \times 10^{-6} \) to \( 8.0 \times 10^{-7} \) cm/sec and Lugeon value from 0.02 to 4.04.

3.6 Pressuremeter tests

A total of 35 pressuremeter tests were carried out at different depth within residual soils, highly weathered rock and partly weathered rock at selected borehole locations. For these tests, Elastometer-200 made by Oyo Co., Japan was used. A digital indicator recorded pressure (p) and the deformation of sonde (\( \epsilon \)). A p-\( \epsilon \) curve was plotted from a series of recorded data and borehole lateral reaction modulus (\( K_r \)) and deformation modulus (\( E_r \)) could be calculated by the following equations (Dohwa, 1990):

\[ K_r = \frac{6p}{\Delta r} \]  

\[ E_r = (1 + \mu) \times r_a \times K_r \]  

where \( r_a \) is the radius of sonde, 
\( \mu \) is Poisson’s ratio (assumed to be 0.3), 
\( E_r \) is deformation modulus (kg/cm²), and 
\( K_r \) is borehole lateral reaction modulus (kg/cm²) which denotes the slope of the straight portion of a p-\( \epsilon \) curve.

Table 4 shows borehole lateral reaction and deformation moduli at 10 boreholes inferred from pressuremeter tests. In total, 35 tests were performed. As a whole, deformation modulus shows a wide variation, 70 – 360 kg/cm² in residual soil, 717 – 15,000 kg/cm² in slightly or partly weathered rock and 7,000 – 41,000 kg/cm² in fresh sound rock. There is a general tendency that deformation modulus increases as depth increases. Figure 3 shows a p-\( \epsilon \) curve from a pressuremeter test at borehole B-8.

Based upon drill logs and test results, rock masses were classified according to RMR method. The deformation modulus (\( E_r \)) shown in Table 5 was inferred from the following correlation with a RMR value (Bieniawski, 1984):

\[ E_r = 10^{0.20 \times \text{RMR}} \quad \text{Gpa} \quad \text{if RMR} \leq 50 \]  

\[ E_r = 2 \times \text{RMR} - 100 \quad \text{Gpa} \quad \text{if RMR} > 50 \]  

The deformation moduli (\( E_r \)) obtained from pressuremeter tests were correlated with \( E_r \) as shown in Figure 4 and the following correlation between \( E_r \) and \( E_r \) was achieved;

\[ E_r = 4.26 \times E_r \]
Table 5. Deformation moduli

<table>
<thead>
<tr>
<th>Hole no</th>
<th>depth (m)</th>
<th>RMR rating</th>
<th>$E_u$ (GPa)</th>
<th>$E_r$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>15.6-15.9</td>
<td>20</td>
<td>18,146</td>
<td>4,367</td>
</tr>
<tr>
<td>B-3</td>
<td>21.7-24.9</td>
<td>18</td>
<td>16,172</td>
<td>5,097</td>
</tr>
<tr>
<td>B-5</td>
<td>28.8-37.9</td>
<td>10</td>
<td>10,204</td>
<td>3,667</td>
</tr>
<tr>
<td>B-6</td>
<td>20.4-25.0</td>
<td>31</td>
<td>34,180</td>
<td>25,886</td>
</tr>
<tr>
<td>B-9</td>
<td>23.0-25.0</td>
<td>24</td>
<td>25,031</td>
<td>5,826</td>
</tr>
<tr>
<td>B-11</td>
<td>18.7-23.0</td>
<td>15</td>
<td>13,607</td>
<td>9,408</td>
</tr>
<tr>
<td>B-12</td>
<td>12.1-15.7</td>
<td>56</td>
<td>163,565</td>
<td>24,847</td>
</tr>
<tr>
<td>B-13</td>
<td>35.3-46.5</td>
<td>36</td>
<td>45,380</td>
<td>7,569</td>
</tr>
<tr>
<td>B-17</td>
<td>30.2-44.8</td>
<td>50</td>
<td>102,041</td>
<td>18,733</td>
</tr>
<tr>
<td>B-19</td>
<td>12.6-16.1</td>
<td>20</td>
<td>16,146</td>
<td>14,771</td>
</tr>
<tr>
<td>B-22</td>
<td>16.0-16.3</td>
<td>24</td>
<td>22,844</td>
<td>5,361</td>
</tr>
<tr>
<td>B-23</td>
<td>16.8-21.0</td>
<td>24</td>
<td>22,844</td>
<td>8,791</td>
</tr>
</tbody>
</table>

Figure 4. Correlation between $E_u$ and $E_r$

4 CONCLUSIONS

The considerations to decide an excavation method and supporting system in the site of interest are as follows:

1. the site is very wide and long
2. excavation is carried out in considerable depth
3. excavation can take very long
4. important structures are located near the site such like subways, department store, etc.
5. a stream is located beside the site

With the results of site investigation and the above considerations, it is decided to apply soldier beam and lagging systems with soil nailing in soil layers and in highly to slightly weathered rock layers. In partly weathered to fresh sound rock layers, it was recommended to put a berm at the toe of the soldier beam systems and to excavate directly to the bottom with rockbolting. Even in the lagging system with soil nailing, a considerable amount of deformation is expected. Therefore, chemical grouting around the site is needed to reduce it. Presently, the site has been excavated down to the soil layers and highly weathered rock layer.

REFERENCES


Aspects of geotechnical site characterization for large international power projects

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ABSTRACT: The number of large international power projects designed and built by U.S.-based firms has increased over the last several years. Geotechnical site characterization for international projects has become ever more challenging because of issues such as available equipment, site access and equipment mobility, language and cultural differences, and approach to field work. This paper provides a general discussion of these issues and illustrates some of them using examples from actual geotechnical site characterization programs for large fossil-fueled power plants. These issues also apply to other types of large international projects. An increased awareness of them is important to successfully developing and implementing geotechnical site characterization programs for large international projects.

1 INTRODUCTION

Implementation of large lump-sum, turnkey energy projects includes the participation of multi-party management, engineering, procurement and construction groups. Engineering representatives from the owner, the engineering/procurement/construction (EPC) contractor, financial institutions, and regulatory agencies are often involved. This large number of engineering parties results in heightened attention to all phases of the project, and consequently in the need to adhere to tight budgets, schedules, and quality standards. Subsurface data collection for geotechnical site characterization thus becomes a highly visible task, deserving of utmost care. When these projects are located outside developed countries, such as those of North America and Europe, additional difficulties often arise due to issues such as available equipment, site access and equipment mobility, language and cultural differences, and approach to field work. On occasion these issues tend to affect the quality and consistency of subsurface data and bring undesirable consequences.

This paper summarizes the authors' observations related to their experience in developing and implementing geotechnical site characterization programs for international projects. The statements and opinions expressed in the following sections are by no means intended to minimize the quality of field personnel in the countries where these projects are located. The statements and opinions are based on experience in more than 10 countries, but should not be generalized. The focus of this paper is to bring awareness of these issues to U.S.-based geotechnical engineers working on overseas projects, but the information it presents could be used by any geotechnical engineer working in a country other than his/her own. Lack of awareness and/or improper handling of these issues can result in lengthened site characterization schedules and cost overruns. On occasion, the result may be the collection of data of questionable quality with potentially serious impacts on construction costs and schedules.

The paper is written from the perspective of a large EPC contractor with its own in-house geotechnical staff that provides support to earthwork and foundation activities from proposal preparation to construction (Drilling and laboratory testing services are subcontracted to local firms.)

2 GEOTECHNICAL SITE CHARACTERIZATION PROGRAMS

A typical geotechnical site characterization program for a large international fossil-fueled power plant project generally includes, as a minimum, soil and/or rock borings with sampling, installation of ground water observation wells, cone penetrometer test (CPT) soundings, electrical resistivity testing, and
laboratory testing. Field tests such as geophysical seismic refraction surveys, seismic crosshole tests, pressuremeter and dilatometer tests, and field permeability tests are often performed as well. Site characterization programs often include both onshore and offshore investigations. Onshore investigations are used to develop subsurface data related to earthwork and foundations for the plant and ancillary structures, while offshore investigations are needed to develop subsurface data for fuel unloading structures and cooling water intake and discharge structures. These investigations can provide preliminary subsurface data for cost estimating purposes or detailed design/construction data.

The cost of geotechnical site characterization programs for large international power projects depends on factors such as the size of the plant, amount of available geotechnical information, and scope of the investigation. A preliminary onshore site characterization program consisting of several borings and CPTs can cost US$20,000 to US$50,000, while a detailed program including both onshore and offshore investigations can cost several hundred thousand U.S. dollars in drilling and testing subcontractor’s costs alone. Travel, subsistence and jobshop costs for a geotechnical engineer to provide field technical direction can run an additional several thousand dollars.

An understanding of how geotechnical site characterization programs are implemented in other countries can help control the large costs associated with them. Moreover, the potential for even larger costs during construction when unexpected conditions are encountered makes it absolutely necessary to obtain reliable subsurface data.

3 EQUIPMENT

Truck and ATV-mounted equipment is largely unavailable outside developed countries. Available drilling equipment often consists of small skid-mounted rotary drill rigs with a tripod for standard penetration test (SPT) and sample retrieval. Because the hydraulic power of these rigs is rather small, hollow stem augers are generally not used and boreholes are advanced by wash methods. Standalone pumps are used to circulate water and/or drilling mud. An example of such a rig is shown in Figure 1 (photo taken September 1997).

The vertical travel distance of the hydraulic pistons in drill rigs such as the one shown in Figure 1 is generally small (0.5 to 0.6 m), thus requiring additional time to advance the borehole. Because of the small size of these rigs, Shelby tubes often cannot be pushed. The typical drilling rate in soils for these types of rigs is 10 m/day (30 ft/day), sometimes even less. The typical coring rate in rocks is 3 m/day (10 ft/day), sometimes even less.

Sometimes drilling involves pure craftsmanship, as illustrated in Figure 2 (photo taken March 1997). At this site a drilling rate in soils of a little over 10 m/day was achieved, despite the almost completely manual procedures being employed. Regardless, these drilling/coring rates are substantially lower than those achieved in the U.S., for instance, with the use of truck and ATV-mounted rotary drill rigs.

Figure 3 illustrates the fact that modern, truck-mounted rigs sometimes can be found outside developed countries (photo taken October 1997).

Equipment for offshore subsurface investigations is often inadequate outside developed countries. The tendency is to use floating barges or boats, which are relatively inexpensive. However, unless such equipment is used in fairly calm waters, it does not provide enough stability for high quality data collection. Our experience has included attempts by subcontractors to use bamboo rafts, two boats tied together (Figure 4, photo taken March 1997), etc.,
for offshore drilling. The result is always the collection of data that are at least partly inadequate. It is quite clear that offshore investigations have to be performed from stable platforms (jacksup) to provide adequate data. This type of equipment is generally very expensive, but justified for large-scale tunnel projects.

4 SITE ACCESS AND EQUIPMENT MOBILITY

Site access outside developed countries is sometimes difficult and can add considerably to daily work schedules. During a recent site characterization program the travel time between the site and the closest town, located about 30 km (about 20 miles) away, was 2 hours. This required an additional 4 hours to achieve an 8-hour daily work schedule, with obvious implications to jobhour budgets, as well as less tangible effects on field personnel such as fatigue, etc., which tend to affect work quality.

On site equipment mobility can hinder work progress, thus lengthening schedules. Skid and tripod rigs have no room in which accessories such as drill rods, hoses, tool boxes, bentonite bags, casing, PVC pipes for wells, sand bags, etc., can be stored. As a result, moving the equipment generally involves several trips, often on foot because of the lack of support vehicles. During a recent site characterization program the skid rigs had to be moved manually from one borehole location to the next. Because of the multiple trips required to move the accessories, and unfavorable topographic features of the site, this operation took about 1 day.

5 LANGUAGE AND CULTURAL DIFFERENCES

Communication with drilling and testing subcontrac-
tors in other countries before contract award can be difficult and sometimes misleading. For example, enforcement of U.S. standards for a subsurface investigation and laboratory testing specification may not be entirely feasible. The reasons are related mostly to equipment, but also to local standards and practices that can be different from U.S. standards.

During a recent bid clarification period for a large onshore and offshore site characterization program the subcontractor indicated that they could not comply with ASTM D 1586 requirements for SPT refusal. Preliminary subsurface information from the site indicated the presence of residual soils and rock, so that all parties involved knew ahead of time that SPT refusal would be encountered in basically all boreholes. When asked why they would not be able to comply with the specified requirement they stated that they were concerned about damage to their split barrel samplers. It was mutually agreed that payment would be made for their damaged samplers so that standard sampling procedures could be followed.

Communication between the EPC’s field geotechnical engineer and local field personnel is often a problem. While drilling and testing subcontractors in some countries can provide field superintendents (geotechnical engineers or geologists) who are fluent in English, this is not always the case. Large U.S.-based engineering companies and EPC contractors have multi-cultural, multi-ethnic work forces and often can identify in-house geotechnical personnel who can provide field technical direction in many languages. However, it is sometimes necessary to assign an in-house geotechnical engineer or geologist without the necessary language skills. In these cases it may be necessary to hire a local engineer (not necessarily geotechnical) with the necessary language skills to provide communication between the U.S.-based geotechnical engineer/geologist and local field personnel. This adds cost to the program and sometimes does not completely eliminate communication difficulties.

Communication between the overseas jobsite and the home office can be difficult unless the site is near a metropolitan area. While the necessary facilities may not be available at the site, most hotels around the world at least have fax machines. The proliferation of telephone calling cards and online service providers also helps make communication easier.

Cultural differences can also add difficulties to communication and work progress. The standard approach to conducting business and delegating responsibility in one country may not necessarily apply to the country where the site characterization program will be implemented. Decision making is often highly hierarchical and can involve several management levels. As a result, what is ultimately decided in terms of equipment and schedule is not always what was expected.
6 APPROACH TO FIELD WORK

Field site characterization techniques used in other countries are mostly similar to those used in the U.S. Differences in standards exist, but the major differences are related to equipment, as previously stated. Unfortunately, equipment limitations and site access difficulties can lead to poor quality data. On occasion, field techniques are simply inadequate and have to be brought to the attention of the subcontractor’s field superintendent. This section provides a few examples of poor quality and/or conflicting data resulting from equipment limitations and/or inadequate field techniques. It also illustrates the fact that the EPC engineer and the drilling subcontractor have to work together and find innovative ways of obtaining reliable results using the available equipment.

In a recent job the drilling subcontractor initially had to drive the sampler to obtain “undisturbed” samples because of lack of counterweight. The subcontractor was instructed to use ground anchors and a jacking system to obtain subsequent undisturbed samples. However, these samples were tested in a laboratory located about 250 km (155 miles) from the site. The travel time between the site and the lab was about 8 hours because of poor road conditions. When tested these “undisturbed” samples disclosed consistency and shear strength values that were much lower than expected from the SPT results, which were typical of hard clays. Conflicting data such as these could significantly affect this project. Fortunately, a cone penetrometer was readily available that could be used to confirm the hard nature of the clay, thus averting what could have become a major source of disagreement.

At another site, the drilling subcontractor was observed using a nonstandard sampling technique early in the site investigation. The subsurface conditions included fine sands at depths of about 3 m below the ground surface. No casing or drilling mud was used to keep the boreholes from caving in. When the split-barrel sampler was lowered into the borehole, it sometimes would stop 1 m or more above the desired sampling depth, i.e., the depth to which the borehole had been washed. The split-barrel sampler would then be driven with a 63.5 kg hammer to the desired sampling depth, at which point the SPT blow count would be started. This technique had two major deficiencies. First, the sample being collected did not necessarily represent the conditions at the desired sampling depth because the sampler was probably filled with soil during the initial driving. Second, the SPT blow counts were significantly increased because the initial driving probably resulted in a full sampler and increased friction along the sides of the sampler and rods. This nonstandard technique was brought to the attention of the subcontractor, and the remaining borings were drilled and sampled using standard procedures. Figure 5 illustrates the effects of the nonstandard technique on the SPT blow counts. Borehole B1 was drilled using the nonstandard technique, and borehole B2 was drilled using the standard procedure, which consisted of stabilizing the borehole and taking a sample at the bottom of a clean borehole. The SPT values for the lower sands in borehole B1 are two times higher than those for borehole B2. The SPT values for borehole B2 are typical of all other boreholes and were confirmed by the use of a cone penetrometer. Clearly, if the nonstandard sampling technique had not been observed early on, the impact on the selected foundation system could have been significant. For instance, a short pile could have been selected that would not provide the necessary capacity. The effects of such a selection on project cost and schedule would be disastrous.

Poor equipment, and perhaps also inexperienced field personnel, led to misleading subsurface data at another site. This site is located on a coastline and includes a low lying area near the coast where a power plant is currently being built and 60 m hills where a large coal storage area will be located. The
site requires substantial grading and shore protection. An initial investigation was carried out, and highly fractured "rock" was cored on the hilly portions of the site. These cores often had very low recoveries, and the "rock" was generally beneath a thin layer of soil. It was unclear whether the "rock" was fractured naturally or as a result of the poor equipment being used and/or the apparent lack of experience of the field personnel. Geophysical seismic refraction testing was also performed along several lines and disclosed compressive wave velocities that indicated that the "rock" might need to be blasted (the values were not high enough to conclusively indicate the need for blasting). Because the hilly portions of the site would have to be cut well into the rock, a conservative approach was taken and an earthwork specification was issued that included blasting for rock excavation. A supplementary investigation was performed by a different subcontractor before the start of earthwork activities. The supplementary investigation disclosed highly fractured rock much deeper than disclosed by the initial investigation. Actually, it appeared that the "rock" encountered during the initial investigation consisted of rock fragments within a matrix of very hard residual soil. During the initial investigation, coring resulted in the soil matrix being washed out of the boreholes, while the rock fragments were kept in the core barrel. The results of this finding had two effects on earthwork costs for the project. The first was mostly beneficial because rock blasting was not necessary, and excavation was thus less expensive. The second was mostly negative because rock for shore protection had to be imported.

7 CONCLUDING REMARKS

The discussions and examples provided reflect the perspective of U.S.-based geotechnical engineers working for large engineering and/or EPC contractors, but could be generalized to geotechnical engineers developing and implementing site characterization programs in countries other than their own. An increased awareness of these issues on the part of geotechnical engineers working on overseas projects is important to successfully developing and implementing geotechnical site characterization programs in other countries.

Based on the authors' experience, and as illustrated by some of the examples provided, successful geotechnical site characterization programs for large international projects should include, at a minimum:

- A technical specification that uses U.S. standards, leaving room for use of alternative procedures and equipment pursuant to prior approval.
- A requirement for fixed platforms for offshore drilling, if meaningful data are to be obtained. Floating platforms can be used only when drilling in calm waters.
- Bid clarification conference calls, or, preferably, a face-to-face meeting with the subcontractor to close all outstanding technical issues (equipment to be used, field procedures, etc.).
- The presence of a geotechnical engineer at the site to work collaboratively with the subcontractor to resolve procedural issues and propose alternative techniques as needed.

In the authors' opinion, it is also imperative that the field and laboratory data for large, lump-sum, turnkey projects have to be completely reliable. If the use of nonstandard procedures is observed, it should be corrected immediately. If such correction is not possible, it is better to use alternative procedures that will result in reliable data. This can only be achieved if a geotechnical engineer is assigned to the field.
Precise geological characterization for the wider application of high quality site data

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ABSTRACT: Where good quality data have been obtained from careful monitoring of in-situ instrumentation, it is essential that the exact geological conditions are fully analysed. Without this it is impossible to check the design assumptions or to apply the results to similar situations elsewhere. The problem is illustrated with reference to a monitoring programme currently underway for a bored pile retaining wall supporting an 8m high cutting in a sequence of sandstones and mudstones in Coventry, England.

1 INTRODUCTION

The progress of geotechnical engineering requires a flow of high-quality site data obtained from reliable instrumentation and careful monitoring. It is the means whereby theories and design assumptions can be checked and the data back-analysed to determine fundamental soil and rock properties. However, if these data are to be applied more widely, rather than becoming site specific to a one-off case, it is necessary that the geological conditions are characterised in a way which makes it possible for the interpretations to be applied in a wider context. This requires more than a purely routine investigation of the geology and, in sites with complex geological structures, a thorough investigation of the full complexity.

This principle is illustrated in the paper by reference to a quality A investigation currently being carried out for a retaining wall supporting a highway cutting in Coventry, Warwickshire, England. The retaining structure is a bored pile wall with a stabilising slab supporting an 8m high cutting in weak rock. An intensive set of instrumentation and monitoring, which includes inclinometers within two of the piles, vibrating wire embedment strain gauges installed within two adjacent piles, pressure cells beneath the stabilising slab together with extension monitoring of the wall top, is designed to yield high-quality data on the deformation of the pile wall (Figure I).

2 OBJECTIVES

The site history and geology are complex. An extensive Site Investigation had been carried out but further data was required to permit the wider aims of checking the design assumptions and interpreting the results of the monitoring programme. The geological character of the site had to be more precisely defined. There were at least five main questions that needed an answer:

1. What geological strata are present at the exact location of the instrumented piles?
2. What are the lithologies and geotechnical properties of these particular strata?
3. What discontinuities are present and how might they influence the behaviour of the strata?
4. Would these strata behave as soils or rocks?
5. Would the weak rocks be subject to deterioration due to weathering during the engineering lifetime of the retaining wall?

In order to answer these questions engineering geological studies have been carried out on the strata exposed during the construction work; over the whole length of cutting shown in Figure 1. This work has been added to, and to some extent re-interpreted, the original, pre-construction Site Investigation. Geophysical tests to determine stiffness profiles have also been carried out and reported to this conference (Hope et al. 1998).

3 SITE HISTORY

The site forms part of the Coventry North South Road which links the M6 motorway to the north of
Coventry, with the M5 trunk road and M40 motorway to the south. The central section commences approximately 1.5km north-east of Coventry City centre and is approximately 1.5km in length. The road mostly follows the line of a disused railway constructed early this century. In the area of the research, the railway was located within a cutting with side slopes of approximately 60°.

The new road has been designed as a dual carriageway which requires widening the base of the original railway cutting. However, the area is urbanised with both residential and light industrial development so that it was not possible to widen the original railway corridor. The new road could only be accommodated by increasing the slopes to vertical and for which a strong retaining wall was deemed to be necessary. In the area of study, this is designed as a bored pile wall with a stabilising slab, to support the 8m high cutting (Figure 2).

4 GEOLOGY

The site geology is more complex than first appears. The principal formations present in the area are:
- Fill: mainly ashes and topsoil (Recent)
- Glacial Till (Quaternary)
- Bromsgrove Sandstone Formation (Triassic)
- Coventry Sandstone Formation (Carboniferous)

The geology at the instrumented section of the cutting is shown in Figure 3. The Glacial Till which is of variable thickness, is not present at the instrumented section and the Fill is of negligible thickness. Owing to the similar lithology and lack of clear evidence the Bromsgrove Sandstone Formation and Coventry Sandstone Formation are difficult to distinguish and since they contain similar lithofacies there is some doubt as to where the junction of these two formations lies. However, based on field studies and literature of the area there is reason to believe that the whole of the cutting comprises Bromsgrove Sandstone Formation (Sumbler 1989, Old 1988, Old et al. 1990 and Crossland, personal communication).

The Bromsgrove Sandstone Formation consists of cross bedded sandstone interbedded with mudstone. The sandstone is grey to buff, with a gypsum or calcite cement when fresh, but weathers to a soft buff or brown rock. It is generally well sorted, containing fine to medium grade quartz and feldspar, and commonly abundant mica, concentrated on bedding planes and cross-laminite. The mudstone units are generally 1 to 2m thick, and of a dark red-brown colour but in some cases mottled greenish grey. They are usually fairly massive and may be silty or sandy (Sumbler 1989).

The sandstones often grade upwards into mudstone, forming fining-upwards cycles suggesting a
fluvial environment. The sandstones represent the deposits of migrating river channels, and the mudstones, over-bank material deposited during flood stages. As would be expected from this model, individual sandstone and mudstone units are lenticular in form (Newells 1989).

The Browngreave Sandstone Formation is regionally tilted very gently to the east or south-east (Old 1983). Locally in this area however, the observed dips are at variance with this broader interpretation due to the undulating nature of the near horizontally bedded deposit.

After installation of the piles, the remaining rock in the cutting was excavated back to the pile rims. As seen in Figure 4, the material adhering to the pile picks out the stratigraphy seen in the geological section, Figure 3.

Weathering

Weathering of the sandstone occurs along the discontinuities and in part of the rock mass. The sandstone at the very top of the cutting is frequently completely weathered. Lower down the strata is typically slightly to moderately weathered.

Study of the strata exposed by the construction work indicated that any recent weathering undergone on the faces of the original railway cutting was relatively superficial. There was no evidence for any deep deterioration of the strata within the slope itself. This gives reason to believe that with the additional protection of the retaining wall there would not be any significant reduction in the engineering performance of the strata within the foreseeable lifetime of the retaining wall.

5 DISCONTINUITIES

Three types of discontinuities were identified as shown in Table 1. The orientation data plus other observations were recorded in accordance with ISRM Suggested Methods (1981). The data collected was then analysed using the computer software program 'DIPS' which displays data using stereographic projection. DIPS is designed for the interactive analysis of orientation based geological data and was developed by M.S.Diederichs and E.Hook of the Rock Engineering Group, Department

Figure 2. A view of the highway construction looking North. The pile cap in the foreground extends across the instrumented piles of the bored pile wall. Beyond this, the support to the cutting is a conventional concrete retaining wall with ground anchors. In the right foreground is a temporary prop placed during the excavation phase and while the stabilising slab is constructed at sub-grade level. The photograph also shows the high density residential housing which prevented any extension of the original railway corridor with its 60° side slopes.
Table 1. Mean orientation for each discontinuity set.

<table>
<thead>
<tr>
<th>Discontinuity</th>
<th>Orientation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint Set 1</td>
<td>230 / 85 SW</td>
</tr>
<tr>
<td>Joint Set 2</td>
<td>139 / 85 SE</td>
</tr>
<tr>
<td>Joint Set 3</td>
<td>235 / 21 SW</td>
</tr>
<tr>
<td>Bedding</td>
<td>068 / 07 ENE</td>
</tr>
<tr>
<td>Fault Set A</td>
<td>234 / 84 SW</td>
</tr>
<tr>
<td>Fault Set B</td>
<td>166 / 85 SE</td>
</tr>
</tbody>
</table>

Orientations are quoted in degrees as dip direction / dip amount followed by a compass orientation of the dip direction.

5.1 Joints

Joint opening will have occurred in two separate phases, a geological phase, due to unloading by uplift and erosion, and a recent phase, due to the removal of material during the construction of the railway. Joint spacing is typically in the region of 150mm to 500mm (closely to medium spaced discontinuities, Anon 1981). Several of the joints have been infilled with a light grey fine sand or clay. The amount of infilling is usually around 10mm although as much as 30mm occurs in some joints.

5.2 Bedding

The frequency of bedding increases towards the top of the cutting. This is also likely to be a function of stress relief due to the two phases of overburden removal. Bedding at the top of the cutting is around 50mm (very thinly bedded) whilst lower down in the cutting it is 300mm (medium bedded, Anon 1981). Abundant mica was identified on many of the bedding planes and also in thin layers parallel to bedding. Seepage of water occurs through the discontinuities and often emerges on the dominant bedding planes. The easterly dip of the strata promotes flow of water towards the east which allows water to seep out from the discontinuities on the west side of the cutting more readily than on the east side.

5.3 Faults

Nine faults have been identified and occur at random intervals along the cutting. There are three sequences of stepped faulting and the remainder are singular. The displacements are mostly unknown but some must be at least in excess of 3m. Faults may be closed, with no infilling, or open to a maximum of 100mm, with a red-brown sandy clay or clay material. No faults were observed in the ground adjacent to the instrumented piles.

6 SLOPE STABILITY

The discontinuities identified will undoubtedly have an effect upon the strength and behaviour of the rock mass. Intersecting discontinuity sets form wedges of rock material which, if able to move, will create areas of instability within the rock mass and may influence the stability of the retaining wall. The influence of water within the rock mass must also be considered. Seepage commonly occurs on the west...
side of the cutting allowing pore water pressures to accumulate and therefore having a detrimental effect on the overall stability of the slope. On the east side however, seepage is much less evident and therefore is advantageous to the stability of this slope.

The assessment of slope stability has been carried out using stereographic projection and the data obtained during the discontinuity survey. The method used for identifying important pole concentrations is one developed by Markland, 1972. Markland's test was designed to establish the possibility of wedge failure occurring in which sliding takes place along the line of intersection of two planar discontinuities (Hock & Bray 1981). An angle of friction of 30° has been assumed for the analysis although in practice this value is likely to be higher due to the roughness of the discontinuity surfaces. Any increase in its actual value will only serve to increase the stability of the slope.

Considered first is the original railway cutting which has a north-south orientation with slopes either side of 60°. On the west side of the cutting the criteria for wedge failure to occur is not met and the slope is considered stable. On the east side of the cutting two wedges represented on the stereonet fall close to the wedge failure envelope and, therefore, are potentially unstable. However due to the orientation of these discontinuities and the slope face, plus additional criteria which must be met, wedge failure is considered unlikely to occur.

The stability of the new cutting considered next, orientated north-south with vertical sides, is considered without the support of the retaining wall. On both the east and west sides of the cutting the formation of a wedge is possible due to several intersections which, on the stereonet, plot close to or inside the wedge failure envelope. Those wedges which daylight in the outcrop will result in a small block being formed. Without the retaining wall in place these small wedges of material may form particularly on the east side of the cutting giving to a less favourable orientation. However thanks to the tendency of the seepage to drain down dip away from the east cutting face, it is considered that wedge failure is unlikely to occur on the east side.

In conclusion, cutting back the side slopes of the original cutting from a stable angle of 60° to vertical has introduced potential stability problems. Although no major instability is envisaged due to the favourable orientation of the discontinuity sets

Figure 4. Bored pile wall, with the instrumented piles, exposed during the excavation phase. The excavator is seen digging a deeper trench adjacent to the wall ready for the construction of the stabilising slab. The pile cap is braced by temporary props during the excavation and stabilising slab construction stages. The rock debris clinging to the face of the piles records the stratigraphy as seen in the geological section (Figure 3). The light grey layer is buff sandstone, white below is red-brown mudstone, both belonging to the Bromsgrove Sandstone Formation.
and the cut face, minor wedges may form. Without the protection of the retaining wall, raveling of the surface material would occur leading to the eventual degradation of the vertical slope. With the retaining wall in place and providing the wedges are mechanically free to slide, they may apply an additional force to the retaining wall. It is evident from these studies that the most realistic analysis of the ground performance is in terms of rock-like rather than soil-like behaviour.

7 CONCLUSIONS

Monitoring of the instrumentation has been carried out during the excavation phase of the work, during the construction of the stabilising slab and will continue for one to two years subsequently. The results of this work will be published separately. The engineering geological studies reported here will facilitate both the interpretation of the data and the recognition of other sites to which the data will be applicable. Without such studies the data would remain purely site specific and of uncertain application.

ACKNOWLEDGEMENTS

Funding for this research on the Coventry North-South Road is provided by the Engineering and Physical Sciences Research Council. Grateful thanks are recorded for permission and help with the site work to Messrs Bahtie, Mott McDonald and Mowlem Construction Ltd. Thanks are also recorded to Mr Nigel Vincent and our colleagues Professor W. Powrie, Dr. D. J. Richards and Mr T. Hayward.
Multiple investigation methods for fault rupture hazard evaluation

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ABSTRACT: Difficult site conditions required a unique approach to evaluating the potential for surface fault rupture in Huntington Beach, California. The evaluation was performed to determine the presence, location, age, and potential displacement magnitude of fault splays that may traverse the site. Evaluating the activity level of possible fault splays was not feasible by typical shallow trenching methods because of the thickness of the Holocene age sediments, extremely shallow groundwater conditions, and numerous aboveground structures and underground utilities at the site.

High resolution seismic reflection data was used as the reconnaissance tool to identify suspected faults at depths greater than 15 meters. Closely spaced cone penetration tests (CPT’s) were performed in the shallow alluvial deposits to verify the presence and trends of suspected faults identified in the seismic reflection surveys. Exploratory borings were drilled along the CPT lines for stratigraphic correlation, verification of CPT data, and obtaining samples for radiocarbon dating.

1 INTRODUCTION

The study was conducted to evaluate the presence, location, age, and potential displacement magnitude of fault splays of the active Newport-Inglewood fault zone (NIFZ) known to exist in the vicinity of the County Sanitation Districts of Orange County (CSDOC) Plant 2 in Huntington Beach, California, about 50 kilometers southeast of Los Angeles. Evaluating the activity level of possible fault splays by typical shallow trenching methods was not feasible because of the thickness of the Holocene age sediments, extremely shallow groundwater conditions, and numerous aboveground structures and underground utility lines at the site.

The investigation consisted of a review of available data regarding previously identified fault traces and a field investigation. The data review, which included review of historic aerial photographs and topographic maps, available published and unpublished geologic reports by government agencies and private consultants, groundwater data, and oil and water well logs was performed to establish a preliminary understanding of the possible fault locations on the property. The field investigation included a geophysical survey using high-resolution seismic reflection to collect deep subsurface information (greater than 15 meters) to locate suspected fault splays at depth. Cone Penetration Testing was also performed to verify the presence of the suspected fault traces by providing detailed stratigraphic correlation in the shallow subsurface materials across the site. Exploratory borings were drilled adjacent to the CPT lines for stratigraphic correlation and verification of CPT data. Sampling of the subsurface materials for the purpose of radiocarbon age dating was also performed in the borings.

2 SITE CONDITIONS

The CSDOC Plant 2 is located on a triangular site and is approximately 0.4 kilometer north of the Pacific Ocean and is bounded by the Santa Ana River Channel on the east, a flood control channel on the south, and Brookhurst Street on the west and north.
as shown in Figure 1. The site is underlain by alluvial deposits that have infilled the ancient Santa Ana River Channel. The alluvial deposits underlying the site consist of approximately 25 meters of Holocene age sediments consisting of thin layers of sand, silt and clay. The uppermost 3 to 6 meters of these materials were derived from a historic flood. The Holocene age alluvial deposits are unconformably underlain by Pleistocene age basal channel gravels that are approximately 12,000 to 18,000 years old.

3 GEOLOGIC SETTING

The lower Santa Ana River floodplain, known as the Santa Ana Gap, is about 3 kilometers wide in the vicinity of the site and is bordered by Newport Mesa on the southeast and Huntington Mesa on the northwest. These mesas are the geomorphic expression of marine terrace deposits laid down during regressions (receding sea level) following glacial, high stands of sea level.

The fluvial sediments underlying the site were deposited in ancestral channels of the Santa Ana River. Approximately 12,000 to 18,000 years ago, the channels were cut into Pleistocene age deposits (approximately 125,000 years old). At that time, sea level was about 105 meters lower and the coastline about 1.6 kilometers farther seaward (Sisemore, 1994). The hydraulic gradient of the ancestral river was substantially greater and extensive basal channel gravels were deposited. These gravels are locally more than 30 meters thick and are between 12,000 and 18,000 years old. The overlying fine-grained deposits are judged to range in age from 12,000 years old at the base (immediately overlying the gravel deposits) to essentially modern at the present ground surface.

Radiocarbon dating of disseminated charcoal obtained about 1 meter above the estimated 12,000-year-old top of the basal gravels, indicates a date of 10,710 ± 250 years. This date provides further evidence that the contact between the basal channel gravels and the overlying fine-grained sediments essentially marks the Pleistocene-Holocene boundary. The dating of the charcoal sample also provides a numerically dated stratigraphic marker to determine the relative activity of the Newport-Inglewood fault zone.

4 EXPLORATIONS

4.1 Geophysical Survey

The geophysical survey was performed using high-resolution seismic reflection lines to identify faults in the subsurface at depths greater than 15 meters. Geophysical data was collected along three survey lines totaling 2,100 meters. Geophones were generally spaced at 3-meter intervals.

Seismic Reflection Line 1 was located along the western boundary of the site. This line “shadowed” the existing facilities at the site to determine if faults might underlie these structures. However, the data produced during this survey was ambiguous in a few areas with regard to identification of suspected faults because of vibrations produced from the plant operations and on-going construction. To provide better quality data in these areas, two additional seismic reflection lines were added. Seismic Reflection Lines 2 and 3 were performed along the southern boundary of the site and along the Santa Ana River levee, respectively. These sections were planned to roughly shadow the triangular perimeter of the property’s boundary as shown in Figure 1.

4.2 Cone Penetration Tests

The field investigation also included several lines of CPT’s to verify the presence of suspected faults identified in the geophysical survey. A total of 106 CPT’s were performed in 8 lines to provide detailed stratigraphic correlations and locate the upper contact of the basal gravel unit. Detailed stratigraphic analysis of the CPT data was based on identification of sediment layers that were very well-defined and consistent between the CPT’s. These included fairly continuous clay layers that were easily identified and could be traced between CPT’s. Correlations between identified sediment layers was possible within about 15 centimeter vertical increments. The locations of the CPT lines are shown in Figure 1. Figure 2 is a representative CPT section that illustrates the detailed stratigraphic correlation that was possible between CPT’s.

4.3 Exploratory Borings

In addition to the geophysical survey and the CPT’s, three exploratory borings were drilled using 12.7-centimeter-diameter rotary vash-type drilling
equipment to a depth of approximately 39 meters beneath the existing ground surface. The soil and groundwater conditions encountered were logged and undisturbed bulk samples were obtained for the purpose of stratigraphic correlation, verification of CPT data, and possible radiocarbon dating. Only one sample (obtained at a depth of about 23 meters beneath the existing ground surface) contained enough organic material to allow radiocarbon dating analysis.

5 FINDINGS

5.1 Geophysical Survey

Fourteen suspected faults of varying activity levels were identified on the seismic reflection sections. These faults are shown in Figure 1. The faults were identified by reflective layers that are truncated and folded. Stratigraphic layers are known to terminate for reasons other than fault truncation. Thus, identified faults were chosen not only because of truncation of a single layer, but truncation of multiple, stacked layers. The presence of folding, which is common along faults, was also used as supporting evidence to prove the existence of faulting.

The seismic reflection surveys did not clearly show the top of the basal gravel unit. Whether faults cut the upper contact of the basal gravel unit had to be verified from the CPT sections.

5.2 Cone Penetration Tests And Exploratory Boarings

Closely spaced CPTs (typically 12 meters but as little as 3 meters apart) were performed to verify the presence of suspected fault traces identified in the seismic reflection lines by providing detailed stratigraphic correlations in the Holocene age alluvial deposits across the site (approximately the upper 23 to 29 meters of sediments). Detailed stratigraphic analysis of the CPT data was based on identification of soil layers that were vertically well defined and laterally consistent between CPTs.

As identified in the CPTs and the borings, the alluvial sediments beneath the site generally tend to be thin layers of distinct soil zones. Fine-grained soil layers (clays and silty clays) identified in the CPTs tend to be laterally continuous over long distances (over 90 meters). Therefore, the stratigraphic correlations between CPTs were based mainly on groups of sediments with similar cone bearing (Qc), sleeve friction (Fv), and friction ratio (Rr) signatures.

The materials encountered in the borings generally correlated with the materials encountered in the adjacent CPTs. For instance, clay layers visually identified in the borings, both laterally and vertically, correlated with clay layers identified in the CPTs, characterized by low cone bearing and high friction ratio measurements. Correlations between the CPTs and the borings were used to determine the top of the basal gravel unit.

The top of the basal gravel unit was identified in the CPTs by a significant increase in cone bearing measurements. The depth of the increase in cone bearing measurements in the CPTs correlated with the depth at which the basal gravel unit was visually identified during drilling of the exploratory borings. The top of the basal gravel unit is identified at depths ranging between 23 and 29 meters beneath the existing ground surface and is relatively level to slightly undulatory in nature.

Five faults were identified on the CPT correlation sections that offset the top of the basal gravel unit and the overlying alluvial sediments. Apparent vertical offset of the top of the basal gravel unit along these faults ranged from 1.2 meters to 2.7 meters. These faults are considered to have a high activity level and are shown in Figure 1. The other nine faults identified on the seismic reflection sections were not observed in the CPT sections. These faults did not offset the contact between the Pleistocene age basal gravel unit and the overlying Holocene age sediments but are confined to the Pleistocene age sediments. These faults are considered to be of moderate to low activity levels.

Where logistically feasible, the trend of the faulting was determined from parallel CPT sections. The northwestern trend of faulting determined from the CPT sections correlated well with the regional trend of the NNZ and the fault trends documented by previous geologic reports regarding faulting beneath the site.

6 CONCLUSIONS

The use of multiple data sources has provided a means to identify faults that traverse the site and evaluate the potential for surface fault rupture and fault rupture magnitude. The combined data
sources provided detailed information about both the shallow and deep subsurface conditions at the site. Furthermore, the subsurface information, in conjunction with radiocarbon age-dating, provided the necessary data to determine the approximate age of the faulted sediments and the activity level of identified faults.

The interpretation of geophysical data identified fourteen suspected faults at depth within the Pleistocene age materials. The correlation of the closely spaced CPTs and the borings showed a detailed section of the shallow subsurface stratigraphy and suspected faults at depth could be verified in the shallow subsurface materials. Due to the close spacing of the CPTs (typically 12 meters but not less than 3 meters), lateral correlation of distinct soil zones was possible over long distances. Offset of distinct soil zones could be distinguished within 15-centimeter vertical increments. Based on detailed CPT correlations, no folding of the Holocene sediments was observed. However, five faults could be identified that offset the top of the basal gravel unit and the overlying alluvial sediments. These faults are considered to have a high activity level since they offset Holocene age sediments.

REFERENCES

Stilson, 1994, "Late Quaternary Stratigraphic and Neotectonic Framework, Wastewater Treatment Plant 2, Huntington Beach, California."
A knowledge-based system approach to soil property determination

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ABSTRACT: In view of the difficulties and complexities involved in soil exploration planning to characterise soil properties of substrata, a knowledge-based system approach is proposed in conjunction with the computer system, SEPS (Soil Exploration Planning System), to assist in the exploration and testing process. The programming is based on an object-oriented approach using C++ language. The major elements of the system consist of a geotechnical test knowledge base and mechanisms that provide retrieval, reasoning using a simple weighting system, and, in particular, applications of the stored statistical knowledge using lower level probability techniques. These mechanisms are primarily intended to provide viable alternatives to the present largely subjective methods for designing soil exploration procedures and to assess the implications as the work proceeds. In this paper, the main features of SEPS are discussed and the implementation of the system for planning soil exploration programmes for undrained shear strength determination is demonstrated.

1 INTRODUCTION

Among various components associated with successful geotechnical design, there is little doubt that a proper understanding of ground conditions is of prime importance. In particular, a knowledge of the engineering characteristics of soils is essential for all geotechnical analyses. In contrast to the material associated with most engineering problems, few natural soil deposits are even approximately uniform and many are extremely variable because of the way they are formed and the natural environmental processes continuously acting on them. The procedures suggested by experienced practitioners (e.g. Hvorslev 1949, Peck et al. 1974, Welman & Head 1983) and in standards (e.g. BS 5930 1981, Geoside 2 1987) for the planning of exploration in subsurface soils are largely qualitative and based on engineering experience. This is mainly because of the fact that the extent and methods of soil exploration depend not only on the nature of the project but to a large degree on the soil conditions. Unfortunately, in some instances, exploration is inadequate and there are difficulties in the interpretation of observations and measurements. These cases may be attributed to the lack of proper accounting for the inherent natural variability of geotechnical materials, the uncertainty of measurement with improper control of equipment and operating procedures. In practice, results of the widely used in-situ tests are commonly interpreted empirically by comparing them with other means of measurement to determine the soil property value. This introduces the complication in the interpretation of uncertainty associated with the conversion of test measurements to design soil parameters because of the inadequacies of empirical correlations.

In view of these difficulties, it would appear to be valuable to devise a soil investigation procedure that takes a more rational quantitative approach; one that would deal with the uncertainties and variabilities involved in all geotechnical information. Probabilistic methods provide a formal basis for assessing the uncertainty in the information derived from a particular exploration programme and in doing so permit realistic optimisation of the levels of uncertainty of a soil property associated with different testing plans, and their costs. However, to avoid substantial computational effort, there is a need to have a computational facility that provides for a systematic approach to the required type of data analysis. The facility should have a substantial knowledge base consisting of a range of test methods and their merits based on the different soil
types and information required. It should also provide statistical data for the application of probabilistic methods in the process of soil exploration.

2 DEVELOPMENT OF SEPS

2.1 General

Use of an object-oriented approach implemented in C++ facilitates such a development and permits a flexible approach while providing a means for structuring the knowledge base to reflect the nature of the test knowledge. This approach is used to develop the computer system, SEPS (Soil Exploration Planning System). The fundamental data for the knowledge base is developed using a statistical analysis of data from a wide range of sites. It is discussed in detail by Fung (1997) and, specifically, addresses some proliferation test data in a companion paper by Fung & Kay (1998). The major elements of the system consist of a geotechnical test knowledge base and mechanisms that provide retrieval, reasoning using a simple weighting system, and, in particular, applications of the stored knowledge using lower-level probability techniques. These mechanisms are primarily intended to provide viable alternatives for designing the soil exploration programme and to assess its implications as the work proceeds.

A particular feature of the system is the incorporation of a help system including an information component that provides the backgrounds and applications for different testing techniques. The information is presented through a variety of media as illustrated in the following.

2.2 Multimedia Materials for Learning

The information component is developed using the Microsoft Windows Help compiler, which is an authoring tool for developing hypermedia applications that allow the user to control the navigation scheme for presentation of the materials. For illustration, Figure 1 shows an on-screen view of a movie clip that demonstrates the calibration of the dilatometer test while Figure 2 presents information about the background of the test.

The major components of SEPS are discussed in the next section and this is followed by an example problem to illustrate the implementation of the system for planning soil exploration programmes for undrained shear strength determination.

3 MAJOR COMPONENTS OF SEPS

3.1 Decision Mechanism Dealing with Quantitative Information

A probabilistic approach for quantifying complexities and uncertainties involved in the work has been developed (Kay et al. 1991, Fung 1997) and this is incorporated into the knowledge-based system as a decision mechanism.

In relation to studying the implication of new information on the initial estimates of the ground at a site and on planning of subsequent exploration, the approach provides for a formal procedure for systematic updating of the variability components through the application of Bayes' theorem. Accordingly, the information that is stored in the knowledge base will become more representative and the influence of the less desirable subjectivity involved in the literature-based estimates will be reduced.

Figure 1. An on-screen view of a movie clip showing calibration of dilatometer blade

Figure 2. An on-screen view of information about background of dilatometer test
3.2 Decision Mechanism Dealing with Qualitative Information

In order to provide an overall view of the viability of different testing methods, a simple weighting system is developed that can produce a ranked list based on the given information about the site, its ground conditions and the project details. Weighting techniques have been found to be useful for dealing with qualitative information or advice that is frequently encountered in geotechnical engineering. Mechanisms developed on the basis of simple weighting techniques have also been adapted to knowledge-based systems for geotechnical engineering problems, such as for selection of groundwater control methods (Davey-Wilson & May 1989) and for selection of pile foundations (Weng 1990). The given information may be processed with the assistance of the stored test knowledge base that contains two important kinds of information, the applicability of tests to different soil conditions and their suitability for acquiring certain geotechnical properties.

In relation to the information on the applicability of tests in different soil types and for determination of a variety of engineering soil properties, there are several ways to assist in its collection. First, valuable opinions suggested by experienced researchers and practitioners can be found in the literature (e.g., Campanella & Robertson 1983, Wroth 1984, Robertson 1986, Robertson et al. 1996). These are useful for providing guidelines for the applicability and usefulness of a test. However, because the purpose of opinions of this kind is for guidance only, these tend to be general. This information is supplemented by results from some comprehensive reviews by Orchant et al. (1988) and Kulfanwy & Mayne (1990). Results from the latter studies are more specific in nature and are particularly valuable for comparisons with the information obtained from the former kind of source. The qualitative information stored in the knowledge base is derived from information obtained from these sources.

In addition, cost data on the various tests under various conditions are incorporated. Although based on published data (Kulfanwy & Mayne 1990), they depend on time and location. They should be modified to suit local conditions.

4 EXAMPLE PROBLEM FOR PLANNING SOIL EXPLORATION PROGRAMMES

To illustrate potential applications of the system to soil exploration planning, use is made of a test knowledge base comprised of eight widely used in-situ soil tests as well as one commonly used laboratory test. For illustration, a project involving the design of pile foundations at a site in a twenty-metre-thick stiff silty clay layer is considered. On the basis of a preliminary assessment, piles with a length of about ten metres are tentatively determined to be adequate for the project. To perform detailed analyses for the design, it is desired to obtain both the undrained shear strength and the constrained modulus of the relevant soil layer.

4.1 General Test Suitability

Figure 3 shows the results including the rank and score for suitable tests using the weighting system using the “qualitative reasoning” approach. Four in-situ tests are found to be useful for the determination and, of these, both the dilometer test and the piezocone test are preferable. However, this is only useful for indicating the overall suitability of the tests for the determination.

More specific guidance can be obtained by initiating probabilistic and cost analyses using the “quantitative approach”.

4.2 Specific Test Evaluation

Figure 4 shows the window for the specification of the testing plans for the four in-situ tests for such analyses. It should be noted that the selected penetration test is considered to correspond with the average from two meters of penetration.

With the assistance of the test costs and the suggested variability components of coefficient of variation for the combination of test type and soil type stored in the knowledge base, the anticipated uncertainty levels of the soil parameter for different testing plans with total cost estimates can be determined.

A summary of results is given in Figure 5 for an analysis that examines each of the alternative plans for the determination of undrained shear strength for the soil at the four locations over the site. Both the piezocone test and the electric cone penetration test appear to be appropriate for achieving the purpose in
Figure 3. Ranked testing techniques in qualitative reasoning window

Figure 4. Specification of testing plans for four in-situ tests
Figure 5. Feasibility report recommending feasible tests based on ranking system, probabilistic and cost analyses.

Figure 6. Bayesian updating for uncertainty component associated with natural soil based on piezocone test results.
this case with the piezocene slightly more preferable. Details of methods used for computations are given in Fung & Kay (1996) and Fung (1997). It should be noted that the specific quantitative analysis is only available for undrained shear strength at the present time.

4.4 Uncertainty Updating

Upon completing the adopted testing programme using the piezocene test, the test results obtained can be used to update the initially assumed uncertainty value for the soil using the piezocene test provided the data number is sufficient. Figure 6 presents the results of the original and updated uncertainty components for the soil using the test. The updated parameter shows less variability in this case but, in some cases, it may be greater.

5 CONCLUSIONS

Applications of a system for design of soil testing programmes for soil property evaluation have been demonstrated. Two decision mechanisms are relied on to provide the assessment. A mechanism that uses simple weighting techniques is included for dealing with advice or opinions of a qualitative type for providing an overall assessment. To address the uncertainties involved in the process of soil exploration, the decision mechanism makes use of lower level probability techniques to formally account for different kinds of uncertainty and variability in soil exploration. The application of Bayes' theorem to study the implication of the newer information on the existing information stored in the knowledge base is included. To assist in the learning process and for providing information about the backgrounds and applications of different testing techniques, a help facility is incorporated that takes advantage of different presentation media. Such a knowledge-based system can provide for a systematic approach to data analysis required for soil exploration planning, which, at the present time, is addressed using more generalised subjective judgement.

REFERENCES


ABSTRACT: This paper presents characteristics of the unstable territory in one of the regions of the Dniepropetrovsk city (Ukraine). It describes deformations of soil surfaces, buildings and structures. Contents of the survey and monitoring programme, that have been applied at the said site, is given. Schemes of structures and facilities for stabilization of the unstable territory have been reported.

1 INTRODUCTION

It is quite an often practice when full set of required works aimed at receiving detailed characterization of a site meant for future construction is not fulfilled before starting development of any new territory.

Such characteristics should include materials of engineering and geological investigations and results of engineering and geodetic survey, forecast of ground water table rise, diagnosis of possible collapsing and landsliding effects, analysis of aftereffect of oversaturation of ground with water under buildings, calculations of hypothetical soil collapsing and building deformations, etc. On the basis of such investigations it is necessary to give recommendations as to schemes for performing building bases and protection arrangements against rise of ground water on the territory, antilandslide facilities (on sloping areas), etc. In case such general characteristic of construction site is not fulfilled at all or carried out without sufficient volume of engineering and geological survey and soil investigations, then, after certain time period of developed territory exploitation troubles such as ground surface collapsing and deformation of buildings will inevitably occur. And, sometimes, lack of required preliminary investigations may cause serious accidents.

Situation of the kind emerged, for instance, in one residential area named “Topol” in the Dniepropetrovsk city (Ukraine). Owing to insufficient volume of survey and investigations to be fulfilled in due time, engineering protection structures against collapsing and landslides effects were not designed and constructed on this territory. Buildings constructed on loess-type soils started to deform after ground water rise. Besides, slopes enclosing this territory become unstable due to oversaturation of soils with water. As the result of the above, catastrophic landslide occurred in this residential area causing numerous destruction and human victims. There are quite a lot of such extremely hazard regions in Dniepropetrovsk. This article presents description of the accident that took place in one of the dangerous regions, situation in which we can estimate as of the utmost emergency.

2 GENERAL CHARACTERISTICS OF THE SITE

The territory under consideration is located on a slope of one of the hills on which Dniepropetrovsk city is situated. This city region is framed by the following streets: Kirov avenue, Titov street, Yavdenchuk street (in fact, it is a slope of Rybalskaya ravine). Total area of this zone (fig. 1) is more than 50 hectares. Many socially significant...
objects are located there: dwelling houses, schools, hospitals, boiler installations for heating of the region, etc. All the above territory has been in a very bad state for a long time. In the course of more than ten years deformations of all the vast territory are taking place: buildings are destroyed, soil falls through and cracks appear and landslides movements are observed, underground and overland communications are broken.

The territory in question adjoins to the deep Ryabikhaya ravine on the slope of which galleries are formed, vegetation layer is damaged, the caves similar to karst formations emerge.

In the course of several years the studies were performed and measures were developed for the most accident-dangerous buildings and structures. However, these actions and facilities were not effective, mainly due to the fact, that the objects being grouted were considered in isolation from general soil situation on the whole territory of the emergency area. Actually, reasons for long-term soil deformations in the region were not established earlier.

The state of the territory considered was becoming more dangerous with every passing year. Due to serious deformations tenants of several nine-storey houses were settled out, two schools terminated their functioning; in parallel to that emergency situations in one-storey dwelling houses were eliminated by means of urgent but temporary measures. Besides the above mentioned, communication lines supplying the region with water, gas and heat were damaged.

It should be noted that at the time being, additionally to destruction of buildings and structures, dangerous soil situation is, in general, harmfully affecting the environment as a whole. Area and soils have been contaminated by sewage leakage, swamped places have emerged, trees died up and fell down, land surface, slightly inclined towards the ravine, has been covered by unstable zones, troughs and small pools of stagnant water.

Major number of buildings located on this territory is one- or two-storey buildings, one-storey dwelling houses, two-three-storey schools and multi-storey buildings. Two nine-storey houses have been subjected to the largest deformations (Fig.1): nine-storey dwelling house in Nakhtinov street, 90 and fourteen-storey dwelling house in Gavrilenko street, 10.

3 SEARCH OF DEFORMATION CAUSES

Analysis of deformation causes of the territory and buildings and structures that are situated on it was complicated by large diversity of destruction and damages that took place.

For instance, one dwelling house consisting of three nine-storey blocks, situated in Nakhtinov street, 90, has been constructed in parallel to the ravine edge and has got sagging cracks lengthwise. Such cracks are typical demonstration of sagging differences in various parts of the building. This proved the fact that collapsing of loess-type soils underlying foundations made of reinforced concrete plate on soil cushion (without applying piles) was the reason for deformations. Initially it was decided that the main reason for deformations is loess-type soils collapsing due to their wetting. However, later offsets and inclines of the houses in Nakhtinov street, 90, towards...
the ravine occurred. At that time suspicion about
effects of landslide shifts of the soil slope started to
arise.

It is worth noting that soil mass under these both
buildings for the depth almost 30-40 m comprises
loess-type and sandy loam that very often possessing
collapsing properties. Eolian and deluvial deposits of
loess complex of Quaternary period rocks participate
in these soils. These deposits are represented by the
interlayers of loess loam and loess sand loam. The
underlayer of the whole thickness of loess-type soils
is the top aeogenin red-brown and grayish-brown
clays with random layers of loam.

Ground water was at the 20-25 m and more depth
for the period of 10-15 years. At present ground
water table reached and measured at the 5-6 m depth,
and at some areas even closer to the soil surface. And
the territory is now at the stage of ground water rise
with an average speed of level increase reaching 0.6 -
0.8 m per year and more.

It should be stated that there were quite weak
interlayers that demonstrated tendency to be forced
out of the soil mass towards the ravine at the thalweg
of which ground water decrement took place. The
above happened at the process of ground water table
rise and water saturation of loess soils through their
thickness. These weak soil interlayers, presumably,
are pressed out into failed slope zone of the ravine
being under weight of overlaying masses as well as
removed by ground water flow if the form of
suffusion. This phenomenon, probably, adds to
collapsing intensity of the whole territory. Owing to
the existence of many factors described above such as
collapsing, landslide, suffusion, - all taken measures
and facilities for the purpose of underpinning of
dwelling house with the address Nakhimov street, 90,
gave no positive result.

Works carried out for stabilization of deformations
of this house were as follows: crushing of metallic
piles through ground slab erected on soil cushion;
crushing of reinforced concrete pile sections;
chemical grouting of base soils (this is a sufficication
or electro-sufficication); lowering of ground water
table by means of radial and trench drains; arranging
of on-land water ways; repair of water supply lines to
reduce leakage into soil and decrease loess-type soil
weakening; etc. Nevertheless, all applied control
methods, that were performed at high costs, did not
bring to a desirable effect.

The result was that all tenants from all three blocks
of nine-storey house were settled out. Vertical
deformations, inclines and cracks of the building
considerably exceed standard values. At the time
being the problem of dismantling or blowing up this
comfortable building of large length is in question.

Since, at present time fourteen-storey house in
Gaviolasko street, 10 also manifests deformations
much exceeding acceptable norms and all tenants are
settled out from it, problem of stabilization of its
foundation collapsing is extremely urgent. Complex
of measures that are planned to rescue the building
will be described in the next section of this article.

Taking into account that there are some damages
in all the buildings and structures on the said
territory, the decision to examine in detail all this
hazard area has been taken.

Extensive complex of engineering and geological
survey and soil investigation has been developed with
the aim to establish factors causing deformations on
this territory. This complex includes: visual
examination of all available deformations in structures
and soils; engineering and geodetic survey and
observations of installed marks and beacons on
buildings and soils; geological survey comprising
boring of holes to the depth of stable rocks, as well as
soil laboratory tests and ones on location;
construction of the geological cross-section from the
upper border of the territory towards the ravine
thalweg together with marking physical and
mechanical and strength characteristics for all types
of rocks; boring pits for buildings having
deformations with the aim to take soil samples from
under foundations; arrangement of network of
observation hydro-geological wells for determination
of ground water table and dynamics. performance of
long-term monitoring of the territory and buildings
situated on it; calculations of the ravine slopes
stability value and values of possible foundations'
collapse; etc. It is worth while noting that owing to
the fact problems are really complicated on this
unstable territory, we had to perform series of special
investigations (geophysics, observation from space
satellites, aerophotography, infra-red imaging survey
and etc.).

It should be stated that by the time being the
programme of surveys and investigations fulfilled is
far from what that has been targeted. Nevertheless,
analysis is being made on the basis of materials
available now.

On the basis of the investigations and surveys part
of which has been already fulfilled by present time, it
is possible to conclude that soil deformations and
buildings’ damages on the territory under
consideration are caused by a number of factors:
loess-type soil collapsing as a result of their high
water content; soil strength properties reduction
under bases due to presence of water under foundations of buildings; landslide events at the ravine slopes; mistakes made while designing and constructing foundations of the buildings and structures; low quality of soil cushions made, that in major cases were foundation base; insufficient consideration of geotechnical peculiarities of construction on collapsing loess-type soil mass of large thickness (additionally to the fact that unstable soil slopes adjacent it); impossibility to prevent rise of ground water table and eliminate leakage out of water supply communications on the developed territory; etc.

As a result, a set of alternative engineering methods for the territory protection is being developed now. These methods are: arrangement of special drain systems; different measures aimed at water level lowering on the major part of the territory; production of anti-landslide facilities of various types; executing of underpinning and strengthening of bases under the most deformed buildings; other geotechnical actions.

It should be noted that owing to the lack of financial resources in Ukraine, all above enumerated engineering protection measures are performed extremely slowly.

4 FACILITIES BEING IMPLEMENTED FOR ENGINEERING PROTECTION

Protection structures and measures on this territory fulfilled for stabilization of the whole area, as it was said before, are being implemented very slowly. Therefore, measures for saving concrete buildings that are in the state of extreme emergency are taken most promptly.

One of such buildings is a fourteen-storey dwelling house in Gvarščenko street, 10.

This house (fig. 2) was constructed on piles foundation with piles of 20 m length and cross-section 35 x 35 cm. Practically, it had no deformations during the first 6-7 years. The layout of the house is as follows: 30 m length (this side is parallel to the edge of the ravine slope), 12 m width. It is situated closely to the ravine.

Deformations of the building began to appear 2-3 year ago (starting around from the 8th year of exploitation). Values of deformation are approximate because geodetic measurements have been carried out only after deviations became obvious visually. At present total collapse of foundations makes up tentatively 400 - 450 mm (and it is not uniform at the corners of the building), and the incline of the house towards the ravine is about 650 mm (deviation of the upper point from the vertical). These deformations are increasing with a rate 1 mm per day.

The building made of brick was constructed on the strip piles foundation with reinforced concrete belts after every two storeys. Owing to stiffness of the building, cracks in its framework are quite small. However, considerably larger cracks appear in soil on the upper side of the building and rocks bulge out on its bottom side. Blinds area under the whole foundation of the building has been separated and collapsed considerably.

Upon analyzing all available materials and results of additional investigations (in particular, soil surface marks/shafts on the upper side of the building) it has been established that the most probable cause of deformations of this dwelling house is lack of piles foundation resisting strength against horizontal landslide pressure of the upper part of the slope. The point is that while designing piles foundation it was presumed that landslide break off was possible only at the place lower to the building. However, due to ground water table rise the upper part of the slope has also become unstable (see fig. 1). And the piles foundation was not designed to withstand the force of such landslide pressure.

It should be stressed that the above mentioned reason is not the only one causing foundation deformations. According to the design, bottom ends of piles did not reach the layer of tight clays (fig. 2). Probably, negative friction hanging over piles added to extra wetting of loess-type soils. This, together with additional moment of landslide pressure, affecting piles foundation, increased loading upon piles. And it is natural that the piles situated close to the ravine's edge (at the lower side of the building) were the most loaded. This caused the incline of the building towards the ravine.

It is worth saying that tests performed by means of static loading of load-bearing piles (directly under the building) demonstrated their insufficient load-carrying capacity.

Probably, there was low quality of production while executing monolithic consisting of pile heads with reinforced concrete plate of the grillage and poor quality of piles' joints that were composed of two sections.
In view of the above it was decided to apply such strengthening measures that could eliminate these full set of reasons that induced growing of building's deformations progressively worse.

It was designed to produce protective enclosure of vertical casing of bored cast-in-situ piles, having 1.0 m diameter, around the existing foundation of the building. This casing was designed for cutting through full thickness of loess soils and the bottom part of these piles should be embedded into tight neogene clays. It was designed to put 3 rows of such piles at the upper side from the building and this allowed to take landslide pressure. Only one row of piles was considered enough to accept surplus of vertical loads and to overlap defects of pile arrangement and joining in the existing piles foundation. Reinforced concrete grillages the old foundation have been reliably connected with the upper reinforced concrete plate contouring upper retaining vertical casing of bored cast-in-situ piles. So, the old foundation was securely suspended by the new pile casing (reinforced concrete vertical 'skirt' having a rectangular cross-section and made of bored and cast-in-situ piles). Some drawback of load-carrying capacity of such 'skirt' as to vertical loading was compensated by manufacturing bored and injection piles that were produced inside the building out of technical basement. These bored and injection piles have also allowed to solve the problem of reducing spans between reinforced concrete beams joining pile heads of the old and new foundations creating one monolithic grillage.

It should be said that prior to making monolithic unit, it was necessary to cut off head of some piles in the old foundation. In such a way it became possible to eliminate incline of existing fourteen-storey building. Unfortunately, for the lack of space in this article, it is not possible to describe in detail technology of correcting building's incline.

Besides the described structures and facilities, other actions aimed at the increase of total stability of the slope adjoining the building were taken. For instance, alongside the slope (from the top to the bottom) so-called buttress drains were arranged. Each such drain was a trench made by means of "wall-in-soil" method (under clay solution). This trench had 800-1600 mm width and 18-20 m in depth in the terms of deepening longer than depth of sliding.
surface of landslide and filling in with drainage material. Such buttress drain is important in two ways: it removes water from adjacent soils herewith decreasing ground water level and increasing slope stability by changing weak soils with hard rocks (crushed rocks, stone, etc.) at the level of the sliding surface.

It should be said that much attention is paid to preventing over-saturation of soils with water on the concerned territory: supply pipe-lines are being repaired and changed; rain run-off systems with accompanying drains are arranged; various water level lowering systems are performed; and so on. However, usually, these prevention measures against rise of ground water table are not quite successful due to numerous reasons of soil over-saturation with water at the developed territories.

5 REASONS OF GROUND WATER TABLE RISE

For the lack of space, we will try only to enumerate the reasons owing to which rise of ground water table occurs at each new developed territory in the course of time.

- Disturbance of the balance between atmospheric precipitation and its evaporation after paving territory with asphalt and constructing various buildings and structures.
- Vapour condensation at the zone of aeration owing to difficulty of processes heat transfer at the regions with dense construction.
- Breakdown of ways of water filtering because of constructed buildings and their underground structures, hampering of natural water flow towards the zone of decrement by man-made obstacles in soil.
- Underground water overpressure due to construction of embankments and hydro-technical structures on rivers.
- Filling of ravines and development of their slopes without building sufficient water removing facilities.
- Decrease of transpiration (moisture sucking in by plants) at the built territories.
- Effect of main water ways made closely to build territories, that in the course of time inevitably become sources for refilling of ground water.
- Intensified additional water seepage into soil caused by leakage from supply lines of potable, service, industrial and process water; this waste occurs near hydro-technical processes as well as in the form of intensive leakage out of long water supply lines (hot water, cold water, sewerage).

- Insufficient pipe capacity of shower receivers and shower run-off collectors; violation of rules of surface run-off due to imperfections of vertical layout.
- Accumulation of drainage and thawing water, discharged from roofs into under-consolidated pockets of foundations.
- Presence of soils with low filtering capacities (for instance, loess-type one) with clay interlayers having low permeability and thus prohibiting lateral withdrawal of ground water and its accumulation in the soil thickness close to the surface.
- Lack of required single drainage system at the newly built territories.
- Effect of powerful irrigation systems.

The above given long list of factors has conditioned the fact that in Dnipropetrovsk the level of ground water increased by 30-35 m in the course of recent 20-25 years.

It is important that in many cases underground water is really the main reason for territory non-stability. In the region that is under consideration, as well as in many other areas in the Dnipropetrovsk city (by the way, and in a number of Ukrainian cities) water has become the major cause for collapsing and landsliding phenomena. However, it is hardly feasible to remove all reasons owing to which soil water content increases. Therefore, trying to prevent ground water table rise, while making extension of built territory, major attention should be given to development of measures and facilities that could allow to neutralize negative effect of ground water on safety of buildings and structures.

6 CONCLUSION

It becomes clear from the above given description of all the troubles which happened in the process of exploitation of the developed territory, that it is very important to forecast all possible geological processes at the site allocated for building prior to the start of design and construction. It is necessary to perform such preliminary investigation and give such characterization of site or territory that make it feasible to accept at least some major recommendations as to engineering protection of developed territory against hazard geological processes.

In case such characterization of the construction site is not fulfilled, it will inevitably
become necessary to spend much more financial funds to eliminate deformations, collapses, landslides, etc.

Residential areas in Dnipropetrovsk may be a very vivid example of actual results of such insufficient engineering preparation of construction. It should be noted that there are many such dangerous regions as to geological problems. They are: residential areas "Topol" and "Sokol", Kosmicheskaya street, territory of Kirov-Titov-Rybalkskaya-Vakulenchuk, Dimitrova street and many others. In all these regions, troubles are caused by ground water, collapsing soils and landslide events. It is not rare that these geological processes result in catastrophic accidents.

And it is very important that Ukraine lacks own financial recourses for liquidation of such emergency situations. It is high time to seek financing in various funds (ecological, charity, centralised world and European, Soros, etc.). Should there be no real assistance, consequences may be the most unfortunate.
Geotechnical cross-sections re-visited

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ABSTRACT: Geotechnical cross-sections are among the most useful tools available for developing and portraying the geotechnical framework of a site and portraying and analyzing a geotechnical problem. Despite this importance, both the quantity and quality of geotechnical cross-sections have declined in recent years. This is attributed to lack of experience in graphical communications among current practitioners plus increased use of and reliance on computers for drafting, analysis, and design. Practical guidance on production of geotechnical cross-sections is given herein along with several example cross-sections.

1 INTRODUCTION

The geotechnical framework of a site consists of those key elements of site geology and project geometry which characterize a site for a particular application (Hanel 1997). Geotechnical cross-sections are among the most useful tools available for developing and portraying the geotechnical framework of a site and for portraying and analyzing a geotechnical problem.

Over the past decade, I have observed a general decrease in both the quantity and quality of geotechnical cross-sections for projects reviewed in the United States and elsewhere. Some major and complex projects have had only one or two geotechnical cross-sections or even none at all. The quality of geotechnical cross-sections has declined dramatically from that of, e.g., two decades ago.

This decrease in the quantity and quality of geotechnical cross-sections has occurred on projects in all areas of civil engineering but it has been most prevalent on projects in geoenvironmental engineering where greater emphasis is often placed on characterization of waste materials, geosynthetics, and, in certain cases, existing contamination, rather than on characterization of the site itself. Some colleagues, particularly those in geoenvironmental engineering, say that, because of competitive pressures, they cannot afford to draw more than one or two cross-sections for a project. I typically respond that, on most projects, they cannot afford to draw only one or two cross-sections because it is generally impossible to determine the most critical or representative cross-section(s) until several in different directions are drawn and examined.

Decreased emphasis on geotechnical cross-sections in geoenvironmental as well as more traditional areas of geotechnical engineering appears to result from two main factors. First, many of the current generation of geotechnical and geoenvironmental practitioners have had little, if any, experience in the pre-computer age of engineering when heavy emphasis was placed on surveying, drafting, and hand-drawn graphical communications which included those of an interpretive nature such as geotechnical sketches and geotechnical plan and cross-section drawings. Second, the increased use of and reliance on computers in civil, geotechnical, and geoenvironmental drafting, analysis, and design has short-circuited the critical contemplative and interpretive processes which formerly occurred when these tasks were accomplished at a speed substantially slower than that of light and electricity.

Because of these two factors, along with the importance and utility of geotechnical cross-sections in developing and portraying the geotechnical framework of sites and problems, it is appropriate to re-visit these cross-sections here. This review of geotechnical cross-sections is
appropriate for the inexperienced and/or uninitiated as well as those who may have forgotten the significance of a basic tool of our profession.

2 CROSS-SECTIONS DURING SITE EXPLORATION

Cross-sections should be plotted in the field by inspection personnel during site exploration. Field cross-sections are most easily plotted on 8-1/2 x 11 or 11 x 17 in. (210 x 293 or 420 x 590 mm) grid paper on clipboards, though larger sheets can be used on tables in field offices or motels. These cross-sections should be drawn with equal horizontal and vertical scales where practical. Sometimes however, exaggerated vertical scales must be used so that field cross-sections fit conveniently on the smaller sheets used on clipboards. Field cross-sections should be drawn in pencil so that additions and revisions can be made as more information becomes available.

The purpose of these field cross-sections is to optimize information obtained during site exploration. After ground surface profiles are drawn and relevant surface features (e.g., soil and rock exposures, wet areas, springs, streams, fills, waste piles, slope failure features) are plotted, the locations and elevations of borings and test excavations should be added as these explorations are completed. Subsurface materials and groundwater conditions encountered in these explorations should be plotted and correlated so that subsurface strata and zones are defined, in at least a preliminary manner, during the course of exploration. These field cross-sections will help identify data gaps in the exploration program and help ensure that boring or test excavation locations and extents in plans and elevation are sufficient to provide the coverage, overlap, and redundancy necessary for development of the geotechnical framework of the site.

Where in situ tests are to be done and/or borehole instrumentation is to be installed, field cross-sections provide guidance relative to establishment of in situ test or instrumentation locations, strategies of testing or instrumentation, and interpretation of in situ test data and initial instrumentation readings. For example, field cross-sections are invaluable in establishing sand zones and tip elevations for piezometers.

3 CROSS-SECTIONS FOR SITE CHARACTERIZATION AND ANALYSIS

Some colleagues advocate development of geotechnical cross-sections in the office by technicians using computer aided drafting (CAD) systems. I do not believe this is the way to develop good geotechnical cross-sections because (1) the computer screen is generally too small for adequate visualization of relevant information, details, and relationships; (2) the technician seldom appreciates the details and nuances of site conditions disclosed by the exploration program and relevant to the project; and (3) electronic data processing typically proceeds too rapidly for the contemplation inherent in the production of good geotechnical cross-sections, particularly with complex subsurface conditions.

Much of the following material on development of geotechnical cross-sections is adapted from Hamel (1997).

I recommend that geotechnical cross-sections be drawn by hand on large sheets of grid paper using soft lead pencils and appropriate erasers. Cross-sections should be drawn by the (presumably experienced) professionals who directed (or inspected) the exploration program. These professionals should be familiar with the overall project for which cross-sections are being prepared and, specifically, with the currently envisioned project layout and design and construction requirements (e.g., type, size, location, elevation of structures and facilities).

This "old-fashioned" way of drawing cross-sections by hand is more compatible with the "art" of producing high quality geotechnical drawings but, much more importantly, it allows the time necessary for critical thinking during preparation, checking, and revision of cross-sections. Items to be critically considered during this "old-fashioned" drafting process include geologic processes; landforms; lithologic, stratigraphic, and structural zones and features; patterns and features of erosion, deposition, weathering, alteration, and groundwater flow; and past, present, and future mining and mineral extraction, excavation, fill placement, dewatering, waste disposal, and other construction operations or remediation activities.

Cross-sections should be drawn as large as practicable. Their horizontal and vertical extents should be sufficient to depict the region of prime
interest with enough additional extent around the edges to show the context (e.g., property boundary, geologic, hydrologic, mining, construction) of this region. The cross-sections should include all relevant project features - surface, subsurface, and construction-related.

Revisions always occur during development of geotechnical cross-sections as various hypotheses of geologic and geotechnical interpretation are considered, tested, evaluated, and refined or rejected. This is why I recommend the use of soft lead pencils and appropriate erasers! After cross-
sections are developed in this manner and thoroughly checked, they can be put into a CAD system and manipulated as desired.

A plan drawing showing the locations and designations of all geotechnical cross-sections should of course be prepared. The scale should be clearly indicated on plan and cross-section drawings. Graphical scales are recommended so they will be appropriate on reductions or enlargements of the drawings. Elevation scales are useful on both sides of cross-sections so that horizontal reference lines can be added to prints and worksheets as needed.

Cross-sections used for geologic and geotechnical interpretations and analyses should have equal horizontal and vertical scales so geometric relationships are accurately portrayed. If necessary, cross-sections with exaggerated vertical scales can be developed after critical interpretations and analyses are done using cross-sections with equal horizontal and vertical scales. Cross-sections with exaggerated vertical scales are sometimes required for presentation of information on standard size drawings or pages. When they must be so utilized, the horizontal and vertical scales and the amount of vertical exaggeration should be clearly noted on each cross-section.

Most projects require preparation of a considerable number of cross-sections in order to develop the geotechnical framework of a site and resolve associated design and construction issues. This is because the "grain" or principal direction(s) of critical site conditions and project features is not always readily apparent or necessarily encountered in the first few cross-sections.

Intersections of cross-sections should always be checked for consistency of geologic interpretation in various directions. Sometimes this checking and subsequent reinterpretation and revision at cross-section intersections gives valuable insight into the geotechnical framework of the site.

From the relatively large number of interpretive cross-sections typically drawn, only a few showing the most important, critical, representative, or typical conditions may be selected for inclusion in reports and other documents. Care must always be exercised in selection and presentation of these cross-sections.

Examples of geotechnical cross-sections are given in Figures 1-4 to illustrate concepts presented above and to show geometric relationships of design and construction features with surface and subsurface conditions. Space limitations here preclude discussion of these cross-sections. Study of Figures 1-4 may prove beneficial to many readers.

4 CONCLUSIONS

Despite their utility and value in site characterization and other aspects of geotechnical practice, geotechnical cross-sections have declined in both quantity and quality in recent years. This decline appears to result from lack of experience in graphical communications among current geotechnical and environmental practitioners along with increased use of and reliance on computers for drafting, analysis, and design.

Basic procedures for developing high quality geotechnical cross-sections in the field to guide and optimize site exploration and in the office to develop the geotechnical framework of a site or problem were outlined. These procedures involve "old-fashioned" drawing of cross-sections with equal horizontal and vertical scales on the largest practicable sheets of grid paper by hand using pencils and erasers. This quasi-artistic process is a vital part of the analysis of site conditions as it both fosters and allows time for critical thinking during cross-section development. After interpretive cross-sections are developed in this manner and thoroughly checked, they can be put into CAD systems and manipulated as desired.

REFERENCE

Ground investigation for grouting design: A case history

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ABSTRACT: A case history of site investigation for grouting is presented. Following observed distress of a freeway drainage system, preliminary studies attributed the distress to groundwater infiltration. A potential flow path through backfill around underground pumping stations was proposed. Additional investigation focused on design of a remedial grouting scheme, however the findings did not confirm the postulated infiltration mechanism. Geophysical techniques were then employed to assess an alternate flow path below the pump station structure. This study identified anomalies interpreted to represent loosened soil zones, below invert, resulting in modified grouting remediation proposals.

1. INTRODUCTION

Grouting can improve ground prior to construction, remediate foundations and modify groundwater flow conditions. Grouting performance is dominated by the nature of in situ soils, thus a decision to employ grouting technologies implies preliminary knowledge of subsurface conditions. This may have been developed through local experience, or a site specific investigation.

Additional ground investigation focused on grouting can confirm the nature of a geotechnical problem, and thus select an appropriate method for resolution. The ground investigation should characterize the spatial extent of soils to be treated and their properties, in particular, particle size distribution, porosity and permeability. In addition, specific constructability issues, such as obstructions in overburden material, dynamic groundwater, and presence of voids or cavities within the treatment zone should be addressed.

This paper presents a grouting investigation case history, initiated by development of depressions and sinkholes along the shoulder of a Los Angeles freeway. A resulting geotechnical study associated the sinkholes with migration of fine soils into the freeway drainage system. The existing conditions, and initial site characterization are presented. These preliminary studies identified groundwater flow through permeable backfill around pump stations as a source of infiltrating groundwater. An additional investigation targeted this perceived geotechnical problem to optimize a grouting solution. The rationale applied to grouting investigation design, the resulting fieldwork program and interpretation of data are described. Study indicated that an alternate groundwater flow mechanism, channeled through materials below the pump station invert may be dominant. Consequently, a further study, employing geophysical methods was performed to refine the understanding of conditions below the structure and develop remediation proposals: Features of interest associated with this project include (a) complementary and phased use of boreholes, SPT, CPT, laboratory tests and geophysics as the investigation evolved to define a geotechnical problem, and (b) an investigation focused on ground treatment by grouting, planned and undertaken by a specialist contractor.

2. SITE DESCRIPTION

A 3 mile length of freeway was constructed in a 40ft deep cut, with pavement elevation at approximately 40ft, and side slopes at 2H:1V rising to the ground surface of surrounding areas, at around elevation 80ft. The freeway crosses an alluvial plain bounded by the Los Angeles River, San Gabriel River and the Rio Hondo. Native soils above
geotechnical studies were reported by Elkan (1990), and confirmed the previously described soil profile; aquiclude deposits comprising lenses of fine to medium sand, silty sand, silt and clayey silt at the elevations of the freeway and drainage system construction, underlain by sand and sandy gravel. Close to the drain pipes, local variations in stiffness of the alluvial soils were observed. These were associated with loosening and migration of fine soils into the storm drain system. The drainage system inspection correlated significant disturbance and infiltration with the proximity of the pumping stations. The pumping stations had been constructed by excavating into the freeway side-slopes, up to 20ft below proposed freeway pavement, fabricating the reinforced concrete structure, then backfilling to final grade. This excavation extended below the main drainage system elevation, penetrating into the sandy aquifer.

Based on this study of the storm drain distress, it was perceived that excavation and backfilling for the pump station construction had established a conduit enabling groundwater infiltration from the aquifer to the storm drain system. Distress of the drain, pavement and structure, was attributed to fines washing out from backfill under the resultant flow. The owner’s geotechnical study developed recommendations for remediation of the drainage system and the infiltration environment. A program of compaction grouting provided short term remediation of loosened fine soils to improve pavement support. Concurrent field testing of (a) grouting backfill around one pump station and (b) large scale groundwater lowering using pumping wells were planned as options to eliminate the source of distress. At this stage, the owner had identified a geotechnical problem, defined subsurface conditions and based on this information developed potential solutions, including grouting.

3. THE GEOTECHNICAL PROBLEM

During the Spring of 1995, open depressions and sinkholes developed at several locations along the depressed freeway section. In response to the pavement distress, the owner conducted a series of investigations to identify the source of difficulty and develop remediation options. The initial investigations employed boreholes and CPT tests, with associated laboratory testing, at intervals along the freeway shoulder, complimented by closed circuit TV inspection of the drainage system. These investigations confirmed the previously described soil profile, and identified the need for remediation techniques to address the distress.

4. DESIGN OF GROUTING INVESTIGATION

The remainder of this paper focuses on the ground investigation undertaken by a specialist geotechnical Contractor, Nicholson Construction Company, to design a remedial grouting scheme at one pump station. This pump station was identified by the owner as the most critical remediation location along the freeway alignment. The original study recognized the need for grouting methods and materials to ensure the backfill would be performed by a specialty grouting contractor. Thus, a contract was
prepared to perform a ground investigation defining the grouting scheme, and then to undertake the remedial work.

The investigation aimed to design a permeation grouting scheme to seal the backfill around the pumping station. This involved delineating the extent of backfill soils to be grouted, and selection of grouting methods (grout type, injection geometry and parameters). The method selection is dominated by permeability, dictating the range of grout types able to penetrate the soil (Figure 2), consequent radius of influence and thus injection geometry. Further, permeability determines the quantity of grout which can be injected at reasonable pressures.

The pump station layout and structure are shown in Figure 3. Freeway grade lies at elevation 43H, slopes rise to a frontage road at elevation 60H, continuing to the slope crest at elevation 80H. The storm drains and under-drains feed into a sump, with invert 11H below freeway grade. Twin 10H square, 8001 long storage boxes extend below the side slope and frontage road, draining into a 35H square pump shaft at the northern end of the structure. The pump station was installed during the original freeway construction, by excavation of a trapezoidal cut into the side-slope. Following concrete fabrication, the temporary cut was backfilled to the final side slope profile.

A desk study identified the following factors influencing selection of an investigation program:
(a) anticipated ground profile comprising a lessened aquiclude overlying clean sand/gravel aquifer
(b) backfill with assumed significant variation in permeability relative to native aquiclude
(c) assumed limits of backfill based on 1:1 slopes cut into side slopes
(d) constraints on equipment access due to sloped profile of site.

The proposed ground investigation comprised three boreholes with SPTs and sampling at 5ft intervals, 15 CPTs and sieve analysis of 10 selected samples. Boreholes enable physical inspection of soils and collection of samples for sieve analysis. The particle size distribution is central to estimating permeability and selecting grouting agent. SPTs provide approximate correlation to density and hence porosity. The CPTs provide rapid and economical method to study spatial variations across site, in this case profiling to delineate interface of native soils and backfill. Selected CPTs were located close to boreholes for correlation with observed soil type and structure variations. Measurement of permeability can establish the order of magnitude, however laboratory testing is constrained by difficulties associated with obtaining undisturbed samples in granular soils, while comprehensive in situ testing can be prohibitive expensive and has inherent uncertainty associated with soil homogeneity. In this case, a limited number of laboratory tests were undertaken. Groundwater elevations were to be determined across site by piezometric dissipation

![Figure 2. Limits of injectability of grouts based on permeability of sands and gravels (Cambo, 1977)](image)

![Figure 3. Plan and elevation of Pump Station Structure showing layout of boreholes and CPTs.](image)
testing, recording the equilibrium pore pressure at
variable depth and spatial locations.

The investigation planned arrays of CPTs and
boreholes across the site, in order to establish sub-
surface cross-sections, delineating backfill. The work
focused on frontage road and freeway shoulder due
to topographic access restrictions. However, some
borings and CPTs were proposed at slope crest to
enlarge overall distribution of investigation. The
planned depths for boreholes and CPTs were selected
to extend at least 10ft beyond an estimated
backfill-native soil interface.

5. FIELDWORK PROGRAM

The fieldwork comprised 3 boreholes and 16 CPTs
to depths ranging from 30 to 600ft, and was
performed by Gregg Institute of Signal Hill
California. The fieldwork duration was 10 days, and
layout is indicated on Figure 3. The three borings,
with SPTs, were drilled using hollow stem auger rigs.
BH1 was drilled from Frontage Road with a Mobile
B-61 truck mounted auger rig. BH3 and BH4 were
bored using a Simco 2400 and Marl MST truck
mounted drill rig, respectively. Split spoon or 3"-
modified California samples were taken at intervals
of less than 5ft. 9 were selected particle size
distribution analyses (ASTM D 422) and 3 for
unaturated hydraulic conductivity tests (ASTM D
5084). The CPT soundings employed at 10 cm3 cone,
with friction sleeve of 150 cm2, and shoulder
mounted pore pressure element, in accordance with
ASTM D 3441. All tests except CPT-15 were
conducted using a truck mounted rig with 25 ton
reaction load. CPT-15 was conducted using CPT
equipment mounted on a truck mounted Marl MST
rig with 5 ton reaction load. The cone recorded tip
resistance, sleeve friction and dynamic pore pressure.
At select phases during penetration, pore pressure
dissipation tests were conducted with monitoring at 5
second intervals.

6. GROUND INVESTIGATION ANALYSIS

The ground investigation identified three soil types
across the site. Material above elevation 40ft
comprises fine sand/silt (SM-ML), with 40 to 70% fine
and approximately 15% clay content. Permeability of this unit was estimated at 10^-6
cm/sec, based on laboratory testing. In the elevation
range of 20ft to 40ft, more varied, stratified sands,
silts and clays were encountered. In combination,
these two strata were interpreted to represent the
Belflower aquiclude. At elevation 20ft, a distinct
boundary with clean fine to medium sands (SP) was
observed. These soils exhibit no cohesion, and have
permeability estimated several orders of magnitude
greater than the aquiclude. Figure 4 presents a CPT
trace from frontage road, indicating strata
delineation. Grading curves contrasting the aquiclude
and aquifer materials are presented as Figure 5.
Groundwater elevation was 36ft at edge of
provenance, increasing to 45ft at northern extent of
pump station, indicating draw down within roadway
prism.

The three soil units described above were
identified at various locations relative to the pump
station, and clear delineation between backfill and
native materials was observed. Materials within the
assumed backfill zone had very similar composition to the native soils, and were compacted to similar in situ densities. Increased stratification and presence of clay layers in the elevation interval below 40 ft was interpreted as confirmation that these soils were native ground. Thus, the program identified an aquiclude comprising silt and clayey fine sand above elevation 20 ft, with no indication of voids or loose ground associated with fines wash out and groundwater flow paths around the pump station perimeter. However, one CPT (28) at the sump inlet indicated significant disturbance in a 20 ft zone below the structure invert. The pump station invert extended to elevation 16 ft, thus penetrated into the sand aquifer.

This investigation demonstrated that backfill soils had permeability comparable to native aquiclude soil, raising uncertainty over the predicted groundwater infiltration path. In addition, the backfill soils were too fine for treatment by permeation grouting, with 40 to 70% passing the #200 sieve. Thus, proposed grouting of the backfill was largely discounted as a remediation measure. However, the study did not confirm the true source of water infiltrating the storm drain system. Structure inspection, the anomalous CPT and distress history focused attention on aquifer soils underlying the pump station.

An additional complementary investigation was planned to study native materials directly below the structure. Due to the site access, conventional drilling techniques were discounted and geophysical study from within the pump station was proposed.

7. GEOPHYSICAL INVESTIGATIONS

The geophysical study, performed by Golden Associates Inc. of Seattle, WA, aimed to identify voids or loosened zones adjacent to the structure. A three-day fieldwork program employed Ground Penetrating Radar (GPR) to map the storage box walls and floor, selecting five characteristic locations which were tested using spectral analysis of surface waves (SASW) to quantify subsoil stiffness.

Ground penetrating radar is a high frequency, continuous reflection profiling technique, transmitting radar pulses into the subsurface. The pulses are reflected by subsurface discontinuities. The reflected pulses are recorded and processed to create an image of subsurface conditions. The ground mapping was developed by running GPR profiles with frequencies ranging from 1 GHz to 100 MHz along the walls and floor of the storage box, at 2.5 ft spacing. The study focused on the storage box, rather than sump or pump house, in order to minimize influence of structural irregularities on data (i.e., construction joints, variations in concrete thickness or rebar layout). The GPR identified anomalous zones below the structure, but did not locate similar discontinuities behind the walls.

Spectral analysis of surface waves is a seismic technique which determines shear wave velocity and shear modulus of sub-soils at selected locations. Geophones are attached to the structure at specified locations, and the floor is tapped with a hammer to generate an impulse. The spacing from geophones to hammer is increased systematically to provide subsurface shear wave records for increasing depth below the structure. Analysis of shear wave records is employed to determine elastic properties of subgrade. SASW studied 5 locations, selected from GPR mapping, correlating reduced sub-grade stiffness with anomalies detected in radar traces.

Thus, GPR anomalies were interpreted to represent loose soil zones. These results indicated that soil disturbance adjacent to the structure was restricted to localized regions beneath the pump house floor. The geophysical studies could not readily be verified by physical testing, since this would have involved coring through the concrete structure into granular soils below groundwater. The combined conventional and geophysical studies focused remedial proposals for groundwater infiltration on densification of soils below the pump station, and reducing permeability in this zone.

8. CONTRACTUAL PERSPECTIVE

Based on preliminary study, the owner recognized that remedial grouting could be employed for this project, and that the grouting scheme could best be selected by a specialist contractor. Bids were solicited from prequalified contractors for three stages of work: (1) ground investigation to define extent and nature of grouting, (2) grout cut-off injection and (3) verification through permeability testing. The ground investigation was a lump sum bid item, and placed liability for performing adequate investigation for the overall project on the contractor. The remedial grouting program defined by contract was permeation grouting. No grout type was specified, although payment was defined by cubic foot of grout injected. In order to prepare a price, the specialist contractor had to make significant assumptions regarding the remedial
scheme without the benefit of the ground investigation, in order to select grout type (cement, microfine cement or chemical), the take (grout volume cubic yard of soil treated) and the volume of ground to be treated. This item involved significant risk to the contractor in terms of quantities, but similarly provided scope for significant margins, should a large volume of backfill with high porosity exist. The verification procedure involved extensive permeability testing and was included for payment under the cubic foot of grout injected. This again presented a high risk margin item where a lump sum cost for well drilling, development and pump testing was to be compensated under a poorly defined unit quantity. Overall, the contract presented significant contractor risk since remedial grouting price was defined as unit rate, prior to performing ground investigation which would determine the means and methods of work performance.

9. CASE HISTORY OVERVIEW

Selection of grouting methods for a project assumes a preliminary knowledge of subsurface conditions. In this case, the owner conducted a ground investigation in response to freeway pavement settlement, which attributed distress to migration of fine soils into the storm drain system. A mechanism of groundwater infiltration through backfill around pump stations was proposed, and permeation grouting was identified as a remediation measure. The owner elected that additional investigation to optimize ground treatment scheme be performed by a geotechnical contractor.

The additional investigation studied the perceived mechanism of groundwater infiltration through backfill, aiming to define grouting extents, estimate grout take and select an appropriate grout type. This focused study was necessary for the Contractor to prepare an appropriate work plan. The investigation employed boreholes, with sampling and laboratory testing to define soil permeability and porosity, then extrapolated spatially across the site using correlation to CPT data. The results of this investigation profiled and classified soil units present, but did not confirm the perceived mechanism of groundwater infiltration through backfill.

Further investigation by geophysical methods, evaluated an alternate mechanism of groundwater infiltration through soil below structure invert. This program identified anomalies in materials below the structure, leading to development of remedial proposals treating this zone. One proposal included a two stage approach. First, a cut-off perimeter wall surrounding structure and extending 10 ft below invert was to be installed by jet grouting, jet grouting was recommended since it could treat the heterogeneous soils encountered in both aquiclude and sandy units. The second phase aimed to compact and reduce permeability of soils underneath the structure by permeation grouting. This work would be performed from within the structure, and would treat the sandy materials below invert. These coarser soils have grain size distribution which permits treatment by permeation grouting. The combined jet and permeation treatment aims to create a great bath-tub surrounding the structure, stabilizing loosened material, and restricting groundwater flow.

This case history defined the rationale of site investigation and progressive stages of site characterization applicable for grouting design. This evolving approach can optimize work efficiency, cost effectiveness and quality of results for owner. Contractual and liability issues associated with delineation of work between owner and specialty contractor are described, suggesting that project success can be enhanced in an environment which allows review and adjustment of conditions to reflect improved comprehension of the geotechnical problem.

10. REFERENCES


Use of SID technique to determine heterogeneity of contaminated sites

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ABSTRACT: A system identification (SID) scheme is developed to determine the spatial variability of the transport parameters at contaminated sites. In order to determine site heterogeneity, the flow domain is discretized into rectangular elements and independent contaminant transport parameters assigned to each element. The determination of the transmissivity field at the site is accomplished by optimizing the error between measured contaminant concentration and that calculated by a selected contaminant transport model. This SID scheme uses the state-space form of the contaminant transport equation as the predictive model and the Levenberg-Marquardt algorithm as the optimization scheme. The SID methodology is applied to the Jordan aquifer in Iowa and to a site located in the Texas Southern High Plains and the results discussed.

1. INTRODUCTION

The remediation of sites with soil and groundwater contaminated by leached fuels and hazardous wastes is one of the most important and financially oppressive challenges that many industrialized countries are faced with today. Investigation and monitoring have begun at many of these sites, but execution of remedial plans is often delayed due to regulatory and financial constraints as well as the limited understanding of the processes that control the movement of contaminants in the subsurface.

One of the major difficulties that is encountered in the risk assessment and the design of remedial systems is our inability to describe the true heterogeneous nature of the subsurface adequately. Due to financial and time constraints it is not feasible to perform a large number of tests that will be necessary for accurate site characterization. At the same time, however, there are already existing monitoring well records that describe the movement of the contaminant in the subsurface. This paper describes a methodology that will make use of the monitoring well data on solute concentrations to improve our understanding of the site heterogeneity. This approach uses the use of a technique known as System Identification Method.

2. SYSTEM IDENTIFICATION METHOD

System Identification deals with the problem of building mathematical models of dynamical systems based on observed response from the system to known input. A system is an object in which variables of different kinds interact and produce observable signals. The observable signals that are of interest to us are called outputs. The system is also affected by external stimuli. External signals that can be manipulated by the observer are called inputs.

The construction of a model from the data involves three basic entities.
1. Data Collection – The input/output data are sometimes recorded during specifically designed identification experiments or from the normal operation of the system.
2. Selection of candidate models – This is the most important choice of the system identification procedure. A priori knowledge and engineering intuition have to be combined with formal properties of models.
3. Selection of a rule by which candidate models can be assessed using the data – This is the Identification Method.

System Identification Method (SID method) and other inverse parameter estimation methods have been used frequently in groundwater hydrology problems for a long time. Most of the inverse problems in groundwater involve the estimation of the spatially varying transmissivity field by using the flow equation. A review of such methods was provided by Yeh (1986). The present application involves the determination of the transmissivity field and other transport parameters at a contaminated site.
by using measured concentrations at various locations in the site in conjunction with the flow equation and the transport equation. The following section describes the formulation of the SIT scheme for this purpose.

3. FORMULATION OF THE SIT SCHEME

3.1 State-Space Models for Linear Dynamical Systems

The relationship between the input, noise and output of a linear dynamical system can be expressed in the following state-space form (Ljung, 1987).

$$ \frac{dx(t)}{dt} = F(\theta)x(t) + G(\theta)u(t) $$

(1)

$F$ and $G$ in the above equation are matrices of dimensions $n \times n$ and $n \times m$, respectively where $n$ is the number of observations in time and $m$ is the number of observations in space. Moreover, $\theta$ is a vector of parameters that typically correspond to unknown values of physical coefficients, material constants etc. The discrete form of Eq.(1) can be written in the following manner (Ljung, 1987).

$$ x(kT+T) = A_x(\theta)x(kT) + B_x(\theta)u(kT) $$

(2)

The above equation provides a relationship between $x$ at time $t = kT+T$ and $x$ at time $t = kT$.

$$ A_x(\theta) = \exp(F(\theta)T) $$

and

$$ B_x(\theta) = \int_0^T \exp(F(\theta)t)G(\theta)dt $$

3.2 Governing Equation for Contaminant Transport

To apply the above methodology to contaminant transport in a saturated aquifer, it is necessary to transform the relevant governing partial differential equation to the state-space form represented by Eq.(2).

The equation used to describe the two-dimensional convection and dispersion of a given non-reactive chemical species in flowing groundwater was derived by Reddell and Sunada (1970) and Konikow and Grove (1977). This equation may be written as follows:

$$ \frac{\partial(C)}{\partial t} - \frac{\partial}{\partial x_1}(D_{11}\frac{\partial C}{\partial x_1}) - \frac{\partial}{\partial x_2}(D_{22}\frac{\partial C}{\partial x_2}) = - \frac{\partial(CW)}{\partial x_2} $$

(3)

where

- $C$ is the concentration of the dissolved chemical species
- $D_{11}$ is the coefficient of hydrodynamic dispersion
- $b$ is the saturated thickness of the aquifer
- $C'$ is the concentration of the dissolved chemical in a fluid source or sink

The first term on the right hand side of Eq.(3) represents the change in concentration due to hydrodynamic dispersion. The second term describes the effect of convective transport, while the third term represents a fluid source or sink that is assumed to be zero in this case.

By assuming that there is no spatial variation in the saturated thickness of the aquifer and by expanding convective transport term, Eq.(3) may be written in the following manner.

$$ \frac{\partial C}{\partial t} - \frac{\partial}{\partial x_1}(D_{11}\frac{\partial C}{\partial x_1}) - b \frac{\partial C}{\partial x_1} $$

(4)

By using Scheidegger’s equation, the dispersion coefficient may be expressed in terms of the velocity of groundwater flow and to the nature of the aquifer (Scheidegger, 1961).

$$ D_1 = \alpha_{1m} \frac{V_m V_n}{|V|} $$

(5)

where

- $\alpha_{1m}$ is the dispersivity of the aquifer
- $V_m, V_n$ are the components of velocity in the $m$ and $n$ directions
- $V$ is the magnitude of the velocity

Scheidegger (1961) further showed that for an isotropic aquifer, the dispersivity tensor could be defined in terms of the longitudinal and transverse dispersivities of the aquifer $\alpha_1$ and $\alpha_2$ respectively. There are related to the longitudinal and transverse dispersion coefficients as follows.

$$ D_1 = \alpha_1 |V| $$

$$ D_2 = \alpha_2 |V| $$

(6)

The components of the dispersion coefficient for two-dimensional flow in an isotropic aquifer may be stated as follows.

$$ D_{11} = D_1 \frac{V_m^2}{|V|^2} + D_2 \frac{V_n^2}{|V|^2} $$

(7)

$$ D_{22} = D_1 \frac{V_m^2}{|V|^2} + D_2 \frac{V_n^2}{|V|^2} $$

(8)

$$ r_{mn} = D_{mn} - (D_1 - D_2) \frac{V_m V_n}{|V|^2} $$

(9)
Figure 1. Notations Used in State-Space Model Formulation

Finite Volume formulation with upwind velocity scheme was used to discretize the Eq. (4). By substituting equations (7), (8) and (9) to Equation (4), it is possible to convert Equation (4) to a state-space model.

In the following derivation, \( C_i(T+T) \) represents solute concentration in cell no. \( p \). Subscripts \( N, S, E, \) and \( W \) denote adjacent cell centers. Lowercase \( n, s, e, \) and \( w \) represent cell faces.

\[
C_i(T+T) = (A_N C_N(T) + A_E C_E(T) + A_S C_S(W) + A_W C_W(N)) \left( \Delta x \frac{\partial^2 C_i}{\partial x^2}(T+T) \right) + \left( A_{NS} C_{NS}(T) + A_{SE} C_{SE}(T) + A_{SW} C_{SW}(T) \right) \left( \Delta x \frac{\partial C_i}{\partial x}(T+T) \right)
\]

where

\[
\Delta = \Delta x \Delta y / \Delta t
\]

\[
A_N = \frac{\Delta y \Delta x}{\Delta t} \left[ (D_{x,1} s) + V_s \Delta y \frac{\partial S_i}{\partial x}(T+T) \right]
\]

\[
A_E = \frac{\Delta y \Delta x}{\Delta t} \left[ (D_{x,1} e) + V_e \Delta y \frac{\partial S_i}{\partial x}(T+T) \right]
\]

\[
A_S = \frac{\Delta x \Delta y}{\Delta t} \left[ (D_{x,1} n) + V_n \Delta x \frac{\partial S_i}{\partial y}(T+T) \right]
\]

\[
A_W = \frac{\Delta x \Delta y}{\Delta t} \left[ (D_{x,1} w) + V_w \Delta x \frac{\partial S_i}{\partial y}(T+T) \right]
\]

\[
A_{NS} = \frac{1}{2} \left[ (D_{x,1} w) - (D_{x,1} n) \right]
\]

\[
A_{SE} = \frac{1}{2} \left[ (D_{x,1} s) - (D_{x,1} e) \right]
\]

\[
C_i(T+T) = A(V_N, V_E, \alpha_N, \alpha_E) C_i(T)
\]

Equation (11) is now in the form of general equation, i.e. Eq.(2).

3.3 Governing Equation for Groundwater Flow

Steady, Two-dimensional flow equation in a non-homogeneous, isotropic confined aquifer with prescribed head boundary conditions is described by Eq.(12) below.

\[
\frac{\partial}{\partial x} \left( T \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( T \frac{\partial h}{\partial y} \right) = 0
\]

where

- \( h \) is the hydraulic head (L)
- \( T \) is the transmissivity tensor (L²/T)

By using a finite-volume scheme with a central difference formula, the following equation is obtained for the head at the center of cell p.

\[
h_p = \frac{\Delta y}{\Delta x} \left[ T_{n,p} + T_{h,n} \right] \frac{\Delta x}{\Delta y} \left[ T_{n,e} + T_{h,n} \right]
\]

In the above equation, the transmissivity values at the cell faces were then approximated by the average of the transmissivity values corresponding to the cell centers on either side.

3.4 Parameter Identification Formulation

The solute concentration at the end of time step can be determined from Eq.(11) provided that \( A(V_N, V_E, \alpha_N, \alpha_E) \) is known. However, in the present inverse problem, \( A(V_N, V_E, \alpha_N, \alpha_E) \) consists of the unknown site parameters that must be determined.

Let us now assume that there are M observation wells at which the piezometric head and solute concentration are recorded over N number of times. We introduce \( \eta_{pk} \) as the error resulting from the comparison of calculated and observed solute concentrations at well no.\( p \) at \( k^{th} \) time step.

\[
\eta_{pk} = C_p(T+T) - C_{op}(T+T)
\]

where \( C_{op}(T+T) \) is the observed concentration value and
\( C_p(kT+T) \) is the calculated concentration value at well no.p.

Subsequently, sum of squared errors can be estimated by the following equation.

\[
SSE = \sum_{p=1}^{P} \sum_{t=1}^{T}(x_{pt} - \hat{x}_{pt})^2
\]  

(15)

Alternatively, an error term be calculated based on head calculations performed by Eq.(13).

\[
\varepsilon_p = h_p' - h_p
\]

where

- \( h_p' \) is the observed head value
- \( h_p \) is calculated head value, which is obtained from Equation (13)

\[
SSE' = \sum_{p=1}^{P}(\varepsilon_p)^2
\]  

(16)

Finally, the error terms obtained from Eq.(15) and Eq.(16) are optimized by using the modified Levenberg-Marquardt Algorithm.

4. VALIDATION OF THE SID PROCEDURE

The first step in the validation process involved a series of tests on the inverse parameter estimation procedure. For this purpose it was first assumed that the necessary transport parameters were known at the site and then the model run in a forward sense to generate hydraulic heads and solute concentrations at various points within the site. Subsequently, this data was used as input to the inverse SID model and transport parameters were estimated over a wide range of initial estimates. The results showed that for a grid of 16 x 16 estimates were within ±1% of the original transport parameters. Additionally, it was observed that the inverse procedure was capable of converging to correct solutions even when the initial estimates were varied over more than two orders of magnitude.

Two separate case studies for which the solutions were available were then selected from literature for the validation of the new SID algorithm.

Case Study 1

The first of these case studies involved the determination of the transmissivity field for the Jordan aquifer of Iowa.

The data needed for this validation was presented by Hockema and Kitanidis, (1985). Like most other, not involve any contaminant movement within the aquifer. This exercise, therefore, was limited to the use of SID technique to determine the transmissivity field. The piezometric head field needed for such analysis was available in the form of head contours.

In the present analysis, the region of interest was divided into 12 x 12 grid. The values for the hydraulic heads at nodal points on the grid were obtained by manual interpolation from the head contours available. The transmissivity field obtained from the back calculation process is shown in the Fig. 2, while the transmissivity field presented by the original authors is shown in Fig. 3.

A comparison between the transmissivity field provided by the original authors and that determined by the SID method revealed that the two compared within ±10%. Larger deviations were observed in the north-east and south-west corners where the...
monitoring wells. The bromide tracer was injected into well B. The changes in hydraulic head and bromide concentration within the aquifer with time were observed from Wells A through K. Unlike the case study 1, in this example, the head distribution within the site did not have sufficient resolution to make accurate estimation of the transmissivity field for the site. Therefore, Fedler et al. (1989) used the data on contaminant levels to estimate the transmissivity field by using the USGS-MOC model by Konikow and Bredehoeft [1978]. This was accomplished by manually adjusting the transmissivity value at each cell in the model until best agreement was obtained between calculated and measured contaminant concentrations in all the wells. Estimated transmissivity values from the USGS-MOC model are also shown in Fig. 4. For the purpose of SID model validation, the same inverse calculation process was repeated using the SID scheme. In this analysis, only that part of the site where significant variation in contaminant levels with time was included. This region included wells B, C, D, E, F, G, and H. Consequently, the grid size was limited to 3 x 3. In this manner, possible errors resulting from interpolation were avoided. The resulting bromide concentration versus time curves for the wells E and H are shown in Figures 5 and 6. These figures also include the results obtained by the original authors. From the results shown in the above figures it is evident that the SID scheme was able to match the actual data much better than the manual procedure could.

The back-calculated transmissivity values are shown in Table 1. Comparison of transmissivity values estimated using SID procedure with those obtained from USGS-MOC model indicates that there is close agreement on the downstream side of the injection well B than on the upstream side. This result must be expected because wells on the downstream side, opposite to injection well B.
Table 1. Back-calculated Transport Parameters in Case Study II

<table>
<thead>
<tr>
<th>Vc</th>
<th>Vr</th>
<th>Tr</th>
<th>Tp</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.549</td>
<td>0.914</td>
<td>5.432</td>
<td>1.600</td>
</tr>
<tr>
<td>Vc</td>
<td>Vr</td>
<td>Tr</td>
<td>Tp</td>
</tr>
<tr>
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<td>2.070</td>
<td>4.942</td>
<td>3.629</td>
</tr>
<tr>
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<td>Vr</td>
<td>Tr</td>
<td>Tp</td>
</tr>
<tr>
<td>0.643</td>
<td>0.330</td>
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Note: \( V_c \) and \( V_r \) are given in cm/day and \( T_r \) and \( T_p \) are given in cm/day.

6. REFERENCES


Site characterization for Hong Kong’s new airport at Chek Lap Kok

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ABSTRACT: This paper briefly discusses the site investigation carried out at the airport particularly with respect to characterising the site for design and construction purposes. The difficulties associated with site characterisation as a result of both the size of the project and quantifying the variable nature of the underlying fill and superficial deposits are discussed. Three levels of characterisation are presented which were used to take these factors into account. Examples of how each of these methods have been applied are given.

1 INTRODUCTION

Hong Kong’s new airport at Chek Lap Kok when complete will consist of two runways, a passenger terminal building, extensive cargo handling facilities and its own infrastructure of roadways, bridges, cut and cover tunnels together with a dedicated airport railway. The island upon which it is constructed covers 1248 ha, 75% of which is man-made reclamation.

The average frequency of boreholes which penetrate the full thickness of superficial deposits beneath the airport is only around 1 per hectare, although a considerable amount of additional site investigation was conducted consisting of pre-boreholes for piled structures, shallow cone penetration tests (CPT) for determination of dredge levels and installation of sub-surface geotechnical instruments. Compared with other civil engineering projects, which are either relatively small or follow long alignments, the airport site has been, and remains to be a challenge to characterise simply due to its large area. This paper will introduce the concept of global and specific characterisation and how applicable each is to the various components of the design and construction of the airport.

2 SITE DESCRIPTION

The airport is located in the Pearl River Estuary, to the east of Hong Kong as indicated in Figure 1. The site is underlain by a Quaternary Age succession of deposits comprising a very soft marine clay that overlies a complex succession of alluvial silt clays, clayey silts, sands and gravels. These in turn overlie a buried topography comprising Mesozoic and Tertiary Age igneous intrusive and volcanic rocks in varying degrees of weathering. The alluvial sands and clays are generally between 10 m and 30 m thick. The average and maximum depths to slightly decomposed granite are 45 m and 120 m respectively.

The soft marine clay was removed by dredging prior to the reclamation of the airport platform. Concurrently the islands of Chek Lap Kok and Lai Chi Chau were both reduced to a level of approximately +6 m NAD to blasting and quarrying. The 107 Mm$^3$ of rockfill generated was used in the reclamation. An additional 70 Mm$^3$ of marine sandfill was also imported to complete the reclamation. In general rockfill was placed by end-tipping from land-based plant and the marine sand was placed by bottom dumping, rainbowing and
spigotting from large dredgers (Uiterwijk et al., 1994).

3 SITE INVESTIGATION

Site investigation specifically for the new airport commenced in 1981 with the Civil Engineering Design Studies for a Replacement Airport at Chek Lap Kok by RMP-Encon (1982). This included construction of a test embankment and the development of a site preparation scheme. In 1990 the Geotechnical Control Office of Hong Kong carried out a further investigation for an extension to the proposed airport, and in 1991 the New Airport Master Plan was completed whereupon further site investigation was carried out for detailed design of the final airport configuration. Seismic reflection surveys were carried out in 1981 and again in 1991 which gave a full coverage of the extended site.

A number of continuously sampled boreholes (termed stratigraphic reference boreholes), instrument holes and CPTs were used in the geological interpretation of the seismic data. Over 3000 shallow marine CPTs were carried out at a 100-m grid in order to define the dredge lounds. Boreholes, both land-based and marine, have been one of the most common methods of investigation. They have been drilled both on the former islands of Chek Lap Kok and Lam Chau and on the new reclamation. The boreholes in the reclamation typically extend to depths of around 45 m. Routine testing such as standard penetration tests (SPT), permeability tests and field vane tests (FVT) have been carried out in the boreholes. Pre-bores were also conducted on piled structures, this being a statutory requirement in Hong Kong.

Samples gathered during site investigations have been tested in the laboratory with the most common tests carried out being the particle size distribution, plasticity indices and oedometer tests. More specialised laboratory testing has also been carried out in the laboratory but this testing is beyond the scope of this paper.

4 OVERVIEW OF GROUND CONDITIONS

The marine geology was interpreted and reported by James et al. (1994), based upon the correlation of borehole and CPT data with the seismic reflection survey data. The Quaternary deposits were identified as belonging to three geological formations; the Hang Hau (QHH), the Sham Wat (QSW) and the Chek Lap Kok (QCK) Formations. These were further divided into units and sub-units based upon the seismic reflectivity, and lithologies from borehole descriptions and diagnostic CPT responses (Fig. 2), as described by Covill & James (1997). This sub-division allowed a detailed site specific three-dimensional geological model to be produced.

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Figure 2 Interpreted seismic reflection survey and CPT log.

Figure 3 Geological and geotechnical cross-section along the northern runway.
The development of the geological model into a geotechnical model followed a conventional approach. The soil strength, consistency and soil indices were reviewed from the borehole logs, laboratory testing, in situ testing such as SPT, PVT and CPT and were correlated with the geological model. This was an iterative process in that both the geological and geotechnical models were refined and developed in tandem, to produce the final geological and geotechnical models.

The geological model is based on the interrelationship between the various stratigraphic units. Each unit can comprise interbedded sands and clays and also contain stiff palaeosol horizons. The geotechnical model is based on a simplification of the lithology of the site and subdivides the main clay bodies into zones which after completion of the reclamation will be normally consolidated, normally over-consolidated or over-consolidated. The geotechnical model provided the framework upon which engineering decisions related to the design and construction of the airport could be based. This model has sufficient detail to provide a general overview of the behaviour of the reclamation and associated facilities.

Figure 3 shows cross-sections along the northern runway from both the geological and geotechnical models. The geotechnical model represented in Figure 3 can be used to provide site-wide estimates of the settlement of the reclamation, but has insufficient detail, for example, for designing piled foundations.

The main deficiency of the geotechnical model relates to the variability of the superficial deposits which underlie the site. These have been deposited over the past 250,000 years, during which time the sea level has fluctuated by more than 100 m. The deposits have been subject to a number of erosion and deposition cycles. This has resulted in a number of erosion surfaces with palaeosol crusts, infilled channels and varying degrees of over-consolidation across the site, that add to the complexity.

The vertical and lateral variability of the materials found at the site are demonstrated on Figure 4, which is a section at the western end of the northern runway, based on boreholes and CPTs at approximately 20m intervals correlated with the
seismic reflection survey. Variability within a particular layer is demonstrated on Figure 5 which shows two borehole logs and two profiles from adjacent corresponding CPT’s. The CPT’s show the variability in the soils in terms of strength, overconsolidation state and variation in material type with depth. Although the simplified boreholes are quite similar, the CPT profiles are still markedly different and show significant variations. Variability even within a single geological unit has also been identified from the scattered results of simple plasticity index testing from clays of the QCR Formation, shown in Figure 6.

5 LEVELS OF SITE CHARACTERISATION

When characterising a site an understanding of the global picture is usually developed before investigation of the specific details. This approach was applied at the airport site and developed further in three distinct processes: A global overview of geology and geotechnical characteristics to produce the geological and geotechnical models; localised investigations to better understand details of ground behaviour, the results of which investigation can be applied to other areas of the site; and finally localised investigations specifically for individual structures or anomalies. At the airport site development of the site characterisation is a continuous process, in that it makes full use of the monitoring data to improve both the investigations at the global and localised levels.

5.1 'Global' characterisation for general overview

The variable nature of the soils proved that it was neither appropriate nor possible to assign specific design parameters to the stratigraphic units. The approach taken for each unit was to determine an average, and associated upper / lower bound value for each parameter, and then use these values in analyses to check the sensitivity of the design to changes in the soil parameters. Two examples follow which were based on the global characterisation which is a feature of the geotechnical model.

Prior to construction of the reclamation, only limited monitoring results were available. It was therefore only possible to predict settlement by analytical methods based upon borehole, CPT and laboratory test results. Following a review of the available test data and the geotechnical model typical soil profiles were selected at which to carry out settlement analyses. Figure 7 shows an example of the settlement prediction made for the reclamation based upon these results and a review of the geotechnical model. Prior to construction this type of assessment was applicable, as the end result of the assessment was to determine the amount of fill to be used in the reclamation to allow for settlement, rather than to predict for example differential settlement over short distances.

The basal sand and gravel is the only stratigraphic unit at the site that is of sufficient extent to enable the geotechnical model to provide a fully representative characterisation. In this unit the monitoring data from a relatively small number of piezometers were sufficient to demonstrate that this layer formed an imperfect drainage boundary at the base of the superficial deposits. Additional investigation of this unit would not improve the site characterisation or improve prediction of performance.

5.2 Specific characterisation for global application

Although certain aspects of the design can be developed using the geotechnical model, the simplifications inherent in this model do not allow a full appreciation of the effects of the variability of the ground to be fully understood. In order to overcome this deficiency, detailed investigations
were carried out at various locations on the site. The results obtained at these locations were then used to characterise the behaviour of the remainder of the site. Three examples of this approach to site characterisation are given below.

One of the main sources of information regarding the platform behaviour has been the geotechnical instruments installed over the platform (Newman et al, 1995). A total of 60 groups or clusters of instruments have been installed together with a number of individual instruments. A cluster generally consists of a central inclinometer / extensometer system and two or three piezometers within a 10m radius. Each instrument also has a surface settlement marker.

Instrumentation clusters were primarily installed to monitor pore pressure in the underlying alluvium and settlement in both fill and alluvium. The instruments were installed in boreholes, the position of the instruments (piezometer tip or extensometer ring) were selected based upon the borehole drilled for the central extensometer. It was decided to cluster the instruments principally because of the lateral variability of the strata, the close grouping thus allowing a good understanding of the various components of settlement to be developed at specific locations.

The network of isolated instruments plus the large number of surface markers then made it possible to interpolate between cluster locations to predict settlement over the whole site. An observational approach to the assessment of settlement was used based upon the instrumentation monitoring results. This reduced the need for the more conventional analytical methods of determining settlement, and thus the need for fully defining geotechnical parameters for the materials.

Another example of specific characterisation for global usage, was the installation and monitoring of three dense grids of surface settlement points each grid covering approximately one hectare. These were installed to investigate the mode of surface differential settlement caused by the variability of both the fill and underlying alluvium. The results which characterise the settlement performance of the reclaimed area clearly demonstrated that the variability of the uncompacted fill material would have a significant effect on differential settlement over relatively short distances (less than 40m). On the basis of the differential settlement characteristics obtained from these small areas, the whole of the northern runway was treated by a surcharge of fill material. This surcharge was progressively “rolled” along the runway and was in place only long enough to treat the fill rather than the underlying alluvial deposits.

This type of approach where the results of a detailed investigation are used to characterise the whole site, was also used during the feasibility stage of the development of the project. Foot et al (1987) reported on the construction and results of a test embankment constructed on 1 ha of the site. The embankment tested various arrangements of vertical drains installed in the soft marl clay. The results obtained were used to characterise the various materials, and these details were applied in various airport designs across the reclamation.

5.3 Specific characterisation for detailed use

In some situations the global characterisation based on the two methods discussed above are not sufficient to allow detailed design. In these situations it is necessary to carry out localized investigations, the results of which are only generally applicable to the particular area concerned. Three examples of this type of investigation are given below.

At the ends of the runway, beyond the reclamation, there are sets of approach lights that required piled foundations in the sea. Figure 4 shows the geotechnical cross-section through one set of approach lights. The piles were to be designed to resist lateral loading from accidental ship impact hence a detailed investigation was required due to the known high variability in the extent (both vertically and horizontally) of the geotechnical strata.

Acoustic turbidity, as shown in Figure 3, tended to mask the geological strata response shown on the results of the seismic survey. The turbidity is a result of occluded biogenic gas within the marine mud. A detailed investigation was carried out (NGI, 1990 & Premchitt et al, 1992) to assess the effect of this on the engineering properties of the sediments within and adjacent to areas of turbidity. The test results indicated that the gas had no significant effect on the geotechnical properties.

Another example of the use of specific site investigation is for design of end-bearing piles. The length of a pile is determined by the depth to a strata that has adequate bearing capacity. As it is difficult to guarantee the exact depth at a particular location without specifically investigating that location, tentative pile lengths are indicated in the initial design, and are proven by pre-boxes prior to pile construction.

6. CONCLUSIONS & RECOMMENDATIONS

The reclamation for the new airport has an area of 938 ha and the soils underlying the site are extremely variable in both extent and engineering properties. The combination of the size of the site and residual variability means that the usual
methods of site characterisation used on smaller projects are not directly applicable. The approach adopted for the site characterisation at the airport illustrate that it is possible to characterise a large and variable site by first developing a global understanding of the ground conditions and then carrying out detailed investigations either for global application or to investigate specific features. This approach is cost effective in that it reduces the extent of detailed investigation required to characterise the site whilst also reducing the risk of unforeseen ground conditions by generating sufficient information in specific areas.

A three level approach has been adopted. At the first level, simplified geological and geotechnical models of the site have been developed. These models do not have sufficient detail to fully represent all the variations in the stratigraphy of the ground below the site. However, they are able to represent the overall behaviour of the reclamation in sufficient detail to allow, for example, global settlement patterns to be predicted.

At the second level detailed investigations have been carried out on a number of small areas of the site to develop a better understanding of ground behaviour. The important aspect of this type of investigation is that the results are then used to characterise the likely behaviour of the remainder of the site.

At the third level, detailed investigation is carried out at specific locations either to investigate anomalies in the underlying stratigraphy or for design of particular facilities to be constructed on the reclamation.

The importance difference between the first two approaches to characterisation and the third is that the locations of investigation points in the former are in essence random, although the pattern of the investigation obviously requires consideration.

As with all site investigation work it is good practice to carry out the work in stages and to develop the site characterisation as the project develops. At the airport site there were a number of stages in the investigation as the project developed.

The investigation continued during construction as the location of the various facilities on the site were finalised. In addition, the performance of the reclamation was monitored during and after its construction. The results obtained from this monitoring work have been of great value when characterising the performance of the site. During the later stages of design of the project the monitoring results have, to a large extent replaced the parameters derived from in situ and laboratory testing. Site characterisation should not stop at the design stage.

ACKNOWLEDGMENTS

The authors thank the Board Members of the Airport Authority Hong Kong for permission to publish this paper. The authors acknowledge their colleagues for advice and discussion, especially Dr G Phuit.

REFERENCES


Characterization and performance at the reclaimed island of Rokko in Kobe

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ABSTRACT: The paper deals with the site characterization initiatives at one of the large scale land reclamation sites in Kobe in the light of the Hyogoken-Nambu earthquake experience. The challenges involved with characterization of reclaimed site for a large scale development project in a seismically active zone is highlighted. Attempt is made to relate the site characterization efforts to the long-term monitoring after land reclamation and the effectiveness of the ground improvement initiative.

1 INTRODUCTION

Reclaimed ground conditions present unique challenges for site characterization, specially when the site is in a seismically active area. One of the heavy concentration of reclaimed land in Japan can be found at the port city of Kobe, located in the Osaka bay about 500km west of Tokyo. Owing to the narrow natural flat ground, the seaward expansion of Kobe city started early on, and can be traced back to several hundred years. Two major land masses reclaimed in recent times are Port island and Rokko island. Reclamation of the 580ha (1433 acres) Rokko island started around 1973 and was completed in 1993. The extent of the urban growth and the reclamation activity around Rokko island over the past 100 years can be seen in Fig. 1.

Considering the potential seismic activity in the region, a conscious effort was made in selecting a suitable fill material. The selected material was considered to be competent, well graded and resistant to large scale liquefaction during strong ground shaking. The magnitude 7.2, January 17, 1995 Hyogoken-Nambu earthquake in Kobe tested these initiatives to the limit. This paper summarizes the site characterization and monitoring efforts undertaken as part of the city core development project in the central part of Rokko island. Attempt is made to provide a prediction scenario performance case study in conjunction with the long term monitoring after reclamation and the efficacy of the soil improvement methods utilized.

2 GENERAL GEOLOGICAL FEATURES

The city of Kobe is located at the foot of the Rokko mountain system (Fig. 2), believed to be created by thrust movements primarily along east-west and northwest-southeast directions, resulting in the development of several active faults at the boundaries. The narrow and oblong coastal plain at the foot of the mountain consists of a complex combination of alluvial fans formed by several small streams flowing into the bay. Rokko island itself looks like an extension of the alluvial fan formed by the Sumiyoshi river in Fig. 2. The geological structure in this region consists of very deep and complex sedimentary formations overlying the mainly granite basement at depths of a kilometer or more.
The uplifting Rokko mountain range is a outcrop of the granite basement, that deeps steeply down towards the bay. The latest Cenozoic crustal movements, to which the Hyogoken-Nambu earthquake process itself may have been related, have resulted in the complex geomorphological features of the region that is known to have one of the highly dense network of active faults in Japan.

3 SITE CONDITION OF THE PROJECT AREA

The reclamation and development of Rokko island consisted of a city core at the central part surrounded by port, storage and industrial facilities all around. The city core is comprised of commercial, housing and recreational facilities separated from the outer zones by green belts. The history of the land reclamation for Rokko island is depicted in Fig. 3, where the numbers indicate the year of reclamation of the different parts of the island. The city core project area is enclosed by bold lines in Fig. 3, and the project area is divided into 12 different blocks, numbered A1 to A6 and B1 to B6.

As can be noted from Fig. 3, most of the project area was reclaimed around 1979-80. Exploration and laboratory tests for the characterization of the project site lasted for about four years beginning October 1986. Altogether 105 borings were made for exploration, field testing and sampling, including frozen sampling. The average boring depth was about 60m, with most of the borings in the range of 50 to 80m. Deepest boring was 167m, and 12 of the borings were more than 100m deep. The overall site condition is summarized in Fig. 4.

The depth of the fill material in the project area varies from about 17m to over 30m. Underlying the fill is the Holocene alluvial clay deposit, whose thickness varies from 7 to 19m. The soft alluvial clay is underlain by 40 to 44m thick diluvial layer consisting of alternate bedding of dense sand and stiff clay, below which lies 12 to 14m thick diluvial clay layer. Below the diluvial clay layer lies interbedding of clayey and sandy deposits in different combination, that seems to continue well below the maximum exploration depth.

3.1 The Fill Material for Rokko Island Reclamation

The fill material for reclamation of Rokko island was primarily brought in from the hills surrounding the Kobe city, where several new town development projects were being undertaken. The decomposed granite, locally known as musando, was brought in from Awaji island to fill the northern part reclaimed...
before around 1978 (Fig. 3). The **nanzado** was the fill material in the adjacent Port island, that witnessed extensive liquefaction failure during the Hyogoken-Nambu earthquake. The fill material of Rokko island consists of a combination of tuff, mudstone, sandstone, chert etc., constituting the so called Kobe group.

### 3.2 Ground Improvement at the Project Site

The ground improvement at the site consisted of the installation of 500mm diameter sand drains at an interval of 3.5m center to center. The depth of sand drains was 30 to 35m from ground level, and were meant to reach at least below the lower boundary of the Holocene alluvial clay layer. The standard penetration N-value of the soft alluvial clay was found to increase to 4 to 10 after improvement, compared to a range of 0 to 5, as shown inside the parenthesis in Fig. 4, before improvement. Similar trend was noted concerning other physical and mechanical properties.

### 3.3 Fines Content Across Depth from Sieve Analysis

Fines content, defined as the percent passing the 75 micron sieve, is an important physical characteristics of soil. A plot of the distribution of fines content across depth shown in Fig. 5 clearly illustrates the nature of soil layering. The fill material (F), Holocene alluvial clay (Ac), diurnal alternate layering (D241), diurnal clay (Dc1) etc. are distinctly demarcated by the distribution of fines content in Fig. 5.

Another important point to be noted from Fig. 5 is that the fill material has a fines content of about 20% that is fairly uniform across the depth. This is considered to contribute to better resistance to liquefaction during ground shaking. It is reported (Ishihara et al, 1996) that the **nanzado** has much smaller fines content in comparison. This difference in fines content may have contributed to the larger extent of soil liquefaction evidenced by ejection of sand and water (Yasuda et al, 1996) in Port island compared to that in Rokko island.

### 4. EVALUATION OF GROUND SETTLEMENT

As also noted above, the Holocene alluvial clay layer under the project site at Rokko island has a thickness averaging about 10 to 13 m. The soft clay layer is overlain by a 24 to 27m thick fill material. Under the situation, the ground settlement is of obvious concern, specially at locations where the ground is not improved. The extent of consolidation settlement, as improved as well as unimproved areas, was estimated based on the results of consolidation tests.

Fig. 6 shows the dependence of the settlement S normalized by the thickness of the clay layer H on the average natural water content Ws of the clay layer. A fairly good correlation can be noted. The open circles in Fig. 6 correspond to improved ground locations, while black ones correspond to unimproved ground.
It is clear that the residual consolidation settlement yet to occur is much larger in unimproved ground than in improved ground. The residual settlement was mostly in the range of 10 to 40cm in the improved ground, while it varied from 60 to 260cm in unimproved ground. Primary reason for the variation in S from point to point is the variation in H itself.

The residual settlement was noted to be small and generally uniform in blocks B1 to B4 (Fig. 3). There was larger variation in case of the blocks on the west side, owing to the variation in H. However, considering the stiffness of the very thick fill layer, large undulation in the resulting surface settlement was not expected even at locations of substantial variation in S at relatively close proximity. This was found to be the case from observation and monitoring.

4.1 Change in Residual Settlement in Three Years

As noted above, various investigations for the site characterization were carried out over a period of four years. In this process, investigations for residual settlement evaluation were carried out twice in blocks A2 and B2 with an intervening period of three years. The results are compared in Table 1, where the results based on investigation made in April 1988 and April 1991 are compared. The trend of decreasing residual settlement is clearly illustrated.

4.4 Long Term Monitoring of Settlement

A dense array system was installed in blocks A5, A6, B4 and B5 for long term monitoring of the surface settlement. In addition, some vertical array were installed for evaluation of the distribution over depth of the time-dependent settlement observed at the surface. The progress of the consolidation settlement of the alluvial clay layer being continuously monitored since June 1988 is shown in Fig. 7, where the vertical dotted line indicated the date of occurrence of the Hyogoken-Nambu earthquake. No significant change in the trend of the settlement progress due to the earthquake can be noted. The detailed analysis of the effect of earthquake shaking on the total ground settlement and its distribution into different layers is being published elsewhere.

The total settlement during the first three year of the monitoring can be found to be 45 to 130cm from Fig. 7, indicating significant variation depending on the location. The difference in computed residual settlement S between the three year interval given in Table 1 varies from about 9cm (North side of B2) to 71cm (central part of A2) indicating similar large variation depending on the location.

5 DYNAMIC CHARACTERISTICS

Considering the apparent seismic hazard of the area, particular attention was given to investigations for evaluating the dynamic characteristics of the site. Extensive field and laboratory tests were undertaken for this purpose, of which the P-S logging, microtremor measurement and dynamic deformation tests were of primary consideration. Results of P-S
logging and dynamic deformation tests are briefly described in what follows.

5.1 P-S Logging for Vs and Vp profiles

The tests for evaluating the profiles of S-wave velocity Vs and the P-wave velocity Vp were carried out at seven locations, one each in blocks A1 to A4, B1 and B2. Most of the tests were carried out to a depth of 110m, and both P and S waves were measured at 1m interval. The results of the investigation are summarized in Table 2, where the layer designations (P, A, etc.) correspond to those in Figs. 4 and 5.

It can be noted in Table 2 that both Vs and Vp change at about 5m depth in the fill layer. The P-wave velocity Vp of fill material within the 5m depth is much smaller than that of water (Vp of water is 1500m/s). This is an indication that the free water surface at the site is about 5m deep. The S-wave velocity Vs of 330 to 350m/s of the fill material deeper than 5m is comparable to the dilatation layers underlying the soft clay layer, indicating a well compacted material.

From the Vs profile of the ground thus obtained, the elastic amplification characteristics of the ground was analyzed based on the theory of multiple reflection of SH waves. The computed amplification characteristics at ground level relative to a depth of about GL-98m for block B2 is given in Fig. 8 as a typical case. From such analysis, it was seen that the fundamental period of the ground is about 1.2 to 1.5 seconds. This range of the period was also confirmed by microtremor measurements as well, and is considered a reasonable value for similar reclaimed sites in Japan.

Fig. 9 shows the response spectra of the two horizontal components of the free field motion recorded at the project site during the Hyogoken-Nanbu earthquake. Comparison with Figs. 8 shows some interesting resemblance of the amplification characteristics, specially with respect to the EW component.

The largest peak of the response spectra of the EW component in Fig. 9 is seen to occur at around 1.7 seconds, and there is a distinct peak around 2.0 seconds in case of the NS component as well. Considering that the ground period is likely to elongate during strong shaking (Karkee et al., 1993), the peaks appearing around 1.7 to 2.0 seconds in the response spectra in Fig. 9 may correspond to the fundamental ground period during strong shaking. Further investigation would be needed to confirm this.

5.2 Dynamic Deformation Test

The dynamic deformation tests were conducted to evaluate the strain dependence of the shear modulus.
6. LIQUEFACTION POTENTIAL EVALUATION

The fill material at the site is well graded with fines content of 10 to 30%, sand content of 40 to 50% and gravel content of 30 to 50%. With the S-wave velocity in the range of 330 to 450m/s, the material may be considered resistant to liquefaction under normal circumstances. Evaluation for liquefaction potential was carried out as per the Japanese practice considering a magnitude 7.5 earthquake and a 200cm/s peak acceleration. The practice is based on the method of Tokimatsu and Yoshimi (1983). It was concluded that there was no danger of large scale liquefaction. The extent of liquefaction at the project area during the Hyogoken-Nambu earthquake was only minor, although there was extensive damage to quay walls presumably due to lateral spreading due to liquefaction.

7. CONCLUDING REMARKS

Site characterization efforts for the city core development project in Rokko Island consisted of extensive site investigations lasting over a period of four years. Hyogoken-Nambu earthquake provided a test for the validity of the findings and evaluations based on these detailed investigations. The experience gained from the project is considered valuable for site characterization in similar situations elsewhere.

REFERENCES


SimSite: Computer model to simulate site investigation for groundwater remediation

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ABSTRACT: This paper presents work currently being done on the development of a computer program that simulates a site investigation for groundwater remediation. The program will present users with a contaminated site and provide them with some common tools used for site investigations. The layout of the user interface and available options are presented. Learning modules have been developed to teach basic skills required to conduct a site investigation, and to guide the user through the simulation. An example is presented to illustrate how the computer simulation can be used to improve user skills with investigation strategies, data interpretation, and dealing with uncertainty.

1 INTRODUCTION

The uncertainty and complexity in groundwater flow systems and contaminant sources make environmental restoration one of the greatest challenges facing geotechnical engineers today (Figure 1). Groundwater flow systems and the type and extent of contamination are complex and difficult to define accurately. Site investigations can help in reducing uncertainty about subsurface conditions. All of this uncertainty, however, cannot be eliminated due to technical constraints and resource (i.e., cost and time) constraints.

In current practice, only $1 of benefit is being achieved for every $3 invested in environmental restoration (ENR 1995). Much of this money has been spent in an unsuccessful attempt to eliminate uncertainty in subsurface conditions. In order to maximize the value of a site investigation, the engineer needs to understand what can and cannot be accomplished in reducing uncertainty for a given level of effort. While theoretical tools are available to aid in the design and analysis of investigation programs (e.g., Gilbert and McGrath 1997), practice and experience are ultimately required.

In order to provide students and practitioners with an opportunity to practice and gain experience in site investigation, a computer simulation package, SimSite, is being developed. This package will simulate sites with groundwater contamination that can be investigated by the user. Through working with these simulated sites, it is hoped that users will gain experience with the effort required to reduce uncertainty in subsurface conditions without wasting real money.

2 OBJECTIVES

The primary goal of the SimSite package is to educate future practitioners in performing and interpreting site investigations. We have identified four specific objectives of the simulation experience: (1) develop a model for the site and be able to compare that model with the actual site conditions; (2) conduct the investigation with limited resources including money and time; (3) deal with uncertainty and variability just as in real investigations; and (4) be exposed to some of the common technology used in site investigations. A secondary goal is to aid in the development of better investigation strategies.

A problem-based learning approach is being used to cover both the broad purpose of site investigations (a decision to remediate or not) and the basic skills necessary to perform them (site characterization). Learning modules have been developed to guide the user through the simulation. The purpose of these modules is to highlight techniques and skills pertinent to conducting an effective site investigation. At the end of each simulation, users will be able to compare interpreted with actual site conditions to learn what they could and could not get from the site investigation.
3 THE SIMSITE PACKAGE

The SimSite package is comprised of two components; (1) the simulation program that generates a realistic site for the user to investigate and (2) the learning modules that guide users through the investigation process. The simulation program is a Windows-based computer program written with Microsoft Visual C++. The user interface and some of the more important site investigation options are shown in Figure 2.

3.1 Simulation Program

The simulation program is designed to provide a realistic experience for users. Realism is provided through the use of actual site data from a contaminated site located along the Gulf coast. These data are being used to provide realism through (1) variability and heterogeneity in the subsurface, (2) complexity in the source and transport of contaminants, and (3) a record of cost and time.

The geologic setting is being simulated with the use of random field models calibrated with real data. A probabilistic geologic model including subsurface stratigraphy, heterogeneities, and soil type is being used to render the site. Properties within soil units, such as hydraulic conductivity, are being simulated with three-dimensional random fields.

Groundwater flow and contaminant transport is being modeled using the SWIFT-II model (Reeves 1986). SWIFT-II is a fully-Transient, three-dimensional, finite-difference code that solves the coupled equations for flow and transport in porous and fractured media. This code will not be run within the program, but will be used to generate data files SimSite will use to model the site. The use of simpler models, such as MODFLOW and the Horizontal Plane Source model (Galyn 1987), are being used for run-time modeling of local conditions.

The geologic and transport models are calibrated with measured data from the site using a record moment Bayesian method (Mutchard 1997). The features of this method are: (1) it reproduces measured data from the site exactly; (2) it uses the geologic and transport models to interpolate between data in time and space; (3) it accounts for measurement and modeling errors through the use of random field models; and (4) it calibrates the parameters describing the geologic, transport, and random field models to the measured data.

Just as in actual site investigations, the investigation requires money and time. Projected costs for prospective actions and the actual costs incurred during the simulation are reported to the users as they conduct their investigation. Cost tracking provides the capability to require users to work within a budget. The actual costs charged to
Figure 2. Structure of SimSite program
users are based on current practice costs as reported by local firms. The progression of time between investigations is controlled by the user.

At the end of the exercise, all information about the site at any given time is made available to the user to evaluate the effectiveness of the investigation strategy, a luxury not available with real sites. The users are able to visualize graphically the contaminant plume at any point in time and are able to compare actual site conditions with their site model developed from the investigation.

3.2 Learning Modules

The complexity achieved in the simulation program allows for various types and sources of uncertainty to be represented when the user conducts a site investigation. To ensure that users recognize and deal with the uncertainty and open-ended problems presented by site investigations, a series of learning modules have been developed in conjunction with the simulation. Modules are being designed and tested in hard copy form.

The modules are a source of guidance through the simulation program. They will help maximize the use of the program by students unfamiliar with site investigation work. Each module addresses a different task or technique related to environmental investigation work. Current modules include:

- Potentiometric Surface and Direction of Groundwater Flow
- Hydraulic Conductivity
- Rate of Groundwater Flow
- Location of Contaminant Sources
- Contaminant Concentration in Groundwater
- Monitoring Well Network Design
- Soil Vapor

Module selection and use will be at the discretion of the user, providing flexibility to accommodate a range of skills possessed by simulation users.

4 EXAMPLE

One of the most important and difficult tasks in a site investigation is locating all of the sources of groundwater contamination. A simple example showing how a simulation user may undertake this task demonstrates both the difficulty of the task and the value of the SimSite package.

An actual site located along the Gulf coast is used for this example. A 7.5 acre unlined lagoon was created by dumping various industrial residues into an abandoned sandpit. The water table lies between 1.2 and 3.7 meters below the ground surface. Dense non-aqueous phase liquids (DNAPLs) are suspected of migrating from the lagoon and serving as additional sources of groundwater contamination. Based on previous groundwater monitoring data, these DNAPL pool(s) are most likely located just east of the lagoon. It is in this area that additional groundwater monitoring will be undertaken to better locate this source(s).

The typical approach to detect groundwater contamination is to install monitoring wells. For this example, nine monitoring wells are installed in a regular grid pattern [Figure 3(b)]. From a preliminary site assessment, 1,2 DCA has been chosen as a contaminant indicator for this site. Three quarterly groundwater samples have been taken from each well and analyzed for 1,2 DCA [Figure 3(b)].

A typical approach to analyzing these results is to average the three measurements for each well and contour these values to better define where sources of contamination may be located (Figure 4). From this map, the user would suspect two sources of contamination: one large source located around well MW-6 and one smaller source centered around well MW-7.

In this actual site investigation, this interpreted model can rarely be checked. Using SimSite, users can compare the model that they developed for the site based on their investigation program with the actual site conditions at any point in time (Figure 5). The effectiveness of the investigation strategy can then be measured by the accuracy of the model that is developed and the total cost incurred. In this example, the user would have found that there were actually three contaminant sources at the site (sources A, B, and C in Figure 5). Further, the two sources suspected from the site investigation (sources A and B) are not appreciably different in size. If the engineer had decided not to investigate further, one source would have been missed completely (source C) while the size of one source (source A) would have been underestimated.

The SimSite package will help users understand the limitations and the associated uncertainty of their investigation strategy. This uncertainty exists due to the spatial and temporal variability of the sources, measurement errors, well locations, screen lengths, and the geological complexity of the subsurface. Users will also get experience with what they can and cannot learn from their investigation.

For example, the data gathered from the monitoring plan [Figure 3(c)] provides detailed information about the presence of DNAPL pools below the monitoring well screen (source C). Other deficiencies in the plan are that the concentration contours were derived without considering the physics of the problem and without considering the reliability of the concentration data. The plan is limited by the geometry and number of monitoring wells and the number of samples taken from each well. It is hoped that SimSite will give users experience in learning how to interpret groundwater
Figure 3. Example site investigation (a) groundwater monitoring plan (b) results

Figure 4. Inferred Contaminant Plume

Figure 5. Actual Site Conditions in July, 1997
monitoring results and recognize the limitations in the interpretations.

A secondary goal of the SimSite package is to help in the development of better investigation strategies. For example, more information could have been obtained for the same level of effort with a different monitoring well geometry (e.g., longer screen lengths). Recent work at the University of Texas at Austin in this area includes Gilbert and McGrath (1997), McGrath et al. (1996), and Muchard (1997).

CONCLUSIONS

Finding a balance between reducing uncertainty and conducting a cost-effective site investigation usually only comes with experience. The SimSite package is being developed to provide a simulated experience for users to help develop these skills without spending real money. The value of the SimSite package lies in its ability to realistically simulate contaminated sites and provide users with a resource for evaluating site investigation strategies for reducing uncertainty.

ACKNOWLEDGMENTS

The authors wish to acknowledge the Texas Advanced Technology Program Grant #400 for providing funding for this project. We wish to thank Dr. Darcy Hardy, Coco Kishi and Susanna Herrdon of the Center for Instructional Technology at the University of Texas at Austin for their assistance. We also want to acknowledge Bernard Briggs whose help with the design and programming of the user interface was invaluable.

REFERENCES

A contribution to the geotechnical characterization of large areas for land planning

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ABSTRACT: The paper presents the results of a study on the geotechnical characterization of two large areas in Southern Italy where a unique geological stratigraphy was recognized. The geotechnical study was aimed at verifying if the mechanical properties of these strata were homogeneous in the whole area. To this aim some zones in the area have been selected and compared, using a statistical procedure based on CPT profiles.

1 MAIN LINES OF THE RESEARCH

The geological and geotechnical characteristics of the subsoil should play an important role in the selection of the location on the territory of large and important constructions (such as urban and industrial developments, large infrastructures for transport). Actually, ideal land planning should consider at least a rough estimate of these characteristics before the feasibility evaluation of the project. The studies on large areas are difficult (due to the extension of the territory to investigate) and not very frequent, therefore they are more related to research than to consulting activity and require the cooperation of multidisciplinary expertise (at least geological and geotechnical). Being these studies based on previously performed investigations, the preliminary collection of data is essential: the geotechnical data concern borings, SPT and CPT, geophysical surveys, laboratory testing, typologies of existing constructions and information on their performance. On the basis of the regional geology, the geologist defines the formations present in the subsoil and locates them in the single stratigraphic profiles. At this point the geotechnical study can begin. The “geotechnical stratum” can be defined as a subsoil region, of initially unknown dimensions, whose physical and mechanical properties are constant, if a deterministic model is considered, or are defined by a certain probability distribution, if a statistical model is introduced. The geotechnical stratum can be coincident with the geological one, or can be a part of it. To this aim, the geotechnical engineer is obliged to follow different criteria, according to subsoil characteristics and available data; on one side, it is appropriate to adopt wide grouping of soils, in order to simplify the model, on the other side, it is necessary to guarantee a sufficient adequacy of the model to the physical reality, nevertheless it is foolish to pretend an accuracy above certain limits related to the considered soil and the available data. Once the geological stratigraphy is defined, each single stratum has to be characterized from the mechanical point of view. The whole process is strongly influenced by the wide scattering of the mechanical properties of natural soils, whose variance has not been yet systematically evaluated. Unfortunately, in most cases, the geotechnical aspects are only considered for the design of single constructions, with obvious negative consequences. The main scope of this paper is to offer a contribution on this subject, reporting some general observations and some recent applications concerning typical situations in Campania (Southern Italy).

2. STATISTICAL CRITERIA FOR THE DEFINITION OF THE STRATIGRAPHY

The CPT profiles, thanks to the continuous information provided and their easy execution, are probably the most useful tests for the definition of soil stratigraphy and the following evaluation of mechanical properties. In the present paper two procedures are proposed for the elaboration of static penetrometer measurements. These tests, as it is well known, do not allow to measure directly the intrinsic properties of soils: the cone penetration resistance (\(q_c\)) is an indirect measure of the undrained strength (\(c_u\)) in cohesive soils and of the friction angle (or relative density, \(D_r\)) in cohesionless soils: it is well-known that it is influenced also by in situ state of stress (\(\sigma, \sigma'\)).
According to the deterministic approach, the stratum of cohesive soil can be defined as a region of the subsoil where undrained cohesion is constant with depth ($q$), if the soil is overconsolidated, or is linearly variable with depth, if the soil is normally consolidated, thus for the cone penetration resistance the following expression can be written:

$$q_c = N_c \cdot C_v + \sigma_v$$

(1)

For strata of cohesionless soils, Lancellotta (1981) proposed the following expression:

$$D_x = A + B \cdot \log \left( \frac{q_c}{(\sigma_v)_y} \right)$$

(2)

where $A$, $B$ and $x$ are constant. In a geotechnical stratum of cohesionless soil, defined as regions where relative density is constant, from expression (2) a linear relation can be derived between the logarithms of $q_c$ and $x$.

Therefore, in all the described cases, the CPT profile can be interpreted through a relation such as:

$$q_c = f(x)$$

(3)

that is varied according to soil nature.

If the statistical approach is adopted, expression (3) must be considered as a regression curve around which the single measures of $q_c$ are more or less dispersed.

In order to verify the planimetric extension of the stratigraphies (that is to establish if the soils in two neighbouring areas can be considered as parts of the same geotechnical stratum) in each profile, each stratum can be substituted by a function of type (3). Anyway, because of the intrinsic variability of soil properties, the coefficients of the regression function will vary within each homogeneous area according to a certain statistic distribution.

Having "$n$" CPT profiles available, a regression analysis can be performed on each of them, through the following phases (procedure 1):

1. each planimetric profile is divided in parts that can coincide with the geological strata or can be part of them, when the cone resistance shows different trends according to a qualitative evaluation;
2. regression curves, selected according to soil nature, are inserted among the data in each part of each vertical and their coefficients are determined;
3. a hypothesis test is performed on the previously defined coefficients to verify their significance, by comparing the scatter of the observed data around the regression curve and the estimated standard error (Hansborg, 1974); in the case of a relevant number of positive results, the chosen function can be considered as fitting well the data and the calculations can be continued;
4. all data related to a stratum (belonging to all the available verticals) are grouped together and the regression curve of type obtained at phase 3 is inserted: the coefficients of the aggregation are thus determined;
5. the significance of these coefficients is verified: if it is positive, the aggregation has been successfully constituted, if not, the aggregation is constituted by different families that should be singled out and separated (level analyser) according to the planimetric distribution of the profiles, in order to select a number of geotechnically homogeneous areas.

6. each vertical, according to its planimetric position, is compared to the family defined for its area, to verify if it is part of it; to this aim a test is performed that verifies if the coefficients obtained for the generic vertical are or not significantly different from those of the family at a level of 5%.

7. the coefficients obtained for areas recognised as geotechnically homogeneous are compared, again with a hypothesis test, to check for the hypothesis of them being different, in this phase it is possible to gather these areas to form wider zones.

8. once the stratigraphy has been defined, each single stratum of a homogeneous area can be characterized by function (3) and its dispersion, from which the strength and deformability characteristics of the soil can be obtained.

As a matter of fact, it is not always possible to define a function that correctly fits the planimetric data, whose parameters' estimators are unbiased (not affected by systematic errors), efficient (with minimum dispersion around average values), consistent (with a trend towards the "true" value with increasing sample dimension). In fact it should be previously verified that the scatter of the observed values around the corresponding computed ones on the regression curve compensate in the average, so that they are casual, not correlated and have constant variance with varying depth.

At present, attention has been focused on the selection of a suitable statistical procedure without performing any of validation test previously described, being the study in a preliminary stage. Unsatisfactory results of the validation tests would necessary imply the use of more complex procedures.

Pellegrino (1994) proposed a procedure similar to the described one (procedure 2), where the stratum is subdivided in stripes of 1m of thickness. Then, grouping the profiles according to their planimetric distribution, geotechnically homogeneous zones are defined and stripes at the same depth are compared, if, for a relevant number of stripes, the average values of $q_c$ are not significantly different, these
zones are grouped to define a new geotechnically homogeneous area, of wider dimensions.
In this way a function like (1) is not inserted within the values of $q_c$ of a given stratum; this procedure thus appears in principle weaker than the first one.

3. SOME CASE HISTORIES

The first case refers to the geotechnical characterisation of the Campanian Plain (Piana Campansà), a wide area of more than 1000 km$^2$, north of Naples, crossed by the Volturno river. This is probably an "extreme" application of the described procedures, since, although the outcropping soils have a more or less common origin and stress history, significantly different local conditions occurred due to the wideness of the area.
The complexity of the stratigraphy was overcome by a simple scheme which considered the essential features of the local differences.
The base formation is everywhere constituted by Campanian Ignimbrite, on which is present a covering stratum of remarkably different soils: dune sands, soft clays and pyroclastic soils (Figures 1 and 2). therefore it appears appropriate to assume a geotechnical stratigraphy coincident with the geological one.
The thickness is however widely variable and some strata are absent in some zones of the area.
A wide amount of data has been collected from public and private institutions, until a total number of 705 investigated verticals (410 boreholes, 160 CPT profiles, 130 SPT profiles) and laboratory tests on more than 100 samples.
As CPT tests' processing is concerned, each profile has been subdivided into strata, according to the previously defined criteria. The part of the profile which falls within a single stratum has been interpreted with a regression curve (Table 1), verifying in how many cases the estimated parameters where significantly different from zero.
Concerning CPT results, it has been showed that in most cases:
a) in clays they can be interpreted with a profile constant with depth (the $\alpha$ coefficient is significantly different from zero in 31 cases over 100);
b) in dune sands they are well fitted by a function like (2), where the coefficients are significant in 72% of the cases;

![Figure 1. Campanian Plain. Analysed area and CPT positions.](image-url)
Table 1 - Campisan Plain. Statistical analysis of the cone penetration resistance: cases when the coefficients of the regression curve are significant

<table>
<thead>
<tr>
<th>Description of stratum</th>
<th>N. of profiles</th>
<th>Regression curve: $q = a + b \cdot \frac{D}{L}$</th>
<th>Significance of $a$ and $b$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clays</td>
<td>79</td>
<td>$q = 2.2 + 2.1 \cdot \frac{D}{L}$</td>
<td>0%</td>
</tr>
<tr>
<td>Dune sands</td>
<td>24</td>
<td>$q = 3.4 + 0.5 \cdot \frac{D}{L}$</td>
<td>54%</td>
</tr>
<tr>
<td>Pyroclastic soils</td>
<td>105</td>
<td>$q = 6.2 + 0.6 \cdot \frac{D}{L}$</td>
<td>67%</td>
</tr>
<tr>
<td>C. Ignimbrite</td>
<td>75</td>
<td>$q = 9.6 + 0.4 \cdot \frac{D}{L}$</td>
<td>59%</td>
</tr>
</tbody>
</table>

In this case, it can be deduced that the different areas singled out in the Piana Campana Plain are not homogeneous only with reference to clayey soils. Similar methodology has been used for another area of smaller extension (about 300 ha), in the Nord of Naples, where 125 CPT profiles were available.

The subsoil is mainly constituted by coalescent pyroclastic soils, partly laying on the Yellow Tuff formation, partly present until a great depth where the Yellow Tuff is absent (Figure 3 and 4).

With the help of the penetrometric profiles, completed by all the information obtainable from stratigraphic boreholes, the typical soils present in the subsoil have been defined. Within the investigated thickness, the geology contributed in the definition of three strata of pyroclastic soils, whose deposition took place in different periods; the geotechnical study concentrated on the intermediate pyroclastic deposit, which has been divided in two strata according to the qualitative evaluation of penetrometric profiles.

With procedure 1, the linear relation happened to fit 63% of the examined cases in the first stratum and 90% in the second stratum (i.e. the slope of the line is significantly different from zero in the same number of cases).

Four zones initially considered homogeneous have been defined, grouping profiles planimetrically close. Then for each zone it has been verified that the linear relation fits the aggregation and that the coefficients of the average line are significantly different from those obtained for the single profiles only in a limited number of cases (Table 2).

In this way it was confirmed that the four zones have been correctly formed.
Table 2 - North Naples zone. Statistical analysis of the cone resistance: cases where the coefficient of the regression line of single profiles is significantly different from the average value calculated for the zone.

With reference to the whole area, the comparison between the coefficients of the regression lines determined for each zone allowed to exclude that they were significantly different, therefore the whole examined area can be considered geotechnically homogeneous.

The second procedure gave the same results, showing that the four zones defined according to the planimetric distribution of profiles are geotechnically homogeneous.

4. CONCLUSIVE REMARKS

On the basis of the described applications, the following conclusions can be drawn:

- procedure 1 is more strict than procedure 2 because it expresses cone penetration resistance as a function of depth, as a matter of fact in the Piana Campana case the two procedures gave different results;

- on the bases of a reasonable number of investigations, a geotechnical characterisation of a large area in the North of Naples was obtained through two functions of type (3), each valid within one of the two strata considered; it should be anyway taken into account that in this second case history not only the extension of the area was considerably less than in the first case, but the density of information is remarkably higher.
REFERENCES


Order in chaos: The geotechnical characterization of melange bimrocks

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ABSTRACT: Geotechnical engineers generally use very conservative strength and deformation parameters when characterizing chaotic rock masses such as melanges and other bimrocks (block-in-matrix rocks). However, the strength and deformation properties of melange rock masses are simply and directly related to their block volumetric proportions. Accordingly, a systematic approach can be taken to characterize bimrocks that allows application of mechanical properties of laboratory test specimens to in-situ melange rock masses, as long as the block volumetric proportion used is the block linear proportion (measured from drill core) suitably adjusted for uncertainty.

1 INTRODUCTION

The orderly and disciplined characterization of rock masses for geotechnical design is a systematic procedure of mapping, surveying, drilling and testing that reduces the rock mass to a collection of names, descriptors, illustrations and other data. The information may then be input into Rock Mass Classifications schemes, which describe adequately the strength and deformation character of relatively well-behaved rock masses such as unweathered granites and massive sandstones. But such orderly rock masses are the exception rather than the norm. Many rock masses are lithologically varied, disrupted and weathered, and their characterization presents major challenges to geotechnical engineers: even rhythmically interbedded sandstones and mudstones are described as "complex formations" (AGI, 1977). But true complexity exists in chaotic rock masses with varied lithologies, heterogeneous mechanical properties, erratic states of weathering, and deranged discontinuity structures. These include melanges (from French: mélange, or mixture), laharls, breccias, saprolites (like decomposed granites), and blocky uniform rock with broad, weak discontinuity fillings.

Diverse geological processes produce these fragmented and mixed rock masses which are described by a rich geological literature of over 200,000 references and a geological lexicon of more than 1000 words (Laznicka, 1988), but few engineering studies. Chaotic rock masses are geotechnically intricate because of their spatial and mechanically variability.

Figure 1: Generalized melange bimrock

The mechanical contrast between generally strong and weak components frustrates good core recoveries, reliable laboratory test results and accurate geotechnical characterizations. Because the standard orderly characterization schemes used to predict strength and deformation properties cannot easily be applied to chaotic rock masses, engineers generally assume that the mechanical and spatial characteristics of the weakest components of the rock masses will govern the rock mass behavior. This assumption is often too conservative.
Furthermore, expensive difficulties can be encountered during construction excavation if the distribution of the stronger components of the rock mass is not considered.

This paper proposes that an orderly approach can be adopted to characterize the strength and deformation properties of chaotic rock masses by using the results of recent research gained by working with the chaotic melanges of the Franciscan Complex (the "Franciscan") in northern California.

2 FRANCISCAN MELANGES

Melanges are disarranged assemblages of relatively strong blocks of rock embedded within weaker matrices of sheared sedimentary and metamorphic rock. Melanges are associated with compressive tectonic margins and have been identified in the mountainous regions of over 70 countries (Medley, 1994a), including much of the western coast ranges of North America.

Blocks in Franciscan melanges are angular to smoothly ellipsoidal with major/minor axis ratios of about 2:1 and range in size between surf and mountains (Figure 1). Large erosion-resistant blocks are conspicuous along the northern California coastline and rivers, and protrude from hillside. Blocks are most commonly composed of graywacke, with rare inclusions of chert, greenstone, serpentinite and exotic metamorphic rocks (Medley, 1994a). Large blocks (tens of meters in largest dimension) tend to be intact relatively except at the margins. Small blocks are often intensely fractured, but if intact may be plucked by hand from the melange. Groundwater is generally found within the larger blocks, which act as discrete tanks, and seeps into the matrix materials.

The matrix of melanges is commonly composed of siltstone, argillite, mudstones, and serpentinite, pervasively sheared to a soil-like consistency. (800 shears per foot have been measured by Savin, 1982.) Shears tend to concentrate around the boundaries of large blocks (Figure 1.) The block/matrix contact is often a thin weathered to a slick clay skin. Because of the weak matrix, earth slump landslides are common in block-poor melanges, whereas debris flows are spawned from large fractured blocks.

Medley (1994a) and Medley and Lindquist (1995) examined geologic maps and photographs and measured the maximum dimensions of blocks within several chaotic Franciscan melanges. Blocks ranged in size between millimeters and kilometers. The block-size distributions were determined to be power-law relationships with negative exponents (fractal).

In soil mechanics parlance, such size distributions are referred to as "well-graded." Block size distributions for blocks at many scales were observed to be very similar in appearance, which indicated that the arrangements of blocks in Franciscan melanges was scale-independent. Thus, order was discovered within the chaos of melanges.

The engineering significance of these results is that regardless of the scale of interest, blocks will always be found within a melange, and should not be ignored. Also, melanges with similar volumetric proportions of blocks will appear similar whatever the scale of interest. A triaxial specimen from a melange rock mass will have the same general appearance as the parent rock mass, given that the volumetric proportion of blocks is similar. A triaxial specimen is thus a scale model of the rock mass, and the mechanical behavior of the triaxial specimen will be similar to the mechanical behavior of the parent rock mass. Although blocks may exist at the scale of the specimen, these same blocks are assigned to the matrix at the much larger scale of the parent rock mass.

Medley and Lindquist (1995) also discovered that within a volume of Franciscan melange there is a continuum of blocks between about 5 percent and 75 percent of the size of the theoretically largest block within the rock mass, nominated "minax." (In a volume of melange, the length of the theoretically largest block, diameter, is also equivalent to VA, where A is the plan area of the rock mass under consideration; however, the largest actual block is generally about 0.7VA.) For engineering purposes, the dimension of the largest block may be substituted by some other characteristic dimension of the volume, such as a tunnel diameter, a triaxial specimen diameter, or the thickness of a landslide.

The results summarized above, although generated from study of Franciscan melanges, are believed to be applicable to other chaotic geological materials, since their block sizes range the scales of engineering interest (centimeters to tens of meters), and conform to negative power law or exponential functions.

3 STRENGTH OF MELANGE BREAKS

Medley (1994a) abbreviated "block-in-matrix rocks" (a term originally proposed by Raymond, 1988) to bimrocks, to indicate that independent of geological origin (or geological name) many mixed and fragmented rock masses have similar engineering behaviors and pose similar design and construction problems. Bimrocks were further defined as "mixtures of rocks composed of geotechnically significant blocks, within bonded matrices of finer texture" (Medley, 1994a).

Geotechnically significant blocks are those blocks that have: 1) a significant mechanical contrast with
the matrix (for instance a ratio of block strength and matrix strength of 2.0 or more); 2) a range in sizes between 5 percent and 75 percent of some characteristic dimension which indicates the scale of engineering interest (dmax, tunnel diameter, etc.); and 3) a block volumetric proportion of between about 25 percent and 75 percent of the rock mass.

The common engineering approach to the characterization of a melange binnock is to assume that it is a "soil with boulders." Thereupon, following the legacy of soil mechanics, the strength of the block/matrix mixture is then taken to be equivalent to the strength of the matrix. This approach is too conservative for chaotic rock masses with block proportions higher than about 25 percent.

Research performed by only a few researchers shows that, beyond a threshold proportion of about 25 percent, the presence of strong inclusions influences the mechanical behavior of block-in-matrix materials. For instance Irfan and Tang (1993) showed that the shear strength of bouldery collium depend on the volumetric proportion of boulders, once the volumetric proportion exceeded about 25 percent (Figure 2.).

Lindquist (1994a,b) and Lindquist and Goodman (1994) showed that the mechanic properties of triaxially compressed specimens (150 mm diameter) of physical model melange and specimens of Franciscan melange collected from Scott Dam (northern California), were simply and directly related to the volumetric block proportion above a threshold of about 25 percent. It was also discovered that the angle of internal friction increased, cohesion decreased, and the modulus of deformation increased as block volumetric proportion increased to about 75 percent volumetric proportion. Beyond the upper limit of block proportion, blocks tend to touch and the materials are no longer block-in-matrix (for example: blocky rock with irregular discontinuity infillings).

The strength of the blocks is irrelevant to the overall strength of a binnock: only the volumetric proportion of blocks is important. It requires only a modest mechanical contrast between blocks and matrix to force failure surfaces or pre-existing shears, to pass continuously around blocks (Figure 1.) (However, during construction, the strength of the blocks is very much a matter of concern to a tunnel excavation contractor.) The greater the number of blocks, the greater the tortuosity and the higher the angle of internal friction. Fractal block size distributions, with a few large blocks and many smaller blocks, result in greater tortuosity.

Lindquist (1994b) combined the results of Irfan and Tang (1993), his physical model data and results from testing real melange at Scott Dam, to demonstrate relationships between volumetric block proportion and incremental angles of internal friction (Δψ, additive to the friction angle for the matrix materials). The relationship is valid for melanges with volumetric proportions between 25 percent and 75 percent. As shown for the "conservative trend" in
Figure 2 the total increase in friction angle can be as much as 14 degrees, for a melanite with 75 percent block volumetric proportion. On the other hand, higher block proportions result in greater surface areas of block/matrix contacts, which are the weakest zones within bimorhok. Consequently, cohesion declines with increasing block volumetric proportion (Lindquist 1994b.) An increase in the proportion of blocks increases the stiffness of the bimorhok, which reduces deformation. However, the orientation of blocks to loading also has an influence on deformations: blocks aligned parallel to axial loading results in reduced deformation. The strength of the matrix at rock mass scale can be determined using laboratory specimens which will contain blocks (because of the scale independence of block-size distributions for melanite bimorhoks.) It is geotechnically conservative to select specimens that lack larger blocks. But recovery of laboratory specimens by core drilling is neither standard practice nor trivial: efforts may be more successful if 150 mm diameter core is drilled. Neither is it common practice to perform geotechnical tests on specimens containing large inclusions, so new procedures have had to be devised to perform the tests and interpret the data (Lindquist, 1994b; Professor Richard E. Goodman and Dr. Anders Bro, personal communications.) The results of the work summarized above can probably be extended to other bimorhoks such as decomposed granities and breccias, based on the assumption that the block size distributions are both fractal and scale-independent over the range of scales of engineering interest (centimeters to tens of meters.)

4 ESTIMATING BLOCK PROPORTIONS

The practical use of the findings summarized above requires that the engineer estimate the block volumetric proportion (block proportion) of the bimorhok being characterized. Medley (1994a,b) and Medley and Goodman (1994) proposed that the volumetric proportion of blocks in a bimorhok could be assumed to be equivalent to the linear proportion of blocks measured from scans of such drill core. Although there is a well-established stereological basis to the assumption that linear and volumetric proportions are equivalent (Underwood, 1970) littleprecedence existed in the geotechnical engineering literature for demonstrating the equivalency prior to recent work by Medley (1994a, b; 1997) and Medley and Goodman (1994.)

The linear proportion of blocks is the total length of intersections between the blocks and the core, divided by the total length of the core. (Figure 1 shows a boring through blocks producing intercepts.) The determination of linear proportion from drill core requires little additional effort to that normally expended in a conventional exploration program. But despite the ease of measuring block linear proportion, it is essential that the range of uncertainty (error) be estimated prior to applying the crucial assumption that the block volumetric proportion is equivalent to the measured block linear proportion. Uncertainty is a function of both the total sampling length (drilling) and the actual block proportions being drilled. The more the block proportion of the melanite, and the greater the length of drilling, the more intersection data which are recovered. Conversely, if the melanite mass has low block proportion, there is reduced geometric probability of intersecting blocks by randomly located boreholes.

Medley (1997) used an empirical approach to estimate the uncertainty in an estimate of the block proportion of Franciscan melanite underlying Scott Dam in northern California. The results were used to support the selection of the overall block proportion of the melanite below the foundation of Scott Dam, and thence the strength.

Physical models were fabricated with block volumetric proportions that ranged between 13 percent and 55 percent (which bracketed the estimate of block proportion for the melanite below Scott Dam.) Each model was sampled by 100 model boreholes (about 10 cm deep) and the lengths of the block/borehole intersections were measured. Individual boreholes yielded extremely different linear proportions, but the total linear proportion from all 100 boreholes closely matched the known block volumetric proportion for each model.

![Figure 3: Uncertainty in estimates of volumetric block proportion using linear proportions (based on physical models ranging between 13% and 55% block proportion by Medley, 1997.)](image-url)
validating the stereological principle that given enough sampling, the linear proportion is equivalent to the volumetric proportion.

The effect of drilling with fewer boreholes (less sampling length) was examined by randomly selecting data from the exhaustive data set of 100 boreholes per model. Total sampling (drilling) lengths were expressed as multiples of dmax for each model. Uncertainty was defined as the ratio of the statistical deviation between the measured linear proportion and the known volumetric proportion, divided by the known volumetric proportion. Figure 3 schematically shows uncertainty of block proportions between 13 percent and 55 percent. Uncertainty may be positive or negative but in prudent practice the volumetric proportion should be calculated by reducing the linear proportion by an amount equal to the uncertainty ratio multiplied by the linear proportion.

In preliminary work, Medley (1994a) recommended that a total length of drilling equivalent to at least 10dmax was necessary to adequately estimate volumetric block proportion. However, the results of Medley (1997) show that to be certain that the true volumetric block proportion is the same as the measured linear proportion, the total sampling length needs to be between about 20dmax (for a bimrock with 55 percent block proportion) to more than 100dmax (for a melange with about 30 percent block proportion).

Most drilling programs will not recover anywhere near as much block/core intersection as required to reduce the uncertainty to zero. Consequently, whenever drilling costs must be minimized, uncertainty in the estimate of volumetric proportion will be significant. Indeed, where the uncertainty is prudently applied to melanges with low block proportions, the corrected block proportion may fall below 25 percent. In that case, it would be appropriate to select the strength of the matrix as representative of the entire bimrock mass. Yet the effort expended in drilling is not wasted: the data can be used to construct an approximate block size distribution which may provide useful data to Owners and Contractors intending to excavate the bimrock (Medley and Lindquint, 1995).

5 EXAMPLE CHARACTERIZATION

An example, based on experience at Scott Dam, suggests how to characterize a bimrock in order to determine the strength and deformation properties.

Assume that a real melange underlies a proposed site with a plan area (A) of 10,000 square meters. The size of the largest possible block (dmax) will thus be 10m (V/A), but more likely will be about 70m (based on the most probable largest block being about 0.7V/A.) Assume that an exploration program recovers 700m of core, equivalent to 10dmax. Linear proportions have also been estimated from outcrop scanlines. The linear proportion of blocks measured from the core and outcrops is 40 percent. Laboratory testing is performed on core samples containing a variety of block proportions, and indicates that the angle of internal friction of the melange matrix is 25 degrees for block proportions below 25 percent. For block proportions greater than 25 percent, there are measurable increments to bimrock strength similar to the relationships graphed in Figure 2.

To determine the uncertainty associated with assuming that the linear proportion is equivalent to the volumetric proportions, the results of Medley (1997) indicate that the uncertainty will be about +/- 0.12, or about +/- 5 percent (0.12*0.60). Prudence dictates that the lower bound be selected, or that the volumetric proportion be selected as about 35 percent (40 percent times 5 percent uncertainty.) Using the laboratory data (which for the sake of the example can be taken as the "conservative trend" shown in Figure 2), the incremental strength of the melange (assuming no cohesion) will thus be about 3.5 degrees. Hence the total melange strength will be 28.5 degrees.

6 OTHER BIMROCK OBSERVATIONS

While determination of the block volumetric proportion is critical for the accurate characterization of bimrocks, other observations may be made during characterizations, such as the following:

- Extensive fracturing of blocks may deflect potential failure surfaces within the rock mass, and should be mapped and recorded. However, a predominant shear or fracture fabric in the weaker matrix is more likely to control failure surfaces through the rock mass.
- Blocks smaller than 0.03dmax are assigned to matrix. But at the scale of an engineering site, these "denoted blocks" may still be of substantial size, and where encountered should be measured. The fractal nature of block size distributions in bimrocks means that such blocks will be abundant during excavations or tunneling and may require special handling.

7 CONCLUSIONS

The results of recent work with melange bimrocks demonstrate that the strength and deformation properties of chaotic rocks can be characterized as systematically as is executed for more tractable rock
mazes. There are advantages to approaching the characterization of chaotic rocks in an orderly manner, rather than assuming that the entire rock mass has the same character as the weakest component. If the procedures described here are followed, geotechnical advantages can be won in regard to mechanical properties to be used in design. Additionally, pre-construction foresight into block volumetric proportions, block size distributions and block lithologies can be converted into economic advantages when negotiating earthwork construction contracts.

ACKNOWLEDGMENTS

The work described here was partly funded by Pacific Gas & Electric Co., San Francisco; and in part was supervised by Professor Richard Goodman, University of California, Berkeley.

REFERENCES


Medley, E., 1994a: The engineering characterization of melanges and similar block-in-matrix rocks (bimrocks): Dept. of Civil Engineering, University of California at Berkeley, California; PhD diss.; 387 p.


Raymond, A., 1984, Classification of melanges in Melanges: Their nature, origin and significance, SP 228, Geol' Soc of America, Boulder CO; p. 7-20.


Geology and engineering geology in the siting of complex public facilities

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ABSTRACT: The site selection process for large, complex public facilities such as nuclear power plants, low-level radioactive waste disposal facilities or a superconducting super collider, require many technical disciplines. In these cases it is often advantageous to use geologists and engineering geologists. Because of the different career experiences of these otherwise closely related disciplines, it can critical on a fast track project that definition of common terms be defined and understood as early as possible.

1.0 BACKGROUND

Site selection can take many forms. It can be controlled by such nontechnical issues (at least in the context of this forum) as the availability of land or the potential demand for a product or service in a particular area. It can be controlled by issues of social engineering. Or, it can based on technical criteria such as geology, hydrology, biology and environmental and engineering conditions.

This paper concentrates on those portions of the site selection process that involves these technical criteria, though both of the siting examples I use included many so-called non-technical criteria.

1.1 Factors in siting large complex public facilities

When a project involves a large complex public facility, many technical disciplines typically are employed during the siting phase for various reasons. Sometimes it is for the money that may be saved by identifying and addressing potentially critical issues that could later impact construction issues and sometimes it is to meet various regulatory requirements. Sometimes it is demanded by schedule or other requirements of the project.

1.2 Factors influencing the decision to use siting specialists

Solving potential problems at the beginning of the project can save a great amount of money. Delays due to design changes can be far more costly when a project is at full staff than at its early stages.

This almost always was the case with nuclear power plants and remains the case with much smaller projects that are based on "just-in-time" deliveries.

Regulations may require that certain specialist tasks be performed as part of the license application. Resolving these issues during the site selection and characterization process can streamline licensing. The Central Interstate Compact low-level radioactive waste disposal project in Nebraska, is an example of a project where the siting selection and characterization process was driven by licensing requirements. At times, selecting and characterizing a site is controlled by schedule. This was the case when the U.S. federal government announced that it would entertain bids for a state to host the Superconducting Super Collider site. Most states decided that selecting a site using a process with technical rigor enhanced their chances of being selected to host this high tech facility, since there was little time to do anything more than site selection.

1.3 Use of geologists and engineering geologists in site characterization

Typically, the siting and characterization of large, complex public facilities such as high level radioactive waste facilities or nuclear power plants or similar high tech facilities use many technical disciplines during the site selection and
2.0 INTRODUCTION

The role of geologists in many site characterizations is to provide a basic understanding of the framework of the region and site. Working with that framework, the engineering geologists' and geotechnical engineers' role is to apply that interpretation to the planned facility's engineering requirements by quantifying those engineering properties of the site needed to develop and execute the engineering design.

This division of labor generally works very well, but in my experience when geologists, engineering geologists and engineers interact, they each approach some issues very differently. For example, geologists are inclined to solve a geology problem as completely as possible whereas the engineering geologist and engineer look for bounding solutions. Another example is that geologists are use to dealing with natural resources as an asset, while an engineering geologists can sometimes view it as a liability, particularly in site suitability studies.

2.1 Geologists roles in characterization

Geologists evaluate the regional geology as a first step of selecting a site. From this knowledge they can identify features that make a site more or less suited for a particular feasibility.

They evaluate the potential for seismic shaking. They identify potential geologic hazards, such as areas with the potential for landslides, rapid headward erosion or sink holes. They identify areas that can benefit from good drainage or from the presence of materials that can retard contaminant migration.

Where the project requires a license, such as nuclear power plant or a hazardous waste facility, this regional information can be the basis for ferreting out the most important aspects of licensibility (McCure and Hatheway, 1979).

2.2 Engineering geologists roles

Engineering geologists evaluate and quantify sites with rock exposed at the surface. They quantify rock properties and provide foundation designs. They also evaluate the potential impact of regional seismology and ground water on the site and help the engineer apply this to the facility design.

Engineering geologists typically are concerned with foundation construction. They direct line drilling and presplitting operations associated with rock blasting. They design and direct rock improvement operations, including rock bolting and grouting. They evaluate drilled pier and caisson construction.

2.3 Geotechnical engineers roles

In a similar fashion, sites with soil exposed at the surface are evaluated by geotechnical engineers by quantifying the soils properties of the site.

3.0 SUPERCONDUCTING SUPER COLLIDER SITING IN OKLAHOMA

In April 1987 the U.S. Department of Energy (DOE) invited states to submit a proposal for the land necessary to build and operate the superconducting super collider. This technically advanced, complex, public facility was mainly to consist of a ten foot diameter tunnel laid out in an oval with a circumference of 53 miles.

Oklahoma prepared a detailed proposal led by the governor’s office. The technical content of the proposal was prepared by a team that included the Oklahoma Geological Survey and a consortium of companies, that included DMMI, Bechtel, the Williams Company and the Benham Group. Oklahoma Geological Survey geologists, with engineering geology assistance from Bechtel, did a state wide evaluation based on geology and
engineering geology criteria that led to the selection of a site near Kingfisher, Ok about 35 miles northwest of Oklahoma City. A parallel evaluation of potential sites using non-geotechnical criteria also ranked Kingfisher as the preferred site.

There were several complications associated with this unique project because of its size and complexity. For example, the Oklahoma team soon realized that because the oval tunnel was of such a large circumference, the curvature of the earth needed to be taken into account in the design. This meant that the tunnel either would have to be bent or be deep enough to insure that the tunnel not daylight at either end of its long axis. However, if it was decided to bend the tunnel, it had to be done in a particular way so as not interfere with the particle accelerator’s operations.

The work was started in late April 1987, when the governor directed the Oklahoma Geological Survey to select a suitable site for the facility. Since the proposal was due in early September, 1987, there was no time for any site characterization. If the DOE had selected the Oklahoma site, an extensive characterization would still have been necessary. However, the DOE selected a Texas site near Waxahachie about 25 miles south of Dallas. Nevertheless, the siting process did provide useful examples of some of the differences in how geologists and engineering geologists approach some of the issues that can arise in a site selection process.

3.1 Site selection process

One of the main constraints to the site selection process was the short time to do the work. As a result, it was decided that full advantage needed to be taken of the geological survey staff’s detailed knowledge of Oklahoma geology.

3.1.1 Initial geological exclusion criteria

An early set of exclusion criteria were established as the first step in the siting process which eliminated areas with complex geologic terrain, areas with bedrock or alluvial aquifers, (including recharge areas), areas overlain by major alluvial deposits and areas with excessive relief. Thirty candidate sites in 13 study zones were identified.

3.1.2 Second phase engineering geology and geology exclusion criteria

A refined screening was done on these 30 sites using three additional exclusion criteria that combined both engineering geology and geology. The criteria were based on the proximity to evaporites and carbonates, the proximity to active faults and the proximity to natural resources, particularly oil and natural gas. This screening eliminated 12 candidate sites.

3.1.3 Third phase preference screening criteria based on socioeconomic factors

The remaining 18 sites were then subjected to a socioeconomic screening which considered such factors as proximity to universities with strong physics departments, access to a major airport and other transportation facilities and proximity to strong support services and housing. Twelve sites were selected.

3.1.4 Fourth phase screening using preference criteria based on geology and engineering geology

The remaining 12 sites were subjected to a more detailed analysis using three site-selection matrices. A comparison of the lithology, geologic structure, hydrology, natural resources, seismicity, topography and relief at the 12 sites was made in each of these matrices. Each site was ranked either excellent, good or fair for each of these characteristics. The first matrix screening identified seven preferred sites. These seven preferred sites were all subjected to a second matrix screening using the same preference criteria, but with additional refinements. This screening yielded candidate five sites.

A third matrix screening was performed using the same criteria, but as weighted averages. The greatest weighting was given to topography and lithology. The second greatest weighting was given to structure, hydrology, the absence of fuel resources, and relief. The lowest weighting was given to seismology. A preferred site near Kingfisher, Ok, was selected based on this final screening.

At this stage, the top four sites that were selected using geology and engineering geology screening criteria were again subjected to a screening using socioeconomic criteria. This also resulted in the
selection of the site near Kingfisher, Ok. as the preferred site.

3.2 Contrasts between the approach to screening by geologists and engineering geologists

In these final phases of screening once again the knowledge of Oklahoma geology by the survey's geologists was essential in using the preference criteria. For example, the petroleum geologists were particularly important in the assessment of potential interference from oil and gas resources. And as the final candidate site was evaluated for potential problems in orienting the accelerator footprint, the petroleum geologists' knowledge was essential in selecting an orientation that minimized the potential impact from leaking oil or natural gas.

But, it was also this issue that brought out the differences in how a geologist, particularly an oil and gas geologist, approaches a siting problem compared with an engineering geologist. Oil and gas are assets to a geologist, but a liability to an engineering geologist.

3.2.1 Oil and gas as asset or liability

As final documents were being prepared, thinking of oil and gas as an asset constantly intruded into the descriptions of it as a liability. The geologist who was describing why the selected site avoided the risk of leaking oil and gas, had to struggle to not use the positive language of assets.

It was discovered that as the licence documents were being prepared we had to constantly remind and check ourselves on the language we were using when discussing oil and gas reserves. When we discussed exclusion criteria it was relatively easy to identify potential reserves and then just give the areas containing those reserves a low rating.

However, when it came to the consideration of the potential of migration of oil and gas into the accelerator tunnel, it became much harder to use language oriented to oil and gas as a liability. Our petroleum geologist would first describe the "oil and gas prospects" of a stratigraphic unit or an area and then have to rethink and describe these as having "oil and gas contamination potential." This was not as easy as it might seem after a career regarding oil and gas as assets.

Another problem involved the concept of potential reserves. When we described a field as having depleted reserves, we had to remind ourselves that the reserve concept is an economic one that loses a bit of sight of the absolute amount of oil or gas in the ground. If the price of oil or gas triples, the reserves increase. So, it was the amount of oil and gas that could potentially interfere with accelerator operations that was critical to the evaluation, not the amount of reserves.

3.2.2 Well abandonment standards

Also, as sites were being evaluated in the last three phases of the refined screening, we discovered that what the geologists working in the oil patch considered a satisfactory procedures for abandoning wells didn't meet established guidelines for environmental abandonment of oil wells. The most significant differences concerned both the methods and documentation of abandonment.

A capped well in the oil patch is one that does not contaminate potable aquifers. But for planned facility, it would be necessary to either provide records that a capped well was no longer leaking or be prepared to recap the well.

4.0 LOW-LEVEL RADIOACTIVE WASTE DISPOSAL FACILITY SITING IN NEBRASKA

The Central Interstate Compact's low-level radioactive waste disposal facility site selection and characterization process in Nebraska was extensive. It was based on a rigorous defined set of criteria. All of the early phases of site selection were based on well defined criteria that were mostly technical, but included a few non-technical criteria such as avoidance of highly populated areas and of certain designated parks and preserves. The three criteria given the ninety percent of the weighting in the site selection process were related to geology, ground water and surface water hydrology.

For economy of space in this paper it is assumed the reader will refer to another paper in these proceedings (Dugas, 1998) for a description of the siting process. However, it should at least be said that a key feature of the process was that the regulators closely followed the data collection process, particularly during characterization. As a result, when the license was under review, both the applicant and regulator was able to draw on this
common experience when evaluating the data from
their unique points of view.
It also may be important to note that much of the
site selection process was streamlined because of the
availability of extensive geologic resources in the
state geologic survey offices at the University of
Nebraska. These included the extraordinary
repository of geologic maps and logs. The
engineering geologists who did the site selection had
the advantage of being able to discuss these maps
and logs with many of the geologists who had done
the original field work. This helped the engineering
geologists understand some implications that they
otherwise may have missed. In some instances this
became important because of differences in how
geologists and engineering geologist approach data.

4.1 Issue of using bounding solutions

For example, one of the predominant issues that
arose in the characterization and licensing of this
facility was that of using a bounding approach
to solving the problem. Geologists are trained to collect
and interpret data using the method of multiple working
hypothesis. This approach often takes a geologist
many years to collect enough data to develop the
hypotheses and be convinced that a particular
problem or issue has been resolved. Since geologic
data doesn’t lend itself to proofs, geologists often
have to settle for interim “solutions” and depend on
consensus to test these “solutions.” But, geologists
realize that these are really just hypotheses so they
are ever mindful that additional data may lead to a
revised interpretation.

Site selection and characterization does not readily
lend itself to such an indefinite approach, so
engineering geologists have adopted the engineer’s
tool of using bounding solutions. For example, while
a geologist will often want to date a fault as precisely
as possible, an engineering geologists may be content
to know that the fault hasn’t had any displacement
for at least the past 35,000 years or movement of a
recurring nature within the past 500,000 years
(Adair, 1979).

4.2 Bounding approach in low licensing

A recurring issue in the characterization and licensing
of the Nebraska low-level radioactive waste facility
concerned the potential for contaminant migration.
Rather than measuring the properties of the rock and
soil along every potential pathway, engineering
geologists and engineers approached the problem by
establishing the threshold permeability of the shortest
critical flow time of the critical contaminant. Then,
if the rock or soil permeability was at least that low,
it was felt that it was not necessary the exact values
everywhere.

Another recurring issue concerned the calculated
level of contamination at the boundary. Engineering
geologist argued that if the criteria were met at the
boundary they were met everywhere beyond the
boundary. However, some geologists argued that it
was necessary to measure permeability properties of
rock and soil beyond the boundaries.

5.0 SUMMARY AND CONCLUSIONS

In summary, I have discovered that geologists and
engineering geologist treat similar sounding
correlations much differently. I believe it is important to
stay attuned to those differences to insure that the
accepted meaning of the geologists’ terminology
doesn’t hide potential future engineering geology
problems. I have discovered that what is considered
acceptable quality assurance in petroleum geology is
sometimes not even remotely acceptable in
engineering geology. I have discovered that the
correlation of mineral or petroleum reserve is not well
understood by engineering geologists, but needs to
be in some sites and characterization processes.
And, I have discovered that the engineering concept
of bounding solutions that is much used by
engineering geologists is not readily understood by
many geologists.

REFERENCES

Adair, M.J., 1979, Geology evaluation of site for a
nuclear power plant. In Hatheway and McClure (ed.)
Geology in the siting of nuclear power plants: 27-39
Reviews in Engineering Geology, v. IV, Boulder:
Geological Society of America

Luz, K.V., R.E. Migues and others, 1989, Selection
and Geology of Oklahoma’s superconducting super
collider site. Special Publication 89-1, Norman,
Oklahoma Geological Survey

McClure, C.R. Jr. and A.W. Hatheway, 1979, An
overview of nuclear power plant siting and licensing.
In Hatheway and McClure (ed.) Geology in the
Selecting a low-level radioactive waste site for the Central Interstate Compact: Toward a checklist for siting small hazardous waste projects

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ABSTRACT: Selecting a site for a low-level radioactive waste disposal site in Nebraska required an in-depth understanding of many licensing guidance documents and regulations. In some cases this included discussion with document authors to insure understanding their intent. It is planned to include the lessons learned from the Central Interstate Compact low-level radioactive waste disposal licensing effort into a checklist to aid in the siting a broad range of small, hazardous waste projects.

1 BACKGROUND

The purpose of this study is to examine the process used to select the site for a low level radioactive waste facility in the state of Nebraska, for the lessons it might provide on smaller hazardous waste projects.

In particular, these lessons may contribute toward the development of a checklist for siting small hazardous waste disposal facilities. The steps taken in selecting the site are described with an emphasis on those resources that may be useful for developing the checklist. The site was selected for the Central Interstate Compact.

1.1 Siting philosophy

During the first six months of the project, the project team adopted a philosophy that the site selection would be rigorously technical and would be done with each step open to public scrutiny. We were convinced that only in this way, could a project that involved the disposal of radioactive waste be successfully accomplished. While the public on all sides of the issue demands technical rigor, it was also recognized that the disposal of low level radioactive waste in the United States is embedded in an intensely political process.

1.2 Structure of the low-level radioactive waste compacts

The U.S. Congress enacted the Low-Level Radioactive Waste Policy Act of 1980 to encourage states to take responsibility, preferably in groups, of their own waste and amended it 1985 when they saw that there was not much movement. (GAO, 1992). Ten compacts regions were established with nine separate site development efforts underway, though only four have licenses approved or pending (see section 1.4). Ten states are unaffiliated with a compact and have little progress toward licensing a site. (Iger, 1992).

1.3 Central Interstate Compact

Arkansas, Kansas, Louisiana, Oklahoma and Nebraska formed the Central Interstate Compact Commission in 1982 and in June, 1987 elected U.S. Ecology and Bechtel to develop, construct and operate a low disposal facility. Various characteristics, such as geology, of each member states was evaluated, including visits to potential siting regions within all five states and in December, 1987 Nebraska was determined as the most suitable state to host the compact's first facility. In July
1990, the contractors in Nebraska completed their site selection process with the selection of a site in Boyd County. The GAO reviewed this site selection process in 1991 and concluded that the contractors had "...conducted an extensive site selection process to identify three candidate sites and select a preferred site. The process involved a combination of scientific assessments and judgements, subjective public involvement and land availability (Iager and Setlow, 1991)." Iager and Setlow also noted some deficiencies (foot) and these were addressed along with issues raised by others in subsequent studies.

1.4 Status of Compact Licensing

To date, only four states, California, Texas, Nebraska and North Carolina are in varying stages of the licensing process and none of the other compact siting efforts have progressed to the stage of identifying specific candidate sites. In addition, a commercial disposal facility in Utah began, in 1991, to accept a subset of of low-level radioactive waste for which states are responsible. The Utah site is outside the compact structure (Romano, S. and G. Paris, 1996).

2 INTRODUCTION

Early work by a panel for the U. S. National Research Council considered shallow land burial as the preferred method for disposal of low-level radioactive waste since the radioactivity associated with waste was in the soil or airborne (NRC-NOS, 1976) and some compacts, such as the Southwestern Compact, have adopted this approach. However, concern over the ability to at least restrict radioactive leaks to the site, convinced the state of Nebraska that an above ground facility with features to eliminate or at least minimize leaks was preferable to shallow land burial.

2.1 Characteristics of a site well suited for lrrw

There are various views on the specific characteristics that a site should possess to be suitable for deep lrrw disposal. The earliest efforts to identify the characteristics associated with a good lrrw site were based on work then being done by the USGS at the Idaho National Laboratory. These studies emphasized the desirability of finding a site with characteristics that were in balance with each other. Subsequent U. S. federal and Nebraska state guidelines reflect this approach.

For example, a site with a reasonably thick, very low permeability clay will retard the migration of contaminants, but if the entire site has such clay at the surface, all flow including contaminants would stay on the surface and rapidly leave the site. On the other hand, rain would rapidly soak into the subsurface at a site with sand at the surface and to a substantial depth below it. In such a case, contaminants would migrate offsite if the rainfall rates were only moderately high. So, a site that balances the amount of clay and sand (and silt) would allow rainfall to soak into the ground a short ways before being retarded by a clayey zone, or better yet stopped altogether, before migrating to any major aquifers.

Similarly, a site with a small recharge area will minimize the amount of moisture available for ground water recharge, which complicates monitoring efforts. But, a small recharge area also means the potential for surface erosion is significantly reduced. The potential for erosion is further reduced if the site has low relief. However, low relief can result in flooding, though if the recharge area is small that flooding will remain minor.

2.2 Development of the criteria for selecting and evaluating the Nebraska site

While background information was being collected on various aspects of Nebraska geology, hydrology and environmental science, a well documented site selection procedure was prepared with what the GAO called a "methodical and detailed multistep process to develop screening criteria (Iager and Setlow, 1991)." It was felt that a careful documentation of the criteria process before the start of site selection would help insure that the criteria would be applied uniformly on all sites.

These various criteria included, among other things, technical specialists identifying these general characteristics that should be considered in picking a site and then ranked them in their order of importance. These characteristics were then described to both a statewide citizens' advisory committee and to the interested citizens in a series of public meetings and ranked by these groups as a validation of the earlier identification and ranking of criteria by the technical specialists.

Once this was done, the criteria were published and site selection begun.

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3 SITE SELECTION

The site selection process began with the identification broad geographic areas within Nebraska that might be suitable for a site. However, since Nebraska's statutory policy required that a disposal facility be located in a community that wanted to act as the host, the contractor invited cities and counties, at their discretion, to participate in the site selection process. Twenty counties formally expressed interest in the site selection process. Actually, many more than 20 counties expressed informal interest. However, after discussing the criteria of an acceptable site and the geological, hydrological and various other characteristics of their counties with both the contractor's own technical specialists, some of the counties decided not to submit a formal request to participate.

3.1 Identification of potential siting areas

The established siting criteria were used to evaluate specific characteristics in these 20 counties. As the data continued to be evaluated and refined, nine counties withdrew from the siting process, many of them after assessing that their counties did stand a very promising chance of being selected. After further evaluations, 111 potential siting areas were identified spread across the 11 remaining counties.

3.2 Selection of candidate sites

These 111 sites reduced to 27 potential siting areas using more detailed data. Then, the citizens' advisory committee was asked to rank these 27 areas, unidentified by county or location in the state, using the siting criteria they had earlier help define. Their ranking of candidate sites closely matched a ranking done by the technical specialists. Using specific data that continued to be gathered both from the literature and from site visits, three candidate sites, in three counties, were selected. Up to this point, the site specific data had not included any onsite exploration.

3.3 Evaluation of the candidate sites

Once the three candidate sites in Boyd, Nemaha and Nuckolls Counties were announced in January 1989, local monitoring committees were established, as required by state law. All three monitoring committees also elected to hire very experienced geologists to closely follow the field exploration work.

Over a period of approximately one year the three sites were simultaneously studied, lead by three separate senior or supervisory geologists, one at each site. Each site supervisor was supported with at least one full time assistant and up to eight part time specialists. Each of the characterization programs were tailored the specifics of each site.

The extensive technical studies included various geophysical surveys, geologic mapping and subsurface exploration. At least 50 bore holes were drilled at each site, with some of them converted observation holes for hydrogeology studies. Surface water hydrology, climatology, biology, geochemical, environmental science, geotechnical engineering, and other studies were also conducted.

3.4 Selection of the proposed site

Based on these studies Boyd County site was selected as the Central Intersate Compact's first low-level radioactive waste disposal facility. After the Boyd County site was selected, the consulting geologists hired by each of the local monitoring committees were convened by invitation of the state and evaluated these studies. They concluded that the evaluations at each site had been done "...in a technically correct and proficient manner and had reached appropriate conclusions about each site on the basis of the information collected. They also agreed that the work performed provided a sufficient basis on which to select the Boyd County site (Jager and Sletten, 1991)."

Following an additional three months of supplemental studies on the Boyd County (or Butte) site, in part based on questions raised by the local monitoring committee's geologic consultant and issues raised in the GAO report, were completed. A multi-volume license application was submitted to the state in July 1990.

Additional legal and laboratory data have been collected based on issues raised by the state regulators as part of their review of the license application. In addition, refined analyses have been conducted, including various numerical analyses, during the various rounds of questions.
4 SUMMARY OF THE BUTTE SITE CHARACTERISTICS

The site in Boyd County is about 1 1/2 miles west of the village of Butte, near the center of the county. Boyd County is in the northeast corner of the state. South Dakota borders the county on the north and the Niobrara River defines its southern boundary (Figure 1).

The geology of the county and site is simple. Knowledge of the site geology is based on well over 100 geologic borings mostly 30 to 80 feet deep. Several deeper borings are 100 to 200 feet deep and one boring is 500+ feet deep.

4.1 Regional Geology

The geology at the site and in the region are very similar. It consists of up to approximately 40 feet of Quaternary and late Tertiary sediments and sedimentary rock overlying at least 500 feet of late Cretaceous Pierre Shale. The Pierre Shale, which is as close to impermeable barrier as I have encountered in nearly thirty years of engineering geology, is underlain by Niobrara Formation claystone. Care was taken to not drill the 500+ boring through this thick claystone sequence and thereby creating a conduit from the surface into the Dakota Sandstone, which is the first major aquifer under the site (Figure 2).

4.2 Isolation of site from aquifers

The Dakota Group is approximately 1100 feet below the surface in this area (Souders, 1976). Souders shows that the Niobrara is underlain by about 250 feet of Carlile Shale and about 80 feet of Greenhorn Limestone and Granerosus Shale (ibid). The Greenhorn Limestone is about 1000 feet below the surface and along with the Dakota Group, exceptionally well isolated from any contaminants originating on the site’s surface.

The only other major aquifer in the area is part of the Ogallala Group. However, this unit is all mapped north and west of the site. The Ogallala Group, including the Ogalala/High Plains Aquifer, is isolated from the site by a ground water divide, which is less than one mile south of the site. The site is on the north side of the ground water divide and the Ogallala Aquifer is on the south side of the divide.

4.3 Site Geology

The only ground water that has been exploited at the site comes from difficult to find, isolated pockets in the 40 foot thick veneer that overlies the Pierre Shale. These wells have such low yields that is useless to attempt to directly withdraw the ground water from the soil and rock with pumps. Instead the farmers using this water construct sumps or small reservoirs into the Pierre Shale and tap the very small yields that trickle into the sumps with overflow pumps.

The sediments overlying the Pierre Shale where deposited on subdued early Tertiary topography of Pierre hills. The sediments mostly consists of poorly indurated clay, claystone, silt, siltstone and fine-grained aeolian sand. Scattered pockets of sand and gravel clasts lie at the base of this sequence and rest on the Pierre Shale or the thin soil horizons that map these buried Pierre hills. The majority of the gravel clasts are derived from the Pierre.

The upper portions and the surface of the Tertiary sequence show evidence of wind erosion, which forms elongated, closed troughs several tens of acres in extent. The surface troughs only flood during the heaviest rainfall and water captured in them mostly are dissipated through evaporation (Figure 3).

5.0 SUMMARY AND CONCLUSION

The purpose of this study is to examine the site selection done in Nebraska for lessons that could be translated into a checklist for hazardous waste siting, particularly for small projects which might not otherwise have the necessary resources to develop some of this information.

The selection of a site for a low-level radioactive waste disposal facility was an intense and very public process. This led to a consideration of over 100 sites over a broad swath of Nebraska and selection of an excellent site in Boyd County. The Butte site, Boyd County, Nebraska is particularly well suited for the disposal of low-level radioactive waste. It’s simple geology consist of a veneer of Tertiary sediments overlying an thick impermeable barrier which protects major aquifers under the site and to the south of the site and is simple to model and monitor. This facility siting process has led the researchers through a large volume of resources and regulations that required extensive study. It appears that some of the lessons learned from studying these resources, particularly as they relate to site decisions, could be applicable.
to hazardous waste siting of small projects. When this idea was first introduced during a presentation in the Midwest on landfill siting (Migues, 1999), it was well received by several individuals who are involved in siting in this region.

REFERENCES


Site investigation for Palau compact road

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ABSTRACT: A subsurface site investigation was carried out for the Palau Compact Road project on the island of Babeldoa, Republic of Palau. The road alignment traverses rugged terrain incised with numerous small streams, thick tropical forests, and dense mangrove swamps. Bulldozer cut trails were required for most of the boring locations with some locations requiring hand cut trails and hand carried portable drill equipment. Proper planning and field coordination were essential to the successful completion of the investigation despite constant washout of roads from heavy rains, lack of nearby fuel supplies, limited communication facilities, and the need to avoid archaeological sites. Laboratory test results of the subsurface soils of the island are presented, and some preliminary recommendations for design are also discussed.

1 INTRODUCTION

The United States will design and construct approximately 85 kilometers of road on Babeldoa, the largest and most mountainous of the islands of the Republic of Palau, as part of the agreement contained within the Compact of Free Association between the Republic of Palau and the United States. The Palau Compact Road will be a two-lane, two-way rural collector road designed and constructed to U.S. standards.

Babeldoa is inhabited by about 3,500 people, located primarily in villages along the coastal areas. The proposed road alignment extends through mostly uninhabited portions of the island. Existing paved roads are limited and are located primarily in the southern end of the island, in the vicinity of the airport. Access roads throughout most of the island consist of dirt roads requiring 4-wheel drive vehicles.

2 SITE CONDITIONS AND GEOLOGIC SETTING

The islands of Palau are a part of Micronesia located in the western portion of the north Pacific Ocean as shown on Figure 1. It lies approximately 885 kilometers to the east of Mindanao, Philippines and about 1,309 kilometers to the southwest of Guam. The Palau Islands consist of a small, elongated, arc-shaped island chain in the Western Caroline Islands, which trend generally north to south. The major islands comprising the Republic of Palau generally trend in a north-south direction and consist of Babeldoa, Koror, Utwe, Eil Malk, Peleliu, and Angaur. The islands of the Republic of Palau are shown on Figure 1.

The proposed Palau Compact Road project is located on the island of Babeldoa, which lies due north of Koror, the capital of the Republic of Palau. Babeldoa was connected to Koror by a bridge locally known as the Koror-Babeldoa (K-B) Bridge, which provided access for vehicles between the islands. In September 1996, the bridge collapsed leaving the islands separated and without means for vehicular access. At the time of our field investigation, the primary means of commuting between the islands was by government-operated barge service and boats. A temporary vehicular bridge was erected and was subsequently placed into operation in early August, 1997.

2.1 Geology of Babeldoa

Babeldoa Island exists as a result of the accumulation of volcanic materials along the crest of the Palau Ridge. The form of the island is principally controlled by the faulting associated with past tectonic activity, numerous historic submarine eruptions and emergence as a result of sea level changes and tectonism, marine erosion, erosion by streams, and the effects of deep tropical weathering of the volcanic materials comprising the island.
In general, Babeldao consists of a central strip of the oldest volcanic rock, overlapped on the east by the next younger unit, and separated by a fault zone on the west from the youngest volcanic rocks, which are arranged in a rough circle around a former caldera at Kamandao Bay. The regional ridge and trench structure, extensive volcanism, and the presence of numerous faults cutting the pyroclastic deposits confirm past tectonic activity. However, there is little evidence of on-going deformation of the arc at the present time.

Because of the humid tropical environment, the volcanic rock exposed at the surface is usually extremely weathered with fresh outcroppings of rock being generally rare in occurrence. The majority of the volcanic materials observed on Babeldao Island have been weathered to residual and saprolite soils with only scattered local pockets of hard, indurated rock. The deep tropical weathering of the island has resulted in a mature stage topography with rounded, rolling hills separated by scoured stream channels.

2.2 Geography

The Palau Islands form a curved chain of islands totaling about 350 in number and covering approximately 458 square kilometers in area. The island chain extends approximately 161 kilometers in length and 64 kilometers in width. Babeldao, the largest island of the chain, is an elongated, volcanic island trending north to south and encompasses about 208 square kilometers of land area. Elevations on Babeldao range from about sea level along the coastal areas to about 242 meters above mean sea level at Mt. Ngerakelehaus on the northwestern side of Babeldao.

The Palau reef encircles the island of Babeldao as a barrier and fringing reefs, creating extensive quiet water lagoons and bays with some deep channels and passages adjacent to the island. Beyond the fringing reefs, the ocean floor drops steeply to greater than 300 meters in depth within a distance of about 1 kilometer.

The island of Babeldao is primarily of volcanic origin with some localized limestone formation. It is characterized by rolling uplands and incised streams in extremely weathered volcanic materials. Flat alluvial areas are mainly restricted to the mouths and banks of larger streams and along the coastline situated behind sand beaches where inland hills do not abruptly terminate at the shoreline. More rugged terrain consisting of sharp ridges and cliff lines occur on the western and southeastern portions of the island. Rounded hilltop knobs and ridge lines rise above the rolling hills at scattered locations, offering prominent landmarks amongst the generally homogeneous backdrop of rolling topography.

The coastline of Babeldao is irregularly shaped with many small bays and headlands. Swamp areas consisting of dense mangrove within shallow tidal-influenced flats exist throughout approximately 70 percent of the coastline areas. The width of the mangrove swamp varies from less than 10 meters to greater than 800 meters in some places. The mangrove swamps encompass the lowland coasts and large areas of bays in addition to some island areas surrounding major rivers and streams.

2.3 Climate

The Palau Islands have a maritime tropical, rainy climate characterized by small seasonal changes. Temperatures are generally uniform throughout the year and rainfall is abundant with a maximum in July and minimum from February through April. The mean annual temperature is about 27 degrees Celsius with a relative humidity of about 82 percent. Rainfall typically occurs as intense showers of short duration and averages about 250 to 500 millimeters per month. Periods of prolonged, heavy rainfall lasting several days at a time occur as typhoon storm centers frequently pass to the north of Palau. Other pronounced rainy periods occur as the equatorial convergent zone encroaches within the vicinity of Palau around June and July and again in September and October.

3 SUBSURFACE INVESTIGATION

The subsurface investigation was carried out in two phases in order to maximize the field exploration effort and to allow the road design to proceed concurrently with the geotechnical field investigation. The first phase consisted of a geologic reconnaissance along the preliminary road alignment to delineate the approximate limits of the various geologic units encountered. Ten borings were drilled in relatively accessible locations within the various geologic units encountered along the preliminary alignment.

The second phase of the investigation included a detailed field exploration program along the selected road alignment. A total of about 180 borings were drilled along the 85 kilometer length of the road. The borings were placed at selected critical locations involving deep cuts and fills, soft ground crossing, bridges, culverts, and causeways.

3.1 Phase I Exploration

For the preliminary field exploration conducted in December 1906, generalized alignment maps and preliminary earthwork information were utilized to
plan and perform the geologic reconnaissance and boring program. The intent of the preliminary exploration was to categorize the general physical properties of the soils found on Palau.

A general geologic reconnaissance was performed to characterize and map, on a regional scale, the various geologic materials located within the proposed alignment. Previously published geologic data including available government reports and maps were used to plan the exploration objectives and define anticipated conditions prior to arrival in the field.

Based on available geologic maps, ten widely-spaced locations were selected for boring exploration, which corresponded to mapped locations of differing soil types. The preliminary borerings were drilled using hollow stem auger techniques penetrating to depths of about 7 meters (22 feet) below ground level. “Undisturbed” soil samples were collected and packaged for air-freight shipment to our soils testing laboratory in Honolulu.

The information from the reconnaissance, borerings and laboratory testing allowed the development of preliminary geotechnical design parameters, such as cut/fill slope ratios, compaction requirements, allowable bearing pressures, pavement thickness, and soft ground stabilization. Armed with these design parameters the civil engineering designers could evaluate the feasibility of the preliminary alignment, evaluate alternate routes, and identify critical areas requiring additional study.

### 3.2 Phase II Exploration

Phase II was undertaken from March through September of 1997 following the submittal of updated alignment information from the design team. Based on the design information, a total of about 180 test borerings were planned at selected locations along the 85 kilometer alignment. The alignment was divided into four regional sections. The exploration effort for each section was executed separately from the other using separate crews and equipment. The field explorations were concurrent with the ongoing concept design effort.

### 3.3 Mobilization

Due to the dense tropical forest, it was extremely difficult to accurately locate the road alignment in the field. Therefore, prior to the field mobilization, the centerline of the alignment was professionally surveyed and staked in the field for reference. Boring locations were selected and tabulated based on the need for subsurface information at deep cut and fill locations, at planned bridge or culvert crossings, at suspected soft ground areas, and other structures. Concurrent with the pre-mobilization planning of the field exploration, shipping of drilling equipment, tools and supplies from Hawaii to the Republic of Palau commenced.

Various drill rigs including skid-mounted, truck-mounted (rubber-tired), and track-mounted (caterpillar) machines were prepared for shipment. To enhance our capability and reduce mobilization time, portable drilling equipment that could be mobilized manually were included to accommodate remote area exploration, such as mangrove swamp or bories sites on extremely steep hillslopes. Sufficient drilling tools such as auger, casing, and coring tools were shipped to outfit each drill rig with the capability to drill the expected depths.

In anticipation of expected rainy/muddy road and trail conditions resulting in poor traction and eventual off-road accidents, the vehicles were equipped with off-road tires, fire chains, tow straps, and electric winches. Civilian Radio (CIV) radio units were outfitted in each crew’s truck to serve as the primary means of communication for coordination and emergency response purposes.

### 3.4 Field Coordination

Because the majority of the road alignment and the boring locations traverse undeveloped, rugged and forested terrain with limited access, the need for clearing of access trails ahead of the drilling sequence was crucial. Prior to mobilization, the location of access trails were plotted to accommodate minimal stream crossings, changes in elevation requiring substantial cuts and fills, and connections to existing dirt roads.

Considerations requiring attention during the field layout of access trail routes and boring sites included continual coordination with other consultants and governmental agencies involved on the project. Consultants included unexploded ordnance specialists and archaeologists. Various national governmental agencies included the Palau Environmental Quality Protection Board, Fish and Game, and individual State Governors, and local landowners.

Unexploded ordnance remaining from historic military occupancy of the island lies buried and exposed throughout the island presenting a risk of ignition by disturbance with heavy equipment operating in the area, especially test boring drilling operations. Prior to any mechanized field operations such as trail clearing or drilling, all work sites, trails and access routes had to be surveyed and cleared by the ordinance specialist team. If potential ordnance was found during the field survey, the boring site or access trail was relocated to a clear location.

In addition, once a potential boring site and access trail was marked in the field and cleared by
ordinance personnel, the sites were surveyed by an archaeologist for approval to traverse archaeological features, if any, or to avoid the features altogether.

If a planned trail route was altered in the field during clearing, the change was approved by the ordnance specialists and archaeologist once again.

3.5 Trail Clearing

Local contractors were hired to provide equipment and personnel for the trail clearing operations. Bulldozers, such as Caterpillar D-2 and D-4 sizes, were utilized due to their compact size and lighter weight, both advantageous for working in rugged, slippery, and sometimes soft ground conditions. The smaller bulldozer size helped to reduce unnecessary destruction to forested areas and archaeological features residing within the proposed alignment or trail routes.

Local manual labor was employed to cut larger trees ahead of the bulldozer, construct stream crossings, and install erosion control measures along the trails. Due to the high rainfall of the region, the terrain of Hahndorf is largely formed by stream runoffs creating a rugged, forested topography of high relief accentuated by sharp ridgelines and incised stream gullies. To effectively clear trails in compliance with local environmental law in this difficult terrain, careful planning and layout of the trail routes were necessary.

Once the trail routes had been flagged and received the necessary approvals in the field, trail clearing operations could commence. The trail clearing operation was monitored on a near full-time basis to assure that conditions of the environmental permit were followed and to direct the daily progress in conjunction with nearby drilling operations. Typically, trail clearing work and drilling operations were coordinated to maintain close proximity to each other for reasons of safety and effective progress. Often, due to rainfall and muddy conditions, the drilling operation would need assistance from the bulldozer to move between drill sites or extract vehicles stuck in the mud.

Monitoring of the trail clearing provided assurance that environmental restrictions related to the earthwork and stream crossings were being adhered to. In addition, on-site changes to trail routing could be coordinated with consultants for faster response time. Where stream crossings could not be avoided for vehicle access, temporary pipe culverts or bridges constructed of logs and branches were utilized. Large trees cleared from the trail route were transported to the stream crossing by bulldozer and placed as flume spanning the stream. Smaller tree trunks and the leaves of Beetlebean trees were used in conjunction with sifted fabric to retain a thin soil cover to complete the bridge construction.

3.6 Drilling Exploration

The drilling operations for each section commenced separately as finalized alignment information became available. Towards the end of the field work, four drilling crews were working simultaneously on each of the four sections. Depending on the terrain and access conditions, various available drill rigs were utilized which best suited the site conditions.

Track-mounted drill rigs worked well at sites where long access trails with minimal ingress points had been cleared or where site conditions were very wet due to rainfall and forest canopy cover. Four-wheeled drive rubber-tired vehicles with tire chains were severely limited under these conditions. Their use was typically restricted to dry weather days or to being towed by a bulldozer. Portable drilling equipment was used successfully to drill shallow borings (<10 meters in depth) located at remote sites, such as extensive mangrove swamps, steep drainage basins, steep hillslopes, or archaeologically and/or environmentally sensitive areas which precluded mechanized trail clearing. Lightweight, compact, trailer-mounted equipment was successfully employed at boring locations where necessary depth of penetration was less than about 15 meters and where trails were narrow and relatively dry between rainstorms, such as along the ridgelines.

Drilling techniques utilized on this project included continuous-flight augers, rotary wash boring, and rock coring. Sampling methods included the use of standard penetration test (SPT), Modified California split barrel sampler, piston sampler, and NX or HQ core barrels. Borings ranged in depths from about 6 meters to 36 meters below ground level. Due to the remote location of the drill sites and lack of water supply, streams and rivers became the primary source of water for the drilling operations.

Difficulties were experienced in transporting water to the drill sites because of the lengthy distance from streams. Several methods were used including loading drums of water by bulldozer (due to wet and muddy trail conditions) or 4x4 tanker truck in addition to pumping horizontal distances up to about 200 meters from nearby streams. To retain the water during drilling, ponds were excavated to contain the water for recirculation. Recirculation of drilling
water was successful due to the massive, slightly fractured character of the rock minimizing water loss in the formation. The need for multiple water hauling trips was reduced, thus saving time and wear on equipment used to haul the water.

3.7 Weather Delays

Trail clearing and drilling progress delays were anticipated at the onset of the project due to the heavy rainfall associated with the monsoon season. The heavy rains typically occurred as regional downsours and because of the large extent of the project site, some crews were not affected while others were.

Through careful planning and daily weather updates from Koor, storm episodes were often anticipated and alternate preparations were made in advance for contingency work. Often times, a single day of heavy rainfall would isolate a particular area from trail access for a period of several days. By having multiple pieces of machinery, including portable equipment that could be operated under variable site conditions, progress was slowed but was not entirely halted. In addition, contingent site evacuation plans were made daily in the event of sudden onset of downsours, which could render the trails undriveable. Alternate vehicles were parked at existing road/trailhead intersections and other safe locations so that the crews would have a means of transportation back to the base camp should other vehicles become stuck in the mud and have to be abandoned.

3.8 Field Communications

Due to a lack of reliable telephone service or cellular capability in the smaller villages, radio communication was a necessity for effective coordination between the four drilling crews spread throughout Babeldaob. Citizens Band (CB) radio was selected for use in Palau based on relatively low cost, ease of use, and lack of local traffic using the CB radio frequencies. Each crew was outfitted with a vehicle mounted radio, and guidelines were implemented for check-in times and designated channels of use. The field project manager served as central dispatch to relay messages between crews and coordinate work projects, sharing of tools, equipment repairs, and need for supplies.

The use of radio communication proved invaluable to the success and timely completion of the project by reducing downtime associated with the long travel time between points. Nightly radio check-in was accomplished as each remote unit would report progress and problems to the field project manager to be coordinated the following day.

4 LABORATORY TESTING

Most of the laboratory soil testing was conducted on Babeldaob. Consolidation and other special testing equipment were sent from Honolulu to supplement existing equipment on Babeldaob. On-site laboratory testing saved considerable time and expense of shipping thousands of kilograms of soil samples to outside laboratories. Experienced testing engineers from our Honolulu office supervised the on-site testing and monitored quality control. Complex tests, such as triaxial compression, were performed in our Honolulu laboratory.

5 RESULTS OF INVESTIGATION

The proposed roadway alignment generally traverses various volcanic, sedimentary, alluvial, and lacustrine deposits ranging from very dense volcanic breccia and hard tuff rock formation, to medium stiff to very stiff saprolite and old alluvial soils, to very soft and/ or loose recent alluvial, mangrove swamp, and coraline lacustrine deposits. Based on the selected alignment for this project, the majority of the earthwork operations will be in the medium stiff to very stiff saprolite soils. Saprolite is composed mainly of silty materials and is typical of the tropical weathering of volcanic rocks.

At the time this paper was prepared, the project was proceeding to final design. Because of the difficult terrain and logistics for the site investigation, the geotechnical engineering exploration was performed concurrently with the Concept Design and is almost complete. Based on our laboratory testing of the on-site materials to date, some of the typical physical properties of the saprolite soils at the site are summarized in Table 1.

The earthwork for this project constitutes a major portion of the costs since the estimated volumes are on the order of about 3 million cubic meters of excavation and about 4 million cubic meters of embankment fills. Therefore, the design slopes for cuts and fills have a significant impact on the construction budget. Flatter, conservative, slope ratios generally increase slope stability. However, in the mountainous terrain along the proposed roadway alignment, flatter slope ratios result in greatly increased earthwork quantities and erosion potential. Therefore, somewhat steeper slope ratios have been considered for this project. We believe that the reduced earthwork quantities and reduced erosion potential outweigh the potential for localized sloughing of some of the steeper cut and fill slopes. In general, we recommended that slope inclinations of one horizontal to one vertical (1H:1V) and 2H:1V be used for the design of the planned cut and fill slopes, respectively.

Due to the high in-situ moisture contents of the
Table 1. Physical Properties of Saproplite Soils on Babeldub Island

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unified Soil Classification System</td>
<td>Claysy Silt (MH) and Silt Clay (CH)</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>70% to 120%</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>30% to 60%</td>
</tr>
<tr>
<td>Percent Silts and Clays (&lt;0.074mm)</td>
<td>70% to 100%</td>
</tr>
<tr>
<td>Wet Density</td>
<td>1,200 to 1,800 kg/m³ (75 to 110 lb/ft³)</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>40% to 89% (average about 65%)</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.5 to 2.8</td>
</tr>
<tr>
<td>CBR value</td>
<td>1.0 to 5.0 (Average 3.0)</td>
</tr>
<tr>
<td>Maximum Dry Density (ASTM D1557)</td>
<td>1,300 to 1,400 kg/m³ (80 to 90 lb/ft³)</td>
</tr>
<tr>
<td>Optimum Moisture Content (ASTM D1557)</td>
<td>25% to 35%</td>
</tr>
<tr>
<td>pH</td>
<td>5.0 to 5.5</td>
</tr>
<tr>
<td>Minimum Resistivity (ASTM G35)</td>
<td>30,000 to 60,000 ohm cm</td>
</tr>
</tbody>
</table>

Saproplite soils, substantial aeration of the high moisture soils would be required in order to achieve the normal 90 percent relative compaction and may not be feasible due to the year-round high rainfall experienced at the project site. As a result, the compaction requirements for general fill placement will have to be reduced in order to facilitate earthwork construction using the high moisture tropical soils in this wet environment. Based on our engineering analyses, we believe that a compaction requirement of a minimum of 85 percent relative compaction would be appropriate for this project considering the wet environment.

In addition, because of the lower compaction criteria, we recommended a fill slope inclination of 2H:1V or flatter for design of the fill slopes. Many areas of the roadway alignment have fills on the order of about 10 to 20 meters thick. When the high moisture sapropelite soils are excavated and placed as fill at the lower compaction requirement of 85 percent relative compaction, the fills will settle due to compression by self-weight, especially for the thick fill areas. Therefore, a settlement waiting period and settlement monitoring program are recommended for deep fills during construction prior to pavement construction.

Because of the significant earthwork volumes for this project, it is important that the earthwork be balanced between excavation and embankment fills. Due to the low dry densities of the in-situ sapropelite soils, we believe that there will be shrinkage of the materials when the sapropelite soils are excavated and placed as embankment fills. Based on our analyses, we believe that the sapropelite soils may experience shrinkage on the order of about 20 to 25 percent. Therefore, the excavation and embankment fill volumes have been designed to be balanced based on this shrinkage factor.

Based on our observations of the existing cut slopes throughout the island, substantial erosion of graded areas was noted. Therefore, an adequate erosion control program would be crucial to the satisfactory performance of the planned cut slopes and fill embankments against potential failures. Due to the inherent properties of the sapropelite soils, establishment of vegetation at the slope face would be very difficult and may require a long time. Implementation of appropriate ground cover for erosion control is still being studied and may include a combination of specific types of grassing and flexible geosynthetic matting.

Based on our laboratory test results, the in-situ sapropelite soils or compacted fills composed of the same materials generally exhibit relatively low CBR values in a very moist to wet condition. Strengthening of these subgrade soils with the inclusion of additives, such as cement or lime treatment, and for the incorporation of ground stabilization fabrics, may be required prior to pavement construction. The specific subgrade stabilization measures have not been determined at this time.

A substantial portion of the roadway alignment (about 4 of the 85 kilometers) will traverse mangrove swamps and/or soft lagoonal deposits including the two causeways for the project. Where the roadway alignment traverses soft and/or unstable ground, such as swamps and mangroves, we recommended that these areas be stabilized by placement of a working platform prior to fill placement. Depending on the soft ground thickness and the planned fill heights, substantial settlements of over 1 meter in magnitude may occur as a result of the consolidation of the soft soil deposits under the new fill loads.

6 CONCLUSIONS
The field investigation for this project was
performed with a crew of 14 engineers, geologists, drillers, and driller helpers over a period of about 5 to 6 months. The field investigation effort required substantial bulldozer clearing and construction of temporary log bridges. The largest obstacle to the field investigation was daily access for the field crew and equipment to the drill sites.

Typically, where projects are located in difficult and sensitive terrain, we would have resorted to extensive helicopter mobilization of equipment and personnel in order to perform the site investigation. Some of our projects, such as Interstate Route H-3 and major electric power transmission lines in the State of Hawaii, USA, are usually done in this manner. However, the availability of helicopters for equipment mobilization was limited in the Republic of Palau at the time of our work. Furthermore, the costs associated with helicopter mobilization would have been substantial.

The field investigation for this project demonstrated that "conventional" exploration methods using bulldozer cut trails can be effective in adverse terrain and weather conditions and that careful planning and field coordination can help to minimize disturbance to the environment and archaeological sites. Backup equipment and a flexible work plan were essential to minimize nonproductive time due to weather and other uncontrollable events.

This project also demonstrated the effectiveness of a two-phase site investigation for design in a compressed time schedule. The data obtained in the first phase allowed the concept design phase to commence with a level of confidence early in the design process. The detailed second phase investigation was performed concurrent with the concept design to minimize the overall design time.

7 ACKNOWLEDGMENTS

The Palau Compact Road project is administered and funded through the U.S. Department of the Interior and managed by the U.S. Army Corps of Engineers, Pacific Ocean Division. The project was divided into four design packages. The prime civil engineering consultants were from Honolulu, Hawaii and consisted of Belt Collins Hawaii, Earth Tech, Parsons Brinckerhoff Quade & Douglas, and R. M. Towill Corporation.

We appreciate the assistance we received from the Republic of Palau government, particularly the Ministry of Resources and Development, the governors of the various states, Black Micro Corporation, and the people of Palau.
The high cost of low bid cost site characterization

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ABSTRACT: Site characterization is an important first step in the design and construction of many civil engineering projects. It has an important bearing on foundation design and behaviour. Improper, inappropriate or inadequate site characterization leads inevitably to unnecessarily conservative designs, project cost increases, and on occasion, avoidable failures. Many public works and infrastructure agencies continue to procure site characterization services on the basis of the lowest bid cost. Some examples from the author's experience prove the case against such procurement practices.

1 INTRODUCTION

Site characterization is an important step in the realization of many civil engineering projects. The work required to properly characterize a project site may range from a basic review of geological data to detailed site specific investigations with boreholes, cone holes, penetrometers, and in situ testing devices. The work may also include ongoing monitoring with piezometers, slope indicators and similar instruments.

Many state, provincial and municipal public works and infrastructure agencies in Canada and the USA are procuring site characterization services on the basis of the lowest bid cost (LBC), with little or no consideration for the selected firm's capabilities, prior experience, or competency to perform the work. An exception to this exists in the USA where the Brooks Act prohibits federally funded projects from procuring any engineering service on the basis of cost alone. The LBC procurement practice, where in effect, is justified on the grounds that it promotes fair competition and hence protects the public interest. Nothing could be further from the truth. LBC site characterization procurement practices cost the tax paying public many times more than any savings in competitive fee bidding, as the case histories described here will prove.

Giberson (1980) has provided a comprehensive review of competitive bidding for architectural and engineering services. Tulloch (1980) concluded that competitive bidding for consulting services is inadvisable, contending that time and money pressures would erode technical competence, thoroughness and innovation. Mirza (1993, 1996) has presented arguments showing that LBC procurement practices increase national/debt and deficit. Perhaps the best way to illustrate the very high cost of LBC procurement practice is through case histories, here taken from the author's own experience. They include only those cases where a firm selected through LBC procurement was given the first opportunity at site characterization, followed by a repeat investigation at the same site by the author's firm or the firm for which the author then worked.

Although LBC procurement of engineering services is prevalent in the private sector, it is less ubiquitous for site characterization studies. The private sector recognizes the large savings it can realize with a proper site characterization. Ironically, the public service sector, which is sworn to protect the public interest, is the most guilty in disregarding the wider public interest.

The case histories described here demonstrate that a proper investment in site characterization produces a return on investment which easily exceeds 2000 per cent.
Due to the sensitivities associated with the case histories described here, names and locations have been omitted. However, the geotechnical facts, some now recalled from memory, have not been altered. It is also pointed out that no consultant involved in these case histories was ever charged with malpractice or found negligent. The quality of the work performed by the consultants selected on the basis of the lowest bid cost simply reflects the financial constraints under which they agreed to perform their work.

2.1 Grocery Retail Store

A 2,000 m² retail store was to be constructed for a national grocery food chain corporation in a small town. Corporate policy required the company's Chief Engineer to obtain a minimum of two price quotations for site characterization. The firm for which the author then worked, Firm A, submitted a cost estimate (not a fixed price quotation) of $4,000. Firm B quoted a lump sum guaranteed price of $500. The apparent saving to the client corporation was an undeniable $3,500. The Chief Engineer complied with corporate policy and reluctantly awarded the work to Firm B.

A few months later, Firm A received a call from the Chief Engineer for a re-investigation of the same site because the estimated cost of constructing the store was in excess of his budget and in excess of costs per square metre for similar stores elsewhere.

The site consisted of a loose, wet to saturated, medium-coarse sand of some 2.5-3.0 m thickness over a dense sand to clayey sand glacial till. Based on Firm A's recommendations, the lightweight steel frame single story retail store was successfully built on a slab on grade underlain by compacted frost free coarse well drained granular fill of about 0.5 m thickness. Service trenches were dug without well point dewatering, by excavating upgradient and allowing the seepage water to be pumped from sumps. Workers were protected within trench boxes. The native material was re-used as compacted backfill above the engineered backfill zone of the service pipes.

The geotechnical design put forward by Firm A saved the food corporation close to $100,000. It was later revealed that Firm B's recommendations involved deep foundations and well point dewatering of the entire site. It was also learned later that Firm B had conducted the site investigation with a backhoe.

By investing an additional $3,500 in an appropriately priced site characterization study, the Chief Engineer saved his corporation $100,000. In ROI (return on investment) terms, the financial gain was $100,000 * 100 / ($4,000 - $500) = 2857 per cent.

2.2 High School Buildings

Two school boards (Board P and Board C) decided to build their respective new high schools on one common site in order to share the playing fields and auxiliary facilities. They both retained one architect but hired their own structural and geotechnical engineering firms. The two school buildings were to be connected with a 6 m long covered corridor at ground level. Each brick clad building was to be a maximum of two stories in height with no basement.

The site characterization work for the building belonging to Board C was awarded to a low bid cost firm (Firm C) which, as it was learned later, quoted $1,000. The site investigation for the Board P structure was given to the author's firm (Firm A) for an estimated fee of $6,000. Note the price differential of 600 per cent.

Both geotechnical firms mobilized to the site during the same week in late autumn. At one point, the drilling machines for the two firms were situated not more than 20 m apart. Firm C completed its investigation in one day. Firm A remained at the site for over a week. Firm C drilled its holes with a solid stem continuous flight auger. It obtained samples in the standard penetration test.

Firm A usedcased wash boring techniques with an H sized casing. It obtained 50 mm and 75 mm diameter thin wall tube piston samples from each boring. Each sample retrieval was followed by in situ vane testing to determine the undrained shear strength of the clayey soil at the site. All recovered thin wall tube samples were carefully capped and waxed in the field. They were then laid on the back seat of a car for transportation to the laboratory 100 km from the site.

A few months after submission of the geotechnical design report to the structural engineering firm
referred by Board P, Firm A received an inquiry regarding the soundness of its recommendation to support the Board P high school building on spread footings above a fairly substantial deposit of firm sensitive marine clay. The architect stated that Firm C had recommended a steel pipe pile deep foundation scheme for the Board C high school building. He wondered how it was possible to have two diametrically opposite foundation solutions for the same site.

A meeting was called to discuss the relative merits and costs of the different foundation schemes. The additional cost for pile support of the structure for Board C was estimated at over $150,000. In light of the technical arguments presented by Firm A, both Boards agreed to the spread footing alternative.

In the 30 years since their construction, neither building exhibits any settlement associated distress. Both structures are underlain by about 3-4 m of very stiff to hard desiccated silty clay over about 10 m of firm sensitive marine clay. Firm A's investigation revealed about 100 kPa preconsolidation in the firm clay stratum. This fact was used to advantage in designing the depth and size of the spread footings.

In this case history, Firm C was obviously pressed for time and money. It completed the site characterization work using a generally accepted but totally inappropriate methodology. Firm A conducted the site characterization using an appropriate methodology and saved the client $150,000, for an ROI of ($150,000)/($50,000) = 3000 per cent.

2.3 Railway Bridge

A medium sized city approached three geotechnical firms, two located in a major city some distance away, to quote a lump sum price for the foundation investigation of a new three-span bridge across an existing railway track. The bridge deck width was to be 36 m, with span lengths of about 30 m each to accommodate standard preecessed reinforced concrete bridge girders. Approach fills were to be 10-11 m high at the two abutment locations. The highest elevation of the north approach fill was located over a 1.07 m diameter precast concrete sanitary trunk sewer pipe buried 6 m below ground level. The effect of the north approach fill on the sewer pipe was a major concern.

Three preliminary borings had previously been drilled at this site. The log of one 20 m deep boring was provided to all three bidders. The cost quotations from the three invited firms were $16,000 (Firm G), $15,215 (author's firm, Firm S) and $3,000 (Firm L), a local consultant who had just established his practice in that city. The work was awarded to Firm L.

The 20 m deep borehole log showed the presence of a 3-4 m thick mottled grey brown silty clay, probably a desiccated crust, overlying 14 m of softer varved silty clay above a limestone bedrock. In situ vane tests indicated the undrained shear strength of the varved clay below the upper desiccated zone to be in the order of 100-150 kPa, with a sensitivity of 3 to 10.

A few months after the award of the work to Firm L, the principals of Firms G and S received a partial copy of Firm L's report and were asked for their opinion on the technical merits of the recommendations. The city was forced to elicit these opinions because the provincial fund sharing agency did not accept the report prepared by Firm L and requested the city to obtain second opinions. Both principals reviewed Firm L's recommendations independently. Working without knowledge of each other's involvement in this review, the two principals reached identical conclusions about the deficiencies inherent in the Firm L data and recommendations.

Firm L's work was found to be deficient in site coverage and depth of borings. Bedrock below the varved clay deposit was not proven by coring yet a recommendation was made to drive abutment piles to the inferred bedrock surface. Firm L also recommended that the two piers be supported within the desiccated crust on spread footings. It had not considered the settlement of the softer soil below. Analyses indicated the piers would have failed in bearing capacity (for a two layer soil system). In addition, the effect of settlement of the north approach fill on the buried concrete sewer pipe was not considered.

Subsequently, Firm S was invited to re-investigate the site for a fee comparable to that paid previously to Firm L, which was $30,000. Firm L had pressed a successful case with the city for an addendum of $27,000 to their original quotation of $3,000 because they had to drill several more borings more than estimated initially to delineate the soil stratigraphy.
Firm S concentrated on sample quality and completeness of stratigraphic cross-section information below the north approach fill. It found the thickness of the desiccated zone to be 5-6 m rather than 3-4 m. The thickness of compressible varved clay below the invert of the sewer pipe was found to range from 0.7 m to 9.8 m from toe to toe of the trapezoidal shaped north approach fill, due to a sloping bedrock profile.

Detailed analyses were conducted to ensure that the sewer pipe could withstand the differential settlements which it would experience after construction of the north approach fill. Laboratory work was extended to a mineralogical examination of the varved clay soil (Quigley, 1987) to explain unusually high preconsolidation values from six consolidation tests on the relatively soft varved clay samples. As a consequence of this additional investment in site specific research and quality sampling and testing, the sewer pipe did not have to be relocated. The estimated cost of pipe relocation was close to $1 million.

In this case history an investment of $30,000 in quality site characterization work resulted in a cost saving of $1 million - an ROI of over 3333 per cent!

In addition, a potentially serious bridge failure was avoided.

2.4 River Bridge

The Town Council of a small community agreed with a road needs study that a 50 year old single span steel low truss bridge across a major river should be replaced. A civil engineering consultant was retained for preliminary design. The municipality decided to invite bids for site characterization studies and awarded it to the lowest bid cost quoted by Firm D. Firm D drilled four boreholes ranging in depth from 31 m to 56 m. It took undisturbed thin walled tube samples from a 10 m thick deposit of varved clay and reported an average undrained shear strength of about 40 kPa based on laboratory vane tests. Consolidation testing on two 50 mm diameter thin walled tube samples indicated a pre-consolidation of 106-183 kPa. Consolidated drained triaxial tests on the varved clay gave a small cohesion intercept (c = 4 kPa) and an effective angle of internal friction, $\phi$, of 26 degrees. Stability analyses were conducted for a proposed increase in approach fill height of 1.0 m, and a shift in the bridge alignment of about 15 m to the east.

Firm D concluded that the increased fill height along the revised alignment would lead to unsafe side slopes, and the safety of the existing structure would be compromised. The existing bridge was to be kept in service until the new structure was opened to traffic. Based on its field and laboratory test data, and subsequent analyses, Firm D recommended that the new bridge should be a 3-span structure with approach fill forward and side slopes of 3:1 (H:V). It also recommended the construction of half height rockfill berms to increase the factor of safety against slope instability.

Firm D's geotechnical report was reviewed and approved by the provincial transportation authority which would be funding 75 per cent of the cost of the structure. The remainder was to come from municipal coffers. The bridge designer prepared what he felt was the most economical design for the three span configuration and submitted his cost estimate of just over $3 million for the new bridge. Of this amount the municipality would be responsible for 25 per cent. This cost estimate did not include costs for property purchase to accommodate the 3:1 approach fills - the existing approach fills were sloped 2:1, with their toes located just within the right-of-way limits.

A councillor asked why it was necessary to build a three span bridge (at a potential cost to the municipality of $750,000) when the existing bridge was a single span structure. He also wondered whether site conditions could have changed so dramatically within 15 m of the existing bridge alignment to justify such a major change in structure configuration. After discussions, it was agreed that the site conditions should be re-investigated by another firm.

The author's firm undertook a site characterization study. Two detailed boreholes were drilled close to the new alignment, one on either side of the river. The general stratigraphy encountered was more or less similar to that reported by Firm D. However, major differences were found in the engineering properties of the varved clay deposit. Careful in situ testing showed the undrained shear strength of the varved clay ranged between 60 kPa and 80 kPa. Tests on 75 mm diameter thin walled tube piston samples of the varved clay gave preconsolidation
pressures ranging from 140 kPa to over 200 kPa. Analyses indicated the fill side slopes could be constructed at 2:1 to avoid property purchases, provided a thin softer upper layer of varved clay was removed from behind one abutment location. In conducting stability analyses, it was found that widening the pile supported abutment footings towards the backfill would increase the factor of safety sufficiently to permit the proposed grade raise.

The following is quoted from the report which was submitted to the client: "The site investigation carried out in this study has more or less confirmed the soil stratigraphy inferred from an earlier investigation at the same site by others. However, the major difference between the two investigations has been the assessment of physical and engineering properties of the main deposit - a firm to stiff varved clay and silt sequence of 12 m thickness. In the earlier investigation, the thin wall samples obtained were 50 mm in diameter. In this investigation 75 mm diameter samples were obtained, and carefully examined for detail of origin and character..... Hence, it must be concluded that the overall varved deposit is generally more competent than it was presumed to be".

The report also included the following commentary in response to the council's concerns: "To the uninitiated in the finer points of detailed geotechnical engineering, it is illogical that a structure built several years ago, without benefit of computer aided analysis and sophisticated laboratory and field testing can be standing whereas one to replace the very same structure, with all the attendant sophistication of modern technology, needs to be different, and perhaps more expensive to build. They seldom understand that successes of the past may be the accident of ignorance, and that present day codes and bridge loadings are far different from those of several years ago. However, it is incumbent upon the geotechnical engineer to rationalize from scientific investigations and the art of his discipline, the reasons for the differences".

In any addendum type of investigation, it is difficult to ignore the work, thought and logic presented in the earlier investigation. Differences in opinion can and do occur, not because of any inherent weakness in the practice of the discipline, but due to differences in either appreciation of the problem or due to differences in personal levels of confidence or experience.

On the strength of the addendum investigation, the new bridge was designed as a single span structure with its abutments placed close to the river banks. Due to concerns with potential liquefaction of deep, loose saturated silt and sand strata below the varved clay stratum, piezometers were installed near each abutment. Pile driving was stopped when the pore pressures exceeded pre-determined values. All nearby buildings (including a hotel) and the existing bridge were closely monitored for movements and settlements. The new bridge was constructed at a cost of just over $1 million. Thus, the municipality saved $1 million in bridge construction and property purchase costs.

In this case history, an investment of $30,000 in a properly funded site characterization and site monitoring study resulted in a total project saving of close to $2,250,000, equivalent to an ROI of 7500 per cent! For the municipality, which paid the full fee of the addendum investigation, the ROI was 3333 per cent.

3 DISCUSSION AND CONCLUSIONS

Oscar Wilde is reported to have said people know the price of everything and the value of nothing. This aphorism certainly applies to low bid cost procurement of site characterization services.

A Canadian newspaper headline in the business section of a 1993 edition read: "Sony chief points to villain in manufacturing's decline". The columns below this heading reported that Margaret Thatcher, then Prime Minister of United Kingdom, asked Mr. Akio Morita, then Chairman of Sony Corporation, what advice he had to give Britain. "Mrs. Thatcher, please change your society's concept to make people respect engineers as they do lawyers or chartered accountants - that's my advice", said Mr. Morita.

In a keynote address at TUNCON '92, Robbins (1992) noted "... the cost of construction will always be reduced by the use of the highest standard of technology and the best use of expertise". The highest standard of technology and the best expertise are hardly synonymous or compatible with LBC procurement of engineering services.

A past President of the Association of Professional Engineers of Ontario (APEO, a self-regulatory licensing body) wrote "... I have asked APEO
solicitor's to advise us on what we can do to prevent "low-ball" engineering from becoming a menace to the public." (Angus, 1993). He went on to state "...insufficient engineering will result in situations affecting public safety, the protection of which is our raison d'etre."

Osterberg (1978) has provided case histories of failures in exploration programs. He states: "Bids are taken and the work given to the lowest bidder. With such a procedure, the most basic goals of an exploratory program (to find the types, thicknesses and extent of various geological formulations and their properties) is missed."

Those involved in site characterization studies need to increase public awareness of the disadvantages of LBC procurement practices. Collectively, the engineering and geosciences profession must educate consumers of the advantages offered by appropriately compensated site characterization services. Case histories such as those presented by Osterberg (1978) and in this paper are a good starting point.

The four case histories presented here have demonstrated the enormous returns on investment which are possible when site characterization work is done without fee constraints. The high cost to the public when LBC procurement is used for site characterization is summarized in Table 1 for the four case histories presented earlier.

On the other hand the returns on investment in appropriately compensated site characterization services are summarized in Table 2.

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<th>Avoidable Cost</th>
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<tr>
<td>Railway Bridge</td>
<td>$ 12,000</td>
<td>$ 1,000,000</td>
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<tr>
<td>River Bridge</td>
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<td>$2,250,000</td>
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<th>Case</th>
<th>ROI - %</th>
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Tables 1 and 2 begin to prove the case against LBC based procurement of site characterization services. More examples are needed from other practitioners to prove the high cost of low bid cost site characterization.

REFERENCES


Site assessment technique on areas with expansive soils

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ABSTRACT: Geological hazards like expansive soils pose formidable obstacles to economical growth in many countries. Although site investigation techniques are becoming more elaborated, but detection of site characteristics associated with expansive soils is proving frequently difficult and unreliable. The expansive/shrinkage behavior of expansive soils primarily depends on composition, particle size distribution of the constituents and the fabric of the deposit. In turn these features are controlled by diverse factors like parent rocks, climatic conditions, conditions of transportation of the soil as well as post depositional history. In many countries of the world, dealing with the problems on expansive soils is getting top priority with increasingly frequent discussions both in the government and non-government sectors as national economies suffer seriously from these unwanted losses. Unfortunately the ensuing discussions, frequently irrelevant, do not help to characterize the actual damage occurring in the engineering structures and the associated problems. The proper assessment of site could be the best solution to minimize the ever increasing losses from expansive soils.

1 INTRODUCTION

Construction and maintenance problems have become a common experience with projects erected over expansive soils. Throughout the world the swelling of soils entails millions of dollars in damage to all types of structures but mostly to low cost buildings, highways, runways, slopes and many other engineering structures. The magnitude of deformation caused by the expansive soils is the function of many factors including landform, climatic conditions as well as depositional history to mention a few. Many researchers (Shadun 1972, Petrov et al 1972) have noted the direct relation between expansive soil and other major engineering geological processes like landslides, pseudo-colluvium, sulfation as well as intensive soil erosion especially in tropical climatic zones.

The need for reliable and accurate assessment of site in advance of occupation has been catastrophically illustrated in many places in the world. For example in Morogoro in Tanzania, a three hour rainfall triggered over 1000 landslides and mudflow within an area of 75 square kilometers. The cost of this single event was placed at about $90,000/US US.

If the engineering properties of expansive soils depend primarily on the soil composition and particle size distribution of its constituents, these factors are basically controlled by the parent rock deposition and post depositional history as well as climatic conditions.

Climatic regimes do give rise to specific suites of deposits influencing particle size distribution, mineralogy and exchange cations exerting major influence on the engineering performance of expansive soils and is the most significant factor in classification and geotechnical modeling.

The influence of the above mentioned parameters on various soil indices like atterberg limits is commonly used as an indication of swelling potential, but there are many factors which are directly related to the magnitude of deformation of engineering structure mostly these factors are unaccountable the determination of swelling potential such as sample disturbances, stress relief and absence of large scale discontinuity contributing to distortion of the experimental data. However these points demands further studies and do not fall within the scope this paper.

Practical application of mathematical and computer technology ensure that the qualitative aspect of complicated natural phenomenon like expansive soils can be analyzed with greater degree of efficiency and precision, since the majority of
Fig. 1  Typification of engineering geological territory on the surface of the earth's crust

GEOSTRUCTURAL UNITS:
I - shield old platform
II - Plate old platform
III - Shield young platform
IV - Plate young platform
V - Epi-platform orogenic and riftogenic folded basement of various age
VI - Epi-geosynclinal orogenic and riftogenic Mesozoic/cenozoic folded belt
VII - Geosynclinal belt and volcanic islands

ENVIRONMENT AND SOIL CONDITION:
A - Water
B - Permanent frost
C - Humid zone
D - Subhumid zones
E - Arid / semi-arid zones

1) Deep waters
2) Fault
3) The boundaries of geostructural units
4) The boundaries within geological units
geological processes are fundamentally genetic and are determined by a wide span of data which make it possible to penetrate into engineering geology with modeling of natural processes. Thus more attention could be drawn to the factors conditioning various deformation observed in various regions of the world.

The solution to the problem of expansive soils depends on the blending of regional experience, with the application of semi-empirical soil data on prediction of swelling potential, this may lead to the development of methods for controlling engineering structural failures. It has been noted that the environment is analogous at many sites where tectonical and climatic conditions resemble each other. The uniform distribution of properties within a typical site is well illustrated in figure 1 according to (Bridgerv 1977). There is every reason to direct our attention and more resources to study methods of typification of the earth surface which can lead to the development of standardized methods for construction and designing for zones with complicated engineering geological hazards like expansive soil, which is the aim of this paper.

2 ASSESSMENT OF EXPANSIVE SOIL SITES

Considerable volume changes in soils can be found in almost all typified territories on the earth’s crust. About 80 typical regions has been identified almost all over the world deformation from expansive soils has been reported. Vulnerability depends mostly on climatic and soil conditions.

Typification of earth surface can provide a rapid appreciation of the distribution of existing potential hazards. Through the use of 9 geotechnical units intersected by soil condition (especially moist content) there has been much success in strategic and preliminary planning for many different engineering geological hazards like earthquakes. However this method is not reliable for assessing territory with expansive soils. A reliable assessment of sites can be gained successfully from field survey and site investigations. Study of the environmental conditions (climatic and topographical) and use of in situ investigations and laboratory tests can lead to the selection of the best solution for tackling the deformation potential.

2.1 Laboratory Tests

Analyzing type and amount of clay minerals can be very useful but sophisticated laboratory installation and highly qualified specialists are required. The most commonly used simple identification tests including atterberg limit do provide vital information for preliminary studies and for practical purposes. Chemical analyses such as ionic exchange capacity can provide more information on the magnitude of expansion/shrinkage. Though Physical mechanical tests have been used to quantify volume change there are still many problems unresolved. Especially on fesswell test many authors have adopted the minimum criteria for swelling soil to be < 30%. At this magnitude deformation is considered to be at minimum. On top of the above mentioned criteria, the author has suggested a flow chart for preliminary site assessment, figure 2. It should be well understood that the magnitude of swelling depends on the initial moisture content and the density of the sample on the given site, the type of instrument used and its installation can give a very different result for the same sample.

2.2 Geographical survey

Topographical survey and climatic condition data do play the most vital part in delivering information for the assessment of sites with expansive soils. While knowing the location of paleosols, rivers, flood plains, artificial drainage areas, landform is useful, the most important single factor is to understand the prevailing climatic conditions. There are many ways of classifying the climatic conditions. In this paper Thornthwaite moisture coefficient has been adopted (TMC).

The TMC value shows the variation of the soil moisture content, and the deficitary condition or an excess of water inside its mass. If 0-TMC<2 the area can be classified as desert. If 2<TMC<5 the area can be classified as semiarid. If 5<TMC<10 the area can be classified as subhumid and if 7<TMC<10 the area can be classified as humid. It has been proved that when the TMC value falls within the range of 2 and 7 with the presence of active clays, it can cause foundation failures (Streany et al 1978). If the natural moisture content is very high and kept constant on site we should not expect shrinkage and swelling and severe damages to engineering structures should not be expected even with the presence of considerable montmorillonite minerals usually more than 10% on site.

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3 ZONING OF TERRITORY WITH EXPANSIVE SOILS

The ultimate requirement for any site assessment should be to determine all environmental conditions which will interfere with the proper construction or exploitation of engineering structures. Due to acute financial constraints, especially in developing countries, generalized site assessment rules usually adopted are sometimes irrelevant and redundant. Blindly extrapolating case histories can lead to colossal financial losses. Therefore geotechnical involvement is vital in the stages of planning for contract operations. For many years engineering geological zoning has been successfully employed in the complex assessment of site with various engineering geological processes. On zoning of sites with expansive soil the author recommends the division of territory into district geological belts and the district geological belt into sites. The zoning of territory with expansive soils based on freewell criteria has been attempted and discussed elsewhere by the group of authors (A.V. Chebanov et al. 1992). This type of zoning is another variation on published principles of engineering geological zoning. The disadvantages of the above mentioned method lies in the uncertainty on freewell/shrinkage magnitude and many other factors not given enough value by the authors. The standard method of division of territory into Regional, province, district and sites depends mostly on the engineering condition of the territory. If the regional and province are usually represented as geotectonic element and geomorphologic view respectively, the division of districts is based on the lithological view while the sites are based on the moisture content as well as the amount of montmorillonite, mixed layered clay minerals and also the freewell test. For practical purposes the detailed study of soil up to the depth of 10 meters has been considered suitable for this purpose.

3.2 Engineering geological district DA(DA1, DA2)

This is typical district characterized by freshly weathered coarse fragments usually of shallow depth and of hard rock. Within the district gravelly coarse sand is common and the shallow parent rock is composed of hard rock conglomerate and course angular gravel. DA1 or gravel and rounded sandy materials - DA2. Sometimes we may find the gravel and sandy materials from 0 to 2 meters from the earth surface. Problems with expansive soils is not common. These districts can be found around the upper parts of hills as well as in the upper, middle and lower part (DA2) of the river basins. Vegetation: Kirkia acuminata, sclerocarya caffra, cammiphora spp. and albizia rhodesica. Value for foundation - Good bearing value, strip foundation recommended.

3.3 Engineering geological district D2B(D2B1, D2B2)

District with shallow to medium depth of hardrock. Within this district the parent rock materials are mostly composed of hard conglomerate and gravel D2B. D2B1 - composed mostly of sandstone. D2B2 - Clay, sandy-clay, sand usually distributed from depths ranging from 2 to 10 meters. These types of regions are mostly developed on the middle part of hills and the middle parts of the river basins and the lower part for (D2B3). Vegetation: Acacia nigrescens, Schinziota pappaphainoides, Digitaria eriantha, Combretum appiculatum. Expansion/shrinkage is possible. Foundation Value: strip foundation, recommended pile foundation especially for D2B2.

3.4 Engineering geological district D3C(D3C1, D3C2)

Characterized by great depths of hard rocks. Fine grained soil well distributed throughout the district D3C to the depth of 10 meters from the earth’s surface. Generally throughout the district the areas with coarse grained soils is very scarce. D3C1- characterized by silts and clay soils, mixed layered minerals and considerable amount of montmorillonite. D3C2 - characterized by clay and silts and by high content of expansive minerals. The given district can
be found around the periphery of mountains, upper terrace and lower part of river basins.
Vegetation: A. grandis, A. toona, Dichanthium, papillosum, Sorghum verticilliflorum, Acacia nilotica, Albizia harveyi.
Generally the possibility of expansion/shrinkage is very high. Correction tests should be performed.
Foundation Value: Special foundation should be used including underreamed piles.

3.5 Engineering geological district D4D(D4D1, D4D2)

generally the district is characterized by active geodynamics processes D4D can be found within the above mentioned districts.
D4D1- District with steep slopes; D4D2- District with swampy as well as high organic soils.
Vegetation: Typical flood plains vegetation including Alnus temuipina, Bombardoa insculpta, Themeda triandra.
This district is recommended for the growth of green belt
Foundation Value: very poor.

3.6 Engineering geological sites DS1(S1A1, S1A2), DS1(S1B1, S1B2), DS1(S1C1, S1C2), DS4(S1D1, S1D2)

Very high value of Hydrothermal coefficient T<10C<10, the expansion shrinkage is very low. Basing on the (SNRP(USSR) 2.02.01-83), Esr varies from 0 to 4 %, Normal continuous brick walls on strip foundation can be used, plant transpiration should be prevented.

3.7 Engineering geological sites DS1(S2A1, S2A2), DS2(S2B1, S2B2), DS3(S2C1, S2C2), DS3(S2D1, S2D2)

Characterized by the medium value of hydrothermal coefficient 5-10C<10, freewelling has medium value of 4 to 8 % except in DS1. Split construction with reinforced brickwork is recommended.

3.8 Engineering Geological Sites DS1(S3A, S3A2), DS1(S3B1, S3B2); DS1(S3C1, S3C2), DS4(S4D1, S4D2).

Low hydrothermal coefficient 2-4C<5, high deformation from expansion/shrinkage, except sites on DS1. Underreamed piles recommended with suspended floors.

3.9 Engineering geological site DS1(S4A, S4A2), DS2(S4B1, S4B2), DS3(S4C1, S4C2), DS4(S4D1, S4D2)

Hydrothermal Coefficient is very low but greater than 2. As above, stiffened raft foundation recommended. Leakage from broken sewer and pipes. Irrigation canals can increase the moisture content leading to expansion of soil.

4 THE ADVANTAGES OF TYPIFICATION

The extent to which ground investigation is carried out depends upon the size and importance of the project. Very frequently construction on expansive soils is associated with low cost housing projects in which funds allotted to geotechnical assessment is very limited resulting in later expenses on maintenance and repair which are usually higher than those incurred through adequate site assessment prior to the foundation design. Identification at the planning stage of suitable site within the planned area for the particular structures will ultimately lead to more economical development through the avoidance of unnecessary expensive foundations or maintenance occasioned by adverse features of site. Once the zone of likely damage has been determined planning strategies or avoidance can be applied, these being the main advantages of typification of regions. Other advantages are:

a) Unification of the methods of design construction and standards as well as structural elements.
b) Concentration of engineering geological investigations on the limited territory providing an opportunity for effective processing of engineering geological information can lead to the introduction of generalized parameters of physical mechanical properties.
c) Coordination to unify engineers and researchers on effective methods of construction, exploitation and designing on complicated sites (Expansive soils).
d) Proper orientation of specialists on engineering geological investigations.
5. CONCLUSION

Assessment of suitable sites for particular structures especially low cost buildings, roads, runways and slopes can in no doubt lead us to more economical development in areas with expansive soil. We could avoid unnecessary ground treatment or the laying of expansive foundation. Planning decisions should take into account the cost of obtaining suitable geological and geotechnical data.

6. REFERENCE

A.P. Mwasha 1992
principal of zoning on areas with expansive soils
(case study: Expansive soil in East African Rift Valley) oral present
XXVII Scientific Conference, KSMA

Paul H. et al. 1972
Landslide in Mgeta Area Western Uluiguru mountains in Tanzania
General statekst Litografiska Anstalt, Stockholm, Sweden

Parasite na proektovanie osnovanie zdanie i
srubshchee (1986)
SNIP 2.02.01-83
Moskva, Stroitel

Suturani i Narodi. (1973)
Zemly i Chudesostros. Obshni Obzor, M.
Vip. 36 Fig. 74-82
Geotechnical investigation for a typical power plant – A case study

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SYNOPSIS: This paper presents a case study of Geotechnical Investigation including Geophysical tests carried out for a thermal power plant, in different stages. Each stage is progressively examined in greater detail from the earlier stage investigation. During preliminary investigation, a dense of weathered sandstone was found at two locations of the boreholes. This was also confirmed from geophysical tests indicating low shear wave velocity in seismic reflection tests. This was further examined at detailed investigation stage. Based on the data obtained and the thickness of weathered sandstone, the main power house area was divided into different zones. This zoning helped to calculate the length of pile, and termination criteria for the installation of piles for a given diameter. At this site depending upon load intensities of the structures, both open and deep foundations were adopted and found to be the techno economic solution.

1. INTRODUCTION

For any industrial structure, like power plant, one of the major requirements in planning, design and construction is proper and adequate subsurface investigation. The object of subsurface and related site investigation is to provide the engineer with as much information as possible about the substrates, and other Geotechnical features of the area for founding these structures in subsoil. The data also help in assessing the suitability and quantum of material for use in construction.

The type and extent of foundation exploration required for power houses of given capacity of plant varies from site to site, type of foundations proposed to be adopted, the subsurface conditions, and perhaps cannot be adequately visualized in advance. The exploration generally proceeds in stages, and the characteristics of the subsols are examined in progressively greater detail as the exploration proceeds.

The subsurface exploration for a power plant is carried out similar to any major industrial plant in four stages described below:

a) Reconnaissance stage investigations,
b) Preliminary stage investigations,
c) Detailed stage investigations and
d) Construction stage investigations.

The information together with the normal topographical survey provides the engineer with complete details of the site and enables him to prepare economical designs for the power house structures.

The data obtained from Geotechnical and Geophysical Investigation carried out in different stages for a typical power plant located at central part of India has been used in characterization of the plant area into various zones to arrive at type and depth of foundations is presented as a case study in this paper.

2. CASE STUDY

For a typical generating power plant of 1000 MW located in central part of India, the Geotechnical Investigation including Geophysical Investigation has been carried out in different stages as mentioned earlier. The plot covering an area of about 160000 sqm is investigated for various power plant facilities and auxiliary systems. However, the
main power house covering an area of about 80000 sqm is presented here for clarity. In this thermal plant, there are structures which are sensitive to settlement (upto 8mm) and few structures are heavily loaded with load intensity of 60-70t/aqm, and there are equipments which induce vibrations. Few structures require deep excavations. The load from the above mentioned structures and equipments are required to be transferred to the founding soil without causing excessive deformations so that the structure is not impaired during its life time. Since, this power plant is adjoining other power plant therefore, the stability of neighbouring plant and its underground services are also considered during various activities of the proposed plant. Hence, the object of subsurface explorations is to provide very detailed and thorough data to enable the engineer to design the foundations of the structures, economically and ensure stability with respect to neighbouring structures.

To achieve this object the investigation is carried out in the following manner:

a) Preliminary desk work
b) Site reconnaissance
c) Planning including drafting specifications for Field and Laboratory tests
d) Conducting the field and Laboratory tests
e) Analysis of the test results.

The first two stages (a) and (b) form the prerequisites and stages (c) to (e) common for Preliminary and Detailed Geotechnical Investigation.

2.1 Preliminary Desk Work And Reconnaissance

In the preliminary desk work, the following information like the Geology of the area was collected from the neighbouring plant, topography maps of the region was collected from Survey of India publication and seismicity of the area was obtained from BIS publication. Coordination with the project group has helped to locate the exploratory tests.

2.2 Preliminary Investigation

The scheme for preliminary Geotechnical Investigation is prepared based on the information obtained from preliminary studies, site reconnaissance and Geotechnical information in the neighboring plant. The method of exploration like boring in soil and drilling in rock, static cone
penetration tests, and Geophysical tests are decided based on the information required for the plant. The depth of boreholes are arrived from the shape, size and nature of structure. Keeping these factors in view, the Technical Specification are prepared conforming to Indian Standards for conducting the Data to be collected from field and laboratory tests.

The investigation includes 12 boreholes and 11 static cone penetration tests (SCPT) under various structures and are shown in Fig.1. The boreholes were 150mm dia. In the boreholes, undisturbed samples (UDS) were collected at regular intervals of 3m. The standard penetration test was conducted at regular intervals of 3m. The depth interval between UDS and SPT was kept at 1.5m.

The water table is found to be at 0.5m to 1.0m depth below ground level. The subsoil strata profile along typical section is plotted in Fig 2. The subsoil comprises of Medium sandy silt of thickness varying from 4m to 6m. This is underlain by Dense silty sand of thickness about 3m to 7m. The overburden is underlain by medium size yellowish sand stone and grayish fine grained sand stone except in BH 8 and BH 12 where weathered coarse grained sand stone of thickness of 10m and 22m is observed in these boreholes respectively. In the sandy silt the field SPT 'N' values ranges between 25 to 40, and in silty sand layer the N values vary between 30 to 50. In the weathered sand stone the value is more than 100.

Fig.3 Shear Wave Velocity
The unconfined compressive strength of the sandstone is between 60 to 85 kg/sq.m.

Based on the soil data for lightly loaded structures open foundations are proposed in sandy silt layer and for heavily loaded structures pile foundations are proposed.

Geophysical Tests

To find the shear zones, faults, weathered horizon and joints, Geophysical tests were carried out. These tests comprised of electrical resistivity test and seismic reflection tests. The traverses are shown in dotted lines Fig 1. The electrical resistivity values generally varied from 60 to 100 ohm-m for sandy silt layer and varied from 80 to 140 ohm-m for silty sand layer. The shear wave velocity obtained from SRT along the traverse (C1C2) and (F1F2) are shown in Fig 3. It is observed from the above figure the shear wave velocity in sandstone is ranging from 2150 to 2550 m/sec. However, at certain location, the value was about 1800 to 1900 m/sec. The decrease in value was decided to examine in detailed stage.

2.3 Detailed Investigation

To provide sufficiently accurate information of substrata under each structure, detailed Geotechnical Investigation has been carried out. The extent of investigation and method of investigation were based on preliminary investigation. The details include 70 boreholes, 5 pile load tests, 20 pressure meter tests in preformed boreholes stage spread within the main powerhouse.

The powerhouse has 2 TG’s and 2 boilers. Under each TG, three boreholes, two at each edge and one at middle were taken. Similarly for the boiler house about 12 boreholes were taken, 4 boreholes under chimney and others were spread within the powerhouse. Typical soil profile at TG area is shown in Fig 4. It is seen that under second unit, the cross-section profile along B57-S8 indicate the sandstone SS2 is at about 10-12 m depth below ground level similar to that observed in preliminary investigation. Where as under first unit, B5 59-60, the sandstone was dipping from depth 22 m to 36 m. Under boiler area similar substrata is obtained. Below overburden the weathered sandstone extends up to 40 m depth. To supplement the borehole information other in situ tests like SCPT, PLT and pressuremeter tests were conducted to obtain the soil properties of different strata.

<table>
<thead>
<tr>
<th>STRATA NO.</th>
<th>DESCRIPTION</th>
<th>BULK DENSITY</th>
<th>VOID RATIO</th>
<th>DIRECT SHEAR TEST %</th>
</tr>
</thead>
<tbody>
<tr>
<td>I.</td>
<td>Mod. to Dense Silt (M3)</td>
<td>1.78</td>
<td>-6.05</td>
<td>0.30</td>
</tr>
<tr>
<td>II.</td>
<td>Dense Silty Sand (SM-SM)</td>
<td>2.00</td>
<td>-</td>
<td>33%</td>
</tr>
<tr>
<td>III.</td>
<td>Coarse Sand Stone - SS1</td>
<td>2.15</td>
<td>0.32</td>
<td>35%</td>
</tr>
<tr>
<td>IV.</td>
<td>Pale Yellow Sand Stone - SS2</td>
<td>2.34</td>
<td>0.22</td>
<td>-</td>
</tr>
<tr>
<td>V.</td>
<td>Grey Sand Stone - SS3</td>
<td>2.68</td>
<td>0.25</td>
<td>-</td>
</tr>
</tbody>
</table>

3 ANALYSIS

The low shear wave velocity recorded during preliminary investigation was examined in detail during detailed stage. From the laboratory test it is found that binding material in SS1 layer is only 7% as against 22% and 25% found in SS2 and SS3 layer. Similarly the sand size particles were about 78% in SS1 layer as against 58 and 50% in SS2 and SS3 layer. This is one of the reason, that during coring no core was retrieved from layer SS1 even though SPT ‘N’ value was more than 100. The lower compressional wave velocity in the weathered sandstone as compared to SS2 and SS3 layer is attributed to poor binding material and high porosity.

Since sampling under high WT and in cohesionless soil was difficult, therefore, to determine the deformation modulus of the layers pressuremeter tests were conducted in preformed boreholes at selected locations. The pressuremeter modulus of sandy silt is between 220-340 kg/sq.m in silty sand the value is between 700-1600 kg/sq.m in the weathered sandstone value is between 1200-2000 kg/sq.m and in the yellowish and greyish sand stone the value is about 3500 kg/sq.m. This data was used for estimating the settlement of pile.

Based on the properties of each layer the bearing capacity and settlement of open foundations were calculated. For instance for footing width of 2.0 m, at 2.5 m below natural ground level, a net allowable bearing pressure of 25 t/sq.m has been arrived for permissible settlement of 25 mm. For heavily loaded
structures, pile foundations were envisaged. Based on the above data and keeping the neighboring plant in view, the RCC Bored Cast-in-situ type of pile has been selected. The diameter chosen was 500mm and the design capacity in compression was taken as 100t equal to structural capacity.

To determine the pile length under various structures, it was necessary to identify the rock level and thickness of weathered sandstone SS1 layer directly underlain by overburden. For this purpose, it was required to plot the rock contours. From the borehole data, the depth of SS1 or SS2 below the overburden are plotted in Fig. 5. Based on this data, the zone where SS1 is encountered has been demarcated by dotted line shown in the above Figure. This was further divided into four different zones depending upon the thickness of SS1 layer. The thickness of this layer was about 1-2m at N-W and increased to about 2-3m towards S-E direction and where the overburden rests on yellowish sand stone. This Zoning helped in estimating the length of pile for given capacity. For instance, where SS2 had been encountered immediately below overburden, the capacity of pile was based on unconfined compressive strength of rock (Canadian Foundation design manual) and when SS1 has been encountered below overburden, the capacity of piles was based on N value (IS:2911 part 1 section 2). This information is used in arriving at the termination criteria of the pile and location of pile load test. The termination criteria of the pile was either based on UCS or N value or both depending upon the strata encountered. The minimum length of pile arrived is 14m. The pile has been tested as per Indian Standards (IS 2911 part 4) and the test pile confirmed the safe capacity of 100t. The data also helped in preparing the Technical Specifications for the installation of RCC piles, which is beyond the scope of the present paper. The construction stage investigation is beyond the scope of the present paper.

4 CONCLUSIONS

It is seen from the present study that to determine the physical, chemical and engineering properties of the subsurface strata either as founding material or as construction material, it is necessary to carry out Geotechnical Investigation.

The preliminary desk study and site reconnaissance has helped in preparing the scheme for preliminary investigation. The type and extent of exploration was commensurate with the size, and nature of the structure and Geotechnical information obtained from neighboring plant. The weathered strata detected in two boreholes during preliminary investigation was also inferred from the seismic refraction tests. The Geophysical tests also revealed that there are no fissures/fractures between the borehole to borehole in horizontal direction.

The detailed Geotechnical Investigation has provided sufficient information of substrata under each structure. Based on the obtained plan area and the zone has been divided into different zones depending upon the thickness of weathered zone of the layer SS1. The properties of the zone were determined from field and laboratory test results. Based on the properties of the substrata, for lightly loaded structures open foundations were adopted and for heavily loaded structures pile foundations were adopted. The zoning of the area helped in arriving at the length of pile and also for locating the area for initial pile load test. Based on the above information termination criteria of the piles has been arrived. The initial load test spread over the various zones have confirmed the capacity and length adopted in various zones. The conditions encountered at this site by judicious selection of both open and deep foundations, adopted for different structures is found to be technoeconomic solution.

ACKNOWLEDGMENT

The author wish to acknowledge and support received from Sh. P.M. Bopanna HOD Civil, NITC.

REFERENCES

Indian Standard : 1892 Code of practise for subsurface Investigation for Foundation.
Bureau of Indian Standards, New Delhi.
Indian Standard : 2911 Part 4 Load test on piles, Bureau of Indian Standards, New Delhi.
Rock type-engineering properties correlation for a site characterization

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ABSTRACT: This paper describes a site characterization methodology, for the design and construction of the Hollywood Water Quality Improvement Project - tank site, based on rock type-engineering properties correlation. It describes the process used to establish the methodology by using laboratory rock testing data, core hole logs, and geological interpretation to characterize the site. A drilling and testing program was performed to determine the engineering properties of local rock types. These rock type engineering properties were used in finite difference modeling to determine static and dynamic forces.

1. INTRODUCTION

The Hollywood Reservoir Complex (HRC) owned and operated by the Los Angeles Department of Water and Power (LADWP) is located in the Santa Monica Mountains approximately seven miles northwest of downtown Los Angeles in the Hollywood Hills. The HRC is composed of two reservoirs, the Upper Hollywood Reservoir (UHR) and the Lower Hollywood Reservoir (LHR). The purpose of the Hollywood Water Quality Improvement Project (HWQIP) is to bring the reservoirs into compliance with the State of California Surface Water Treatment Rule. The HWQIP consists of four major components: Two tanks for covered water storage (see Figure 1); a bypass tunnel around the UHR and the LHR; a microfiltration plant located south of the LHR; and a trunkline connecting the Hollywood Service Area with another major Los Angeles service area.

The tank site is located in bedrock of the middle Miocene upper Topanga formation. The two 110.95-meter (364-foot) diameter, 15.24-meter (50-foot) high, pre-stressed concrete tanks will be among the largest of this kind ever constructed. The tanks will be built in cut material within the bedrock at a bottom elevation of 222.81 meters and will be completely buried and surcharged with 2.44 meters of fill (see Figure 2).

In addition to the rock testing program, a soil investigation was performed to determine geotechnical design parameters to be used in slope stability analyses and foundation design. This paper will focus on the laboratory rock testing and analysis for the characterization of the tank site.

2. IN-SITU ROCK TESTING

Performance of the in-situ testing was an effort between LADWP's Engineering Geology, Geotechnical Design staff and Woodward Clyde Consultants (WCC). Sampling and in-situ testing were conducted in locations and at depths representative of the various lithologies and range of fracture conditions present. In-situ tests included: Dilatometer Testing; Packer Testing; Seismic Refraction Surveys and Downdip Velocity Surveys.

Figure 1. Hollywood Reservoir
3. LABORATORY ROCK TESTING

Laboratory testing was performed by LADWP's Materials Testing Laboratory (MTL) on rock core specimens (samples) recovered from 18 core holes. All testing performed on the specimens was done in compliance with the latest Standards and Test Methods of the American Society for Testing and Materials (ASTM).

3.1 Rock types

Previous testing on specimens from three core holes, for the proposed bypass tunnel, led to an observation that a correlation existed between the rock type and engineering properties. Each specimen was classified based on its physical features and grouped accordingly. The basis for the grouping was whether the specimen was a homogeneous sandstone or a heterogeneous specimen made up of interbedded sandstone with siltstone or claystone. Due to the fact that siltstone and claystone cannot be visually determined, heterogeneous specimens were referred to as "Interbedded Sand/Silt/Claystone". Uniaxial compressive strength (UCS) test results were plotted vs. rock type and a correlation was observed regardless of specimen's recovery location.

As rock testing continued at the tank site for 15 additional core holes, subgroups within the two major rock types were identified, resulting in the five rock types described in Table 1.

4. SITE CHARACTERIZATION

The rock type grouping criteria, graphing and tabulation described in this section lead to a range of engineering properties particular to a Rock Type (referred to herein as "Rock Type Engineering Properties"). Rock type description (see table 1), the graphs (see Figures 3 and 4), and Rock Type Engineering Properties (see Tables 2 and 3) offered in this paper are relative to the analyzed data.

4.1 Rock type engineering properties

Following a grouping criteria based on a specimen’s rock type, dry density, and bedding plane angle (where applicable) Figures 3 and 4 were created. Figure 3 was created by plotting UCS vs. dry density for the homogeneous rock type, and Figure 4 was created by plotting UCS vs. bedding plane angle for the four heterogeneous rock types. The range of test results for the following engineering properties: Young's Modulus, ($E$):
Table 1. Rock Types

- **Sandstone** (S): Interbedded Sand/Silt/Claystone, dry density in the range of 1.94 to 2.16" (ISSC A)
- **Sandstone** (S): Interbedded Sand/Silt/Claystone, dry density in the range of 2.17 to 2.44" (ISSC B)
- **Sandstone with thin layers of Siltstone or Claystone** (S): Dry density in the range of 2.09 to 2.14" (ISSC A)
- **Sandstone with thin layers of Siltstone or Claystone** (S): Dry density in the range of 2.17 to 2.44" (ISSC B)

* = Mg/m³

Poisson's ratio, (ν) and shear strength (angle of internal friction, (φ), and cohesion, (c) were tabulated within four dry density ranges, for the homogeneous rock type, as shown on Table 2, and were also tabulated within seven bedding plane angle ranges, for the four heterogeneous rock types, as shown in Figure 3.

In Figure 3 it was observed that ascending UCS are relative to increasing dry density. Specimens with the lowest dry densities have the largest sand particles (coarse grained) and the dry density increases as the sand particle size decreases.

In Table 2 the following was observed: ascending E values relative to increasing dry density; the ν values do not show a correlation with an increase in dry density; the φ and c values were in the range of 27 to 29 degrees and zero Mpa, accordingly, the dry density was observed to have no influence on the shear strength properties.

In Figure 4 the effect of anisotropic behavior was observed and is discussed here within three bedding plane angle ranges: in the first range (0-25 degrees) the UCS are relative to the rock type's dry density, for each rock type the UCS fall within a narrow range of the highest strengths; in the second range (26-51 degrees) a decrease in UCS for each rock type is relative, for most of the specimens, to the rock type's increasing bedding plane angle, for each rock type the UCS fall within a wide range of high and low strengths; in the third range (52-67 degrees) the UCS are relative to the rock type's bedding plane angle, regardless of the

![Figure 3. Homogeneous rock core specimens](image)

Table 2. Homogeneous Rock - Rock Type Engineering Properties

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Young's Modulus, E (GPa)</th>
<th>Poisson's Ratio, ν</th>
<th>Angle of Internal Friction, φ (degrees)</th>
<th>Cohesion, c (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone (S)</td>
<td>0.08-2.49</td>
<td>0.12-0.48</td>
<td>29</td>
<td>0</td>
</tr>
<tr>
<td>2.19 to 2.24</td>
<td>0.71-3.21</td>
<td>0.14-0.26</td>
<td>27-39</td>
<td>0</td>
</tr>
<tr>
<td>2.27 to 2.37</td>
<td>0.41-3.32</td>
<td>0.19-0.39</td>
<td>27</td>
<td>0</td>
</tr>
<tr>
<td>2.39 to 2.51</td>
<td>8.72</td>
<td>0.38</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**

An engineering property presented as a single value, not as a range, means the value was obtained from a single tested specimen, and for φ and c a single value means it was obtained from a single triaxial test grouping.

E was determined by taking the average modulus of the linear portion of the stress-strain curve.
Figure 4. Heterogeneous rock core specimens

Table 3. Heterogeneous Rock - Rock Type Engineering Properties

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Bedding Plane Angle (degrees)</th>
<th>Young's Modulus, E (GPa)</th>
<th>Poisson's Ratio, ν</th>
<th>Angle of Internal Friction, θ (degrees)</th>
<th>Cohesion, c (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISSC A</td>
<td>0 to 9</td>
<td>0.43</td>
<td>0.22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISSC B</td>
<td>1.97</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sw/SC A</td>
<td>2.51</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Sw/SC B</td>
<td>2.63</td>
<td></td>
<td>0.05</td>
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<tr>
<td>ISSC B</td>
<td>10 to 19</td>
<td>0.60</td>
<td>0.42</td>
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<tr>
<td>ISSC A</td>
<td>20 to 29</td>
<td>0.34-0.79</td>
<td>0.21-0.25</td>
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<tr>
<td>ISSC B</td>
<td>0.37-2.34</td>
<td></td>
<td>0.47</td>
<td></td>
<td></td>
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<tr>
<td>ISSC A</td>
<td>30 to 39</td>
<td>0.40-0.95</td>
<td>0.22-0.42</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISSC B</td>
<td>0.39-1.23</td>
<td></td>
<td>0.11-0.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISSC A</td>
<td>40 to 49</td>
<td>0.11</td>
<td>0.17</td>
<td></td>
<td></td>
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<tr>
<td>ISSC B</td>
<td>0.73</td>
<td>0.35</td>
<td>53</td>
<td>0.09</td>
<td></td>
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<tr>
<td>Sw/SC B</td>
<td>49</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISSC A</td>
<td>50 to 59</td>
<td>0.28-0.44</td>
<td>0.17-0.40</td>
<td>42</td>
<td>0.13</td>
</tr>
<tr>
<td>ISSC B</td>
<td>1.03</td>
<td></td>
<td>0.21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sw/SC A</td>
<td>0.30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sw/SC B</td>
<td>42</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISSC A</td>
<td>60 to 69</td>
<td>0.21</td>
<td>0.10</td>
<td>53</td>
<td>0</td>
</tr>
<tr>
<td>ISSC B</td>
<td>0.17</td>
<td>43</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sw/SC B</td>
<td>16-32</td>
<td></td>
<td>0.41</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
An engineering property presented as a single value, not as a range, means the value was obtained from a single tested specimen, and for θ and c a single value means it was obtained from a single triaxial test grouping.
E was determined by taking the average modulus of the linear portion of the stress-strain curve.
rock type’s dry density, for each rock type the UCS fell within the narrowest range of lowest strengths.
To determine the angle between the applied load and the specimen’s bedding plane angle, subtract the specimen’s bedding plane angle from 90 degrees.

In Table 3 the following was observed: descending E values relative to increasing bedding plane angle; the ν values do not show a correlation with an increase in bedding plane angle; not enough shear strength test results were obtained to thoroughly analyze and conclude on the effect of anisotropic behavior.

4.2. Evaluation of test results

Published laboratory data for typical UCS and Young’s Modulus values were researched and compared and it was observed that the homogeneous sandstone, present at the site, is at the lower range of the published data.

To compensate for the effect of anisotropic behavior and to obtain shear strength properties at various applied loads to specimen’s bedding plane angles, multistage triaxial testing was attempted, but due to the following reasons, it was not possible to obtain accurate data; ASTM specified testing rates were used and the MTL had no means to view, on a screen, stress-strain behavior during testing, the specimen sustained major deformations within seconds after maximum stress had been reached making it impossible to continue testing at a higher confinement; a hydraulic machine was used, and for those specimens which had not been seriously damaged it was very hard to retain a load for a few minutes in order to increase the confining stress for additional testing.

WCC’s geophysical surveys reports were primarily performed to estimate thickness of artificial fill over buried bedrock and to evaluate the riprapability of the bedrock material at applicable areas. The reports can not be accurately analyzed for in-situ vs. laboratory comparisons until the existing reports are presented with more detail, which at this time is not relevant to the project. The authors look forward to continue working on the analysis and testing being performed by WCC and LADWP which in turn will enhance the established Rock Type Engineering Properties.

4.3 Rock type occurrence percentage

For a particular rock type within an examined core hole log, all lengths of bedding intervals were added, then the sum of these bedding intervals was divided by the length (depth) of the core hole and multiplied by 100 to give a value in percentage (referred to herein as “Rock Type Occurrence Percentage”).

5. CASE STUDY - DYNAMIC ANALYSIS TO DETERMINE LATERAL EARTH PRESSURES ON TANK WALLS

The objective of the Dynamic Analysis was to determine the lateral earth pressures on the tank wall when the proposed tank facility was subjected to earthquake shaking. Within a geological cross section of the proposed tanks (see Figure 2), core holes CH96-3, CH96-6, and CH96-9 were analyzed to define and characterize the material properties of the site to use as input into the computer program Fast Lagrangian Analysis of Continua (FLAC).

Based on the specimens tested from these three core holes five rock types were identified. For each of the identified rock types, test results were grouped and average values were determined for the following engineering properties: in-situ density; Young’s Modulus; Poisson’s ratio and shear strength (angle of internal friction and cohesion) resulting in Rock Type Engineering Properties for the five rock types.

Using the Rock Type Engineering Properties and by determining the Occurrence Percentages within each core hole log the values shown in Figure 2 were obtained using the following methodology:

For a rock type within a core hole each Rock Type Engineering Property was multiplied by the Rock Type Occurrence Percentage. These products were added and then divided by 100 resulting in a weighted Engineering Property specific to a core hole (referred to herein as “Core Hole Weighted Material Property (Parameter)”).

The Parameters used for the dynamic analysis were derived from the values in Figure 2 as described in LADWP Report File No. AX503-5A.

6. SUMMARY AND RECOMMENDATIONS

It is recommended to follow a rock grouping based on rock types, dry density and bedding angle (where applicable) to create graphs and tables such as the ones presented in this paper to establish the Rock Type Engineering Properties. These serve as a monitoring tool so that when similar or new rock types are encountered effective decisions can be made to determine the complexity and extent of additional testing.

Rock Type Occurrence Percentage within a studied interval should be noted and reported in order that appropriate testing is performed, minimizing or maximizing testing accordingly. Communication among the rock core logging, rock testing, and design teams is highly recommended for a successful testing program.

This paper has described the methodology used to determine Rock Type Engineering Properties from laboratory testing and the use of core hole
logs to determine the Rock Type Occurrence Percentage. This methodology aided effectively in the site characterization for the Dynamic Analysis for the Hollywood Tanks by making use of available data, without the need for additional testing, and expediting results.

7. ACKNOWLEDGMENTS

The authors would like to thank the following persons for their support: Mr. Robert B. Coardi; Mr. James C. Campbell, and the LADWP's MTL personnel. Mr. Joe Gunther, Mr. Roger Roux, Mr. Jerome Chen, Mr. Jose Garcia, and Mrs. Silvia Nunez assisted in preparing this paper.

REFERENCES


GIS-ASSESS: A spatial analysis tool for site investigation planning and evaluation

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ABSTRACT: This paper describes a quantitative, probabilistic based model for assessing the quality, or thoroughness, of site investigation plans. The approach has been implemented in a GIS-based software tool called GIS-ASSESS. The mathematical basis for the model is described in the paper and two examples of how the method may be used are included. Probability of failure and thoroughness maps for the example problems illustrate how the effectiveness of alternate proposed sampling plans may be evaluated through simulations with GIS-ASSESS.

1 INTRODUCTION

Spatial analysis technologies are potentially useful tools for scientists, engineers, and many others who work with spatial data. This is particularly true for geo-environmental and geotechnical data sets. To date, however, development of spatial tools to perform tasks such as assessing the quality of site investigation plans has been quite limited. Tools available for use in site assessment programs can be divided into a variety of classes including: satellite and airborne imagery, surface geophysics, push technology, drilling and sampling, borehole geophysics and monitoring wells. Tools within each of these categories are used to develop an understanding of the site subsurface conditions. Tools in each class have limitations resulting from factors such as signal attenuation, sensor capacity, the influence of background noise and the data interpretation procedures available. Accordingly, different tools are more appropriate for use at different sites and at different stages of an assessment program. As a result of the wide choice of assessment techniques available for use and their suitability for different stages of a project, the quality of a site investigation program can vary significantly and there is limited formal guidance available to help in optimizing program design outside of qualitative, experience based decisions.

This paper describes the development of a software tool, GIS-ASSESS, which is a GIS-based tool designed to permit assessment of the quality, or “thoroughness” of a site investigation program using geostatistical methods. GIS-ASSESS uses several geostatistical techniques to generate maps of thoroughness for the site of interest. It is designed to work with multi-dimensional data from multiple tools. The result is a quantitative, objective measure of site investigation quality displayed in an easily understood format. The benefits of using such a system include: (1) optimization of site investigation resources by selecting appropriate tools and locations for their use at a given site; (2) the ability to compare and rank the status of investigations at several sites by quality so that future work can be prioritized; and (3) assistance in setting numerical targets for determining when sufficient site investigation work has been completed. This paper describes the mathematical basis for investigating continuous targets. The use of one tool is illustrated with data sets from both geotechnical and geo-environmental cases.

2 WHY GIS?

A GIS allows the user to store, retrieve, sort, and display information according to its spatial characteristics. As such, it is a suitable environment when used by an engineer to integrate the data
management, data analysis and data visualization functions that are typically performed as independent, albeit related, actions in practice today. By integrating these traditionally distinct functions in a GIS framework, the engineer using the spatial based system is clearly the "bridge" between the traditional "islands of automation" as shown in Figure 1 and can thus focus on efficiently "using" the data rather than spending significant levels of effort in just "preparing" the data for use by the various functions.

3 THOROUGHNESS DEFINED

Before an assessment of the quality of an investigation can be made, it is necessary to understand the purpose of the investigation. Most investigations are performed for one of two purposes. Firstly, they may be performed to determine if some natural or unnatural phenomenon is present in excess of some critical amount. For example, the purpose of a geoenvironmental investigation may be to determine if TCE is present in excess of the regulatory limit at a given location. Secondly, investigations are often performed to determine parameters such as the permeability, the bearing capacity or the coefficient of consolidation for geologic units. Design values for these parameters are based on the results of the investigation.

Both types of investigations have two possible outcomes. For the first type of investigation, the concentration of TCE will either exceed the regulatory limit (failure) or it will not (success). For the second type of investigation the actual value of the property will either exceed the design value (failure) or it will not (success). Using geostatistical methods, probabilities can be calculated for both possible outcomes.

The thoroughness is then calculated based on the certainty of the outcome according to the function shown in Figure 2. It can be seen that the thoroughness is not dependent on the outcome but only in the certainty associated with the prediction. As illustrated in the figure, if the probability of success is low or high, the thoroughness will be high. Conversely, low thoroughness values reflect accuracies of success probabilities.

The thoroughness will vary with location because the factors that the probability values are based on, the expected value and the standard deviation of the model error, will vary with location. Therefore, the thoroughness is calculated for a grid of points and a map of investigation thoroughness is produced for the area of interest. The engineer can use these probability and thoroughness maps to determine the likelihood of success or failure at any point, and to draw conclusions about the effectiveness of the tools used and the sampling configurations.

4 THEORETICAL BASIS OF THE METHOD

The targets of a subsurface investigation may be divided into two types, correlated and uncorrelated. Correlated targets are those that are usually present over a wide area, and values at one location can be used to draw conclusions about the target at nearby locations. For example, if the target is the concentration of TCE, then high values measured at one location suggest that the concentration at a nearby location will also be high, assuming that there are no unusual geologic features in between. The assumption that measured values at a sample location can be used to estimate the values at nearby locations is the basis for contour mapping, which is used extensively by GIS-ASSESS.

Uncorrelated targets are those for which data at one location cannot be extrapolated to nearby locations. For example, if the purpose of an investigation is to locate a buried drum or a sump, then a boring taken at one location provides no information regarding the
presence or absence of the target 5 meters away. Use of
nonlinear interpolation packages is not appropriate for
this type of target. The remainder of this paper focuses on
techniques relevant to correlated targets.

The thoroughness of investigations dealing with
correlated targets is calculated using the estimation
technique called kriging. Kriging is often called the
Best Linear Unbiased Estimator. It has this distinction
because it minimizes the variance of the model errors,
it is a linear estimation technique, and it is unbiased
because the expected model error is zero. Kriging
calculates the expected value of a parameter ($x_k$) at
a given location by taking a weighted average of nearby
points according to Equation 1. The values of $x_k$ are
from samples at various nearby locations, while the

$$x_k = w_1 x_1 + w_2 x_2 + \ldots + w_n x_n$$

(1)

weights ($w_i$) are calculated using a function called the
variogram, a function that describes the relationship
between data correlation and distance. For a more
detailed discussion of the mathematics behind the
variogram and the kriging method, the reader is
referred to works by Journel and Huijbregts (1978) or

The advantage of this method is that it also allows
for the calculation of the standard deviation of the
model errors. Given a predicted value and a standard
deviation ($\sigma$) and assuming that the error associated
with the prediction is normally distributed, then the
probability density function can be developed and the
probability that some critical value will be exceeded
is calculated.

This is done by first setting the critical or boundary
value ($b$). This value corresponds to the regulatory
limit or the proposed design value, for example. The
$z$ value for the standard normal distribution is then
calculated according to equation (2) (Dowsdy, 1983).
The probability that ($x < b$) is then calculated by

$$z = \frac{b - \mu}{\sigma}$$

(2)

integrating the probability density function for the
standard normal distribution using equation (3). The
thoroughness is then calculated using equation (4).

$$P(x < b) = \int_{-\infty}^{\frac{b-\mu}{\sigma}} e^{-\frac{t^2}{2\sigma^2}} dt$$

(3)

$$T = \{100 - 2P(x < b)\}$$

(4)

One modification to this process is allowed by GIS-
ASSESS. The system allows for the calculation of a
relative standard deviation ($\sigma_r$) as shown in Equation
5. The relative standard deviation is calculated by
dividing the standard deviation by the

$$\sigma_r = \frac{\sigma}{x}$$

(5)

estimated experimental mean ($x$). This value is analogous to
the coefficient of variation. GIS-ASSESS then multiplies
this value by the expected value at a given location ($x_k$) to
estimate the relative standard deviation at that
location.

The behavior of many geotechnical parameters can
be described by the coefficient of variance (Harr, 1996). For these parameters, the standard deviation
varies linearly with the expected value. Therefore,
areas with small expected values will have small
relative standard deviations compared to areas with
high expected values. The advantage to using the
relative standard deviation is that it accounts for this
relationship while the model standard deviation does
not.

5 OPERATION OF THE SYSTEM

GIS-ASSESS allows for data stored in ARC-INFO
coverages and lookup tables to be accessed based on
data type, depth or other attributes. The user then
generates a grid file of up to 10,600 expected values
and standard deviation values using a geostatistical
package. The file containing these values is used to
create contour plots of probabilities or thoroughness
values and contour plots of expected values and
standard deviations.

Based on this output, the user can optimize the site
investigation strategy. GIS-ASSESS enables the user
to estimate the changes in the effectiveness of the
investigation or monitoring plans or by adding
additional sampling points or removing current ones.
To add additional sampling points, the user selects
points with the mouse and GIS-ASSESS adds new
virtual sampling points at those locations. GIS-
ASSESS simulates sample values by using the
expected value at that location estimated by the
kriging process. The new sample set is written to a file
compatible with the geostatistical package and
the process is repeated. By repeating this process for
different scenarios, the user can estimate the effect that
additional sampling would have on the quality of the
investigation, the locations where those samples would
provide the most useful information, and whether current sampling locations can be discontinued without significantly affecting the investigation quality. In this way investigation resources can be optimized. A flowchart of the operation of GIS-ASSESS is shown in Figure 3.

6 EXAMPLES OF GIS-ASSESS OPERATION

6.1 Geo-environmental case

The example data set shown here was constructed to test GIS-ASSESS during development. Figure 4 shows a site 400 by 300 meters in size. A tank farm that is the source of contamination is present to the north while property boundaries are shown on the other 3 sides. Twenty sample locations are shown located on a 100 meter grid pattern. The data values represent the concentration of contaminant X in ppm. It should be noted that while these samples are taken on a grid pattern, samples may be located randomly or in other patterns.

The sample data was contained in two ASCII files. The first contained x, y, and sample-id information and the second contained sample-id and contaminant concentration information. These files were selected within GIS-ASSESS and a point coverage was automatically created with a lookup table containing the contaminant information.

A file compatible with the geostatistical package was then generated from the point coverage. A variogram model was then generated within the geostatistical package to represent the relationship between data correlation and distance. The relative standard deviation option was used during the variogram modeling process. Using this variogram model, the data was kriged and a grid file of estimated values was created. The grid was 81 cells by 61 cells in size with each cell representing a 5 meter square area. GIS-ASSESS contour plots of expected values and standard deviation values were then generated using this file. Then probability and thoroughness maps for a contamination limit of 30 ppm were generated (Figures 5 and 6). These plots show that while contamination at the property boundary is in excess of 30 ppm is not expected, there is a significant chance that such contamination exists in certain areas.

In some critical areas, such as property boundaries, it is very important that interpretations of contamination or the lack thereof can be made with confidence. Three critical areas with low thoroughness values and low area of high thoroughness have been highlighted with circles in Figure 6. Virtual sampling points were added at these locations. Additionally, Figure 6 shows two sampling locations in the lower corners which are located well into the high thoroughness area which may not be contributing information of great value. These data points were removed and their locations are marked with white squares.
6.2 Geotechnical case

For this data set, GIS-ASSESS was used to evaluate a geotechnical data set from Treasure Island near San Francisco (see Figure 8). The purpose of the analysis was to determine the thoroughness of the investigation in predicting the behavior of the soil at a depth of 20 feet in the event of an earthquake with a magnitude of 6.5 resulting in accelerations of, say, 0.5.

A critical blowcount value of 5 was obtained for this earthquake using the correlation developed by Seed (1985) relating liquefaction to N1 value.

The sample set included all blowcount values for depths between 15 and 25 feet. The average blowcount value was used for borings with multiple blowcounts in this range. The values were then kriged using the relative variance option. A contour map of expected values below the critical value of 5 (Figure 9) and a thoroughness map (Figure 10) were generated.

The contour map shows small areas that are expected to have a N1 value below 5. However, the thoroughness map shows much larger areas where there is some chance that the N1 value will be less than 5. The thoroughness map clearly shows areas where additional investigation should be targeted if a higher level of confidence concerning the outcome is desired and it quantifies the risk that the prediction concerning the boundary value will be inaccurate.

Using this alternate data set, probability and thoroughness grid files were generated to see what effect the changes would have on the thoroughness. The thoroughness grid generated using the original data set was subtracted from the thoroughness grid generated using the alternate data set. The resulting grid was used to generate a contour map (Figure 7) of the change in thoroughness caused by the addition and removal of selected data points.
REFERENCES


7 CONCLUSIONS

Implementation of the GIS-ASSESS tool for assessing the quality, or thoroughness, of site investigations has shown that:

1. A geographic information system, when combined with custom engineering software, is a powerful environment for analyzing geotechnical and geoenvironmental data.
2. A probabilistic model can be used to estimate the quality, or thoroughness, of an investigation in a quantitative way.
3. The thoroughness of an investigation is not uniform but varies with location.
4. Such a model can be used to estimate the impact of the addition or removal of sampling locations and in this way the use of investigation resources can be optimized.

ACKNOWLEDGMENTS

The work described in this paper was supported in part by NSF grant No. CMS-9457549. This support is gratefully acknowledged.
Characterization of soft deposits in the Eastern Nile Delta

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ABSTRACT: The aim of this paper is to discuss the results of a geotechnical investigation carried out in the Eastern Nile Delta, in Egypt. Emphasis is placed on interpretation of the available CPTU records, also in light of the other available field and laboratory tests. The paper focuses on the soft delta units, and in particular on a 30-35m thick deposit of soft clays/silts characterizing the area investigated. The applicability of CPTU-based correlations to the soft clay/silt deposits of the Nile Delta is also addressed, and tentative correlations between the CPTU records and the soil undrained shear strength are proposed. A CPTU-based classification system is applied to establish its suitability to the Nile Delta soils.

1 INTRODUCTION

The aim of this paper is to discuss the results of a geotechnical investigation carried out in the Eastern Nile Delta, in Egypt. The investigation was performed for the design of a gas and condensate treatment plant owned by the Belayin Petroleum Company. The site lies on the Mediterranean coast, about 16km West of Port Said, and is adjacent to the Masala Lake. The site elevation is close to the mean sea level. The plant, which receives natural gas from offshore, includes process components, pipe racks, tanks, buildings, and other settlement sensitive equipment.

The purpose of the investigation was to select and design the most suitable foundation schemes for the plant, and eventually a mixed system composed of deep bitumen-coated driven piles and wick drains was selected.

The available geotechnical data include 15 borehole logs to 50-60m of depth, 7 borehole logs to 15-40m, 5 Dutch-cone CPT records to 35-40m, 11 CPTU records to 45m, 11 CPTU-dissipation tests at variable depths, 9 field-vane tests, odometer tests, UU triaxial compression tests on 'undisturbed' and re-moulded samples, CIU triaxial compression tests, pocket-permometer and torvane tests, classification tests. The CPTU tests were performed by means of a 15cm³ Furgo piezocone, with the porous stone located behind the cone tip base.

In this paper emphasis is placed on the most relevant soil parameters of the soft cohesive units, and on interpretation of the CPTU records. The applicability of CPTU-based correlations to the soft clay/silt deposits of the Nile Delta is also addressed.

2 SITE STRATIGRAPHY

The site is located within the Nile Delta system, where the Holocene delta sediments began accumulating with the rise in sea level at the end of the last glaciation, with an estimated sediment accumulation rate of about 5mm per year (Cousteiller & Stanley 1987). It is characterised by a 30-35m thick deposit of soft clays/silts (Holocene Pro-delta Mud and Delta-front deposits) with sand interlayers, resting on Pleistocene stiff clays/sands of marine origins. The typical site stratigraphy, which is quite constant throughout the plant area, is depicted in Table 1 where essential soil parameters are also presented.

Unit 1 is composed of very soft, medium-sensitive Delta-front clay, silty clay and silt, deposited under lagomar or marsh conditions. It is characterised by a plasticity index varying between 5 and 64%, a liquidity index varying between 1 and 1.4, a sensitivity varying between 2 and 3.2, and by an organic content sometimes in excess of 10%. Beneath Unit 1, a layer of loose to medium dense
Table 1. Typical soil stratigraphy and essential soil parameters

<table>
<thead>
<tr>
<th>Unit</th>
<th>Soil Type</th>
<th>Unit Weight (kN/m³)</th>
<th>Natural Water Content (%)</th>
<th>Plastic Index (%)</th>
<th>Liquidity Index (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-7</td>
<td>silt clay,</td>
<td>15.0-16.5</td>
<td>20-114</td>
<td>8-64</td>
<td>1.0-1.4</td>
</tr>
<tr>
<td></td>
<td>very soft,</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>sandy</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7-12</td>
<td>silt clay,</td>
<td>18.5</td>
<td>0</td>
<td>0</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>loose to</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>dense</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12-22</td>
<td>clay, silt,</td>
<td>16.5</td>
<td>50-75</td>
<td>9-92</td>
<td>0.3-0.8</td>
</tr>
<tr>
<td></td>
<td>to firm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>shell fl.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>22-33</td>
<td>sand, m.,</td>
<td>19.0</td>
<td>0</td>
<td>0</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>to very</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>dense</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>33-60</td>
<td>clay, very</td>
<td>18.5</td>
<td>20-53</td>
<td>26-57</td>
<td>0.6-0.3</td>
</tr>
<tr>
<td></td>
<td>stiff to hard</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Holocene sand (Unit 2) is found, sometimes interlayered with silty clayey soil.

Then, a relatively thick layer of soft to firm, slightly to medium-sensitive Pro-delta clay (Unit 3) is encountered. Unit 3 is characterised by shear strengths increasing linearly with depth, a plasticity index varying between 9 and 92%, a liquidity index varying between 0.3 and 0.8, and a sensitivity in the range 1.1-2.9.

Unit 4 is composed of Holocene-Pleistocene medium to very dense sand, and from this unit down the soil stratigraphy includes overconsolidated (OC) Pleistocene formations. Unit 4 has almost negligible thickness at some locations, and becomes several meters thick at others.

Unit 5 consists of OC, insensitive Pleistocene clay, very stiff to hard, with a plasticity index varying between 21 and 63%, a liquidity index varying between 0 and 0.3 and a sensitivity close to 1. At some locations, significant amounts of Pleistocene very dense sand was also encountered.

3 Cᵥ vs. FIELD AND LABORATORY TESTS

Values of the peak undrained shear strength, $Cᵥ$, in the cohesive units are plotted against depth in Figure 1. The values were obtained by field vane (FV), triaxial UU compression tests (UU), torvane (TV) and pocket penetrometer (PP). Remoulded strength values obtained by field vane (FVr) and triaxial (UU)r tests are also reported. Note that the modifications suggested by Bjerrum (1973) were not applied to the field vane values plotted in Figure 1.

In Figure 1 a line corresponding to the empirical relationship $Cᵥ = 0.23 \times \sigma'ₚ$, (e.g. Janikowski et al., 1985), where $\sigma'ₚ$ is the vertical in-situ effective stress, is also shown for comparison. This relationship is applicable to normally-consolidated (NC) inorganic soils.

In the top Delta-front clay (0-3m) the TV values fit closely the FV values. Their trend shows peak strengths of larger magnitude than predicted by the empirical function, and nearly constant with depth. In the Pro-delta clay (12-35m) the peak, unmodified FV values keep reasonably close to the empirical function, and so do the peak UU values obtained at depths in excess of 24m. On the other hand the UU tests give sensibly lower strengths than the empirical function at depths comprised between 15 and 24m, where the peak values nearly correspond to the remoulded strengths. This is probably due to sample disturbance of the softer specimens. The measurements present considerable scatter, and the
TV values appear to consistently underestimate, on the average, the shear strength inferred from the FV and the UU tests.

In the deeper OC Pleistocene Clay (below 33-35m) an even wider scatter is observed, and on the average the PP values are closer to the UU's than the TV values, which are consistently lower.

4. $C_v$ vs. CPTU CONE RESISTANCE

Typical CPTU cone-resistance records are shown in Figure 2. The $q_c$ values were modified by defining $q_{u}=q_c+(1-\alpha)u_w-\sigma_{vs}$, where $\alpha$ is the net area ratio, $u_w$ is the pore water pressure measured during cone penetration, $\sigma_{vs}$ is the in-situ vertical total stress. With reference to the $C_v$ values shown in Figure 1, an average $N_v=q_u/C_v$ factor equal to 20 is estimated in the NC Pro-delta clay (12-35m), where $N_v=18$ appears suitable in the top Delta front clay (0-7m). In the deep Pleistocene clay (from about 33-35m) the results are by far more scattered, and any estimates of the $N_v$ values are far from being satisfactory. Assuming an average $q_{pm}$ value of 2500kPa and an average $C_v$ value of 200kPa, $N_v=12.5$. Results These evaluations confirm the tendency of $N_v$ to decrease as the OCR increases, which was also observed, for instance, by Lunne et al. (1985) in the North Sea and by Pelli & Ottaviani (1992) in the Adriatic Sea. The main explanation is to be found in the decrease with OCR of the stiffness ratio $f$, (Duncan & Buchignani 1976), whose effects on cone penetration can be described by a theoretical approach (e.g. Teh & Houlsby 1991).

5. $C_v$ vs. CPTU PORE WATER PRESSURE

In the soft clayey formations the pore pressure
recorded during cone penetration, $u_2$, increases linearly with depth, and yields values 2.5-3 times larger than the estimated hydrostatic pore pressure $u_h$ (Figure 3). On the other hand, in the deeper OC Pleistocene clay $u_2$ values smaller than the hydrostatic pressure are detected, although a trend to pressure increase towards the hydrostatic value is observed at depths in excess of 40m.

In the top Delta-front clay, the pore pressure ratio $B_1 = (u_2 - u_0) / u_{eq}$ presents scattered values, probably due to variable permeability, most of which fall within the 0.6-0.8 range.

In the NC Pro-delta clay most values fall between 0.4 and 0.6.

In the deeper Pleistocene clay the $B_1$ values are negative, in the range -0.2 to 0. It is interesting to observe that clays with OCR on the order of 3.5, such as Unit 5, are seldom characterized by negative $B_1$, which instead are more common in unlined clays with OCR in excess of 20 (e.g. Chen & Mayne 1996).

The relationship between $N_{ua} = (u_2 - u_0) / C'_{w}$ and $B_1$ is shown in Figure 4, where $C'_{w}$ was taken as the average estimated value for each unit. A rather consistent trend is found, and a non-linear increase of $N_{ua}$ with $B_1$ is observed. Tentatively, a third order polynomial function is adopted to fit the plotted values.

6 OVERCONSOLIDATION RATIO

The OCR of Units 1, 3 and 5 was investigated by oedometer tests on good quality samples, but useful indications can also be obtained from the cone penetration records and from measurements of the undrained shear strength. OCR values estimated from $C'_{w}$ (Figure 1) and based on the relationship $OCR = (C'_{w} / 0.23 \times \sigma_{w})^{0.5}$ (e.g. Janiszewski et al. 1985) are plotted in Figure 5.

In the very soft Delta-front clayfill, OCR's larger than one are estimated, with relatively high values next to the ground surface. Note that in this unit the plotted data are sensitive to minor variations of the water level, and therefore they should be considered indicative only. Four oedometer tests were carried out in Unit 1 on tube samples recovered at 3 and 3.3m of depth. By interpreting the results as suggested by Nagraj et al. (1999) to keep into account the effects of sample disturbance, a pre-consolidation pressure $\sigma_0'$ between 30 and 400 kPa is estimated. On the other hand a value on the order of $44$ is estimated applying the relationship between $C'_{w}$ ($'$) and $\sigma_0'$ suggested by the same authors, based
on observations carried out on a number of Canadian sensitive clays. Although some mechanical overconsolidation due to minor water-level fluctuations is possible, the geological history of the area suggests that Unit 1 should be essentially NC. Therefore the nearly constant strength and cone resistance with depth are probably due to apparent overconsolidation related to the considerable organic content of the Delta-front unit. This phenomenon is often observed in soft seafloor deposits. The presence of increasing apparent overconsolidation towards the surface was also observed by Shotton (1995) in Holocene estuarine deposits in Fiji, and even in that case the presence of organic matter was considered a possible cause.

In the Pro-delta clay the linear increase of $C_{uu}$, $q_{uu}$ and $n_{u}$ with depth suggests a NC condition. The estimated OCR values are rather scattered, but close enough to unity to support the assumption of normal consolidation. Six oedometer tests interpreted by the Casagrande method give OCR's between 0.54 and 1.03. These low values are probably related to sample disturbance rather than to underconsolidation.

The OCR values of the Pleistocene OC clay are widely scattered. The average value, on the order of 3-4, is close to OCR-3.5 resulting from an oedometer test on a sample recovered at 49m of depth. In this case the negative $B_{v}$ values suggest an OCR larger than real, as discussed in Section 5.

7 SOIL CLASSIFICATION BASED ON CPTU RECORDS

The classification system proposed by Robertson (1990) was applied to the CPTU records, and the results are shown in Figure 6. The net cone resistance $q_{net}$, divided by the vertical effective stress $s_{v} = f_{s}$/ $q_{s}$, is plotted vs. the normalised sleeve friction $f_{s}$/ $q_{s}$ (Figure 6a) and vs. the pore pressure ratio $B_{v}$ (Figure 6b), respectively. The data obtained in the surface Delta-front clay and silt are quite scattered, and fall mostly within Zones 3 (clay, clay to silty clay) and 4 (silt mixtures, clayey silt to silty clay). Few points also show up in Zone 5 (sand mixtures, silty sand to sandy silt) but only in the $B_{v}$ diagram. These results are reasonable for the Delta-front soft unit, which is composed of both silt and clay. The more homogenous Pro-delta clay, on the other hand, falls completely within Zone 3, as appropriate. As for the Pleistocene OCR clay, it also falls within Zone 3 in the $f_{s}$/ $q_{s}$ diagram, and occupies a higher $q_{net}$/$s_{v}$ location with respect to the Pro-delta clay, which is consistent with its higher OCR. In the $B_{v}$ diagram, the Pleistocene clay falls in Zone 4 (silt mixtures-clayey silt to silty clay) with considerably large $B_{v}$ negative values.

The adopted classification system appears quite valuable, and suitable for the Nile Delta cohesive
soils, and only minor inconsistencies are observed between the $f$, $N_q$ and the $R_q$ diagrams.

8. CONCLUSIONS

A geotechnical investigation was carried out in the Eastern Nile Delta, in Egypt. The site is characterized by a 30-35m thick deposit of soft clays/silts with sand interlayers, resting on Pleistocene stiff clays/sands of marine origin. The results of the investigation lead to the following conclusions:

1. The Delta-front silts/clays (from 0 to -7m), which are characterized by a sensitivity up to 3.2 and by a liquidity index generally in excess of 1, display a significant amount of apparent over-consolidation, probably related to their organic content.

2. The Pro-delta clay (from -12 to -35m) is nearly normally consolidated. The undrained shear strength compares reasonably well with the empirical correlation $C_s = 0.23 \times \sigma_{v0}$, applicable to NC inorganic soil.

3. The deep, impermeable Pleistocene marine clay (below -35m) is overconsolidated with average OCR on the order of 3.5.

4. A reasonable prediction of the undrained shear strength from CPTU cone resistance records can be obtained by adopting $N_q$ values of 18 in the top Delta-front clay, 26 in the NC Pro-delta clay and 12.5 in the deep OC Pleistocene clay (this last value provides a weak correlation due to wide data scattering).

5. Considerably high pore water pressures were recorded in the soft deposits during CPTU penetration, where low and even negative pressure values were measured in the deeper OC clay.

6. The use of pore pressure records to provide an estimate of $C_s$ appears promising. A tentative correlation between the pore pressure ratio $R_q$ and $N_{qc} = (u_2 - u_0)/u_0$ is proposed for the Nile Delta soils.

7. An established CPTU-based soil classification system was evaluated with reference to local soil conditions. The system works quite well in the Nile-Delta soils, and only minor inconsistencies between sleeve-friCTION based and pore-pressure based classification charts are observed.

ACKNOWLEDGEMENTS

The information presented in this paper is published with the permission of the Petroleum Academy Company, Egypt, which is gratefully acknowledged.

REFERENCES


Field and laboratory testing program for the seismic evaluation of Tinemaha Dam, California

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Sinan Iotel
Dames and Moore, Los Angeles, Calif., USA

ABSTRACT: This paper is a case study of a site investigation for the Tinemaha Dam (Dam) to determine material properties and characterize the site for further analysis. Due to concerns regarding the seismic stability of the Dam a comprehensive field and laboratory investigation was conducted. As a part of the investigation of the Dam, static and seismic Cone Penetration Tests (CPT), Standard Penetration Tests (SPT), seismic down hole velocity and seismic refraction survey lines were conducted by the LADWP and Dames & Moore consultants (D&M). Normalized standard penetration blow count $N_{600}$ determined from CPT correlations was compared to SPT values. Soil fines content from laboratory data was compared with CPT interpreted fines content. Also the maximum shear modulus ($G_{MAX}$) was determined from shear wave velocity data ($V_s$), various CPT and SPT correlations and from laboratory data; these $G_{MAX}$ values were compared with one another.

1. INTRODUCTION

The Dam and Reservoir owned and operated by the LADWP, is a part of the Los Angeles Aqueduct System which transports water from the Owens Valley to Los Angeles. The Dam, which was completed in 1932, is constructed primarily of alluvial materials borrowed from pits located upstream and downstream from the Dam. Tinemaha Dam creates a reservoir, which is nine miles southeast of the town of Big Pine (Figure 1).

The Dam is approximately 1,680m (5500') long with a maximum height of 10.3m (34'). The Dam is a wagon-rolled compacted earthenfill embankment with approximately 1.5-foot thick lifts. The as-constructed section has 2.5:1 (horizontal to vertical) fill slopes on the upstream and downstream sides. Lava rock was placed on the upstream face of the Dam to prevent erosion. The upstream lava rock
The crest of the Dam is at elevation 1183.3m (3882.08) and the lip of the spillway is at 1180.33m (3872.58).

The normal operating water surface of the reservoir has been limited to elevation 1178.35m (3866.08) due to concerns related to the seismic stability. As a result of lowering the water surface 1.98m (6.51) below the spillway, the water storage capacity has been reduced from 19.77x10^6 m^3 (5000-ac-ft) to 7.45x10^6 m^3 (2000-ac-ft).

Tiaenua Reservoir is located in the Oweis Valley. The Iroyo Mountains are located to the east and the Sierra Nevada Mountains are on the West of the dam. The alluvium layer underlying the Embankment is comprised of Quaternary Valley fill (Figure 2a,b).

2. FIELD TESTING

A field exploration program was conducted, which included: 17 CPTs, 5 boreholes with SPTs, seismic down-hole velocity, and 3 seismic refraction and reflection surveys (Figure 2a).

2.1 Seismic Cone Penetration Testing

Seismic CPTs were performed along the Dam axis and downstream toe (Figure 2a). The seismic CPT was used to measure compression and shear wave velocities in addition to tip resistance (Q_t), skin friction (f_s), and pore pressure dissipation rates. The results of measured shear wave velocities were used to calculate the variation of maximum (small-strain dynamic) shear modulus (G_{max}).

The shear wave velocities (V_s) estimated from these surveys are summarized in Table 1. The seismic recordings were conducted at 1.5m (5ft) depth intervals, from 3m (10ft) below the surface down to CPT refusal. CPT refusal generally occurred within 15m (50ft) of the surface in coarser-grained alluvium.

Figures 4 to 7 are a representation of longitudinal Section AA of the Dam (Figure 2). The limits of the Figures 4 to 7 range from Stn. 5+00 on the East to 55+00 to the West of the Dam, and the depth the data is shown to is approximately 18.3m (60ft). Figure 4 shows a plot of CPT tip resistance.
contours in tsf (1tsf=95.8 kPa). The average values for the Q in the embankment is approximately 5,746 kPa (60 tsf) and the average Q in the alluvium is typically greater than 22,983 kPa (250 tsf). The Friction Ratio (Rf) for the embankment was approximately 2.5%, and about 1.0% for the alluvium (Figure 5). The embankment is classified as an SM-ML, silty sand to sandy silts. The alluvium is a SP-SM, well to poorly graded sands with silts.

The contours (for Figures 4 to 7) were developed using GMS (Ground Water Modeling System) a graphical modeling software for site characterization, geostatistics, and mesh and grid generation. GMS interface was developed by the Engineering Computer Graphics Laboratory of Brigham Young University in partnership with the U.S. Army Engineer Waterways Experiment Station. Ordinary Kriging was used to interpolate the current data.

2.2 Standard Penetration Testing

Standard Penetration Tests were performed at five locations, 196-1 to 196-5, along the Dam axis (Figures 2a & 2b). SPT intervals of 1.5m (5ft) were selected and pitcher tube samples were retrieved between the SPTs.

The distribution of energy- and overburden-corrected SPT blow counts (N160) is shown in Figure 6 along the axis of the Dam. Figure 6 also compares the measured SPT blow counts with a contour plot of N160 values estimated from CPT data according to empirical relations by Robertson et al. (1983). The measured and estimated N160 values compare reasonably well. The average N160 value in the embankment is 11 blows per 0.3m (1ft), and the average N160 value in the alluvium is 43 blows per 0.3m (1ft).

An additional down-hole seismic survey was conducted in borehole 196-3 down to a depth of 36.6m (120ft). The shear wave velocities were summarized in Table 1.

### 3. LABORATORY TESTING RESULTS

Laboratory tests were performed on undisturbed soil specimens retrieved from the boreholes using the Pitcher sampler. The samples were sealed in the field to preserve the field moisture content and carefully transported to the LADWP Material Testing Laboratory (MTL).

The laboratory testing program consisted of classification of soils, moisture density, in place density, consolidated undrained static and cyclic triaxial compression tests. The laboratory tests provided data on the static and cyclic stiffness and

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Standard Comp.</th>
<th>cone tip</th>
<th>Drilled Rate</th>
<th>No. of blow</th>
<th>Penetration</th>
<th>V不慎</th>
<th>Vmol</th>
<th>V&lt;sub&gt;c&lt;/sub&gt;</th>
<th>V&lt;sub&gt;c&lt;/sub&gt;</th>
<th>D&lt;sub&gt;c&lt;/sub&gt;</th>
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<tr>
<td>6.0</td>
<td>36</td>
<td>270</td>
<td>8000</td>
<td>500.0</td>
<td>1.382-06</td>
<td>0.35</td>
<td>3.65-02</td>
<td>2.00×06</td>
<td>7.00×06</td>
<td>1.00×06</td>
<td>36.3</td>
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<td>25</td>
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<td>1000</td>
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<td>7.00×06</td>
<td>1.00×06</td>
<td>36.3</td>
<td>0.00</td>
</tr>
</tbody>
</table>
strength of the embankment fill and underlying alluvial soils.

3.1 Finer Content

The fines content, i.e. soil particles finer than D=0.075mm (No. 200 sieve), was determined in the laboratory and compared to the fines content as estimated by CPT sleeve skin friction and friction ratio to the fines content. Charts developed by Robertson (1990) based on previous correlation studies were used. The laboratory fines content were taken from SPT samples recovered from boreholes B96-1 to B96-5. Figure 7, shows that the interpreted fines content generally agrees with the measured data, except at shallow depths (less than 3.5m) and a particular zone at a depth of about 15.2m (50ft) towards the west abutment of the Dam. The average percentage of fines finer than No. 200 sieve, was approximately 50% by weight in the embankment fill and approximately 20% in the alluvium (Figure 7).
3.2 Field Densities

The in-situ density of the soil samples was obtained by the Drive Cylinder method by hydraulically advancing 7.62cm (3in) diameter Pitcher Tube samples. All soil samples were classified and sorted by group symbol. Table 1 shows field density variation with depth.

3.3 Isotropic Compression Tests

Consolidated Undrained Triaxial Compression tests were performed on undisturbed soil samples at various depths, from bore holes B96-1 to B96-5. The samples were isotropically consolidated at a specific confining pressure while a drain valve from the sample was open to a volume change measuring device. The confining pressure was incrementally increased and the corresponding sample volume change was measured and plotted versus time. The final volume, which was the volume at equilibrium, was plotted against the confining pressure.

Plots of volumetric stress-strain relations for various samples were made and bulk modulus for various soil types were obtained. The soil bulk modulus was obtained by constructing the initial tangent to the experimental curves. The slopes of the initial tangent to the experimental curves (Ks) are provided in Table 1. These values are defined as static bulk modulus.

3.4 Consolidated Undrained Triaxial Test

Consolidated Undrained Triaxial Compression tests were performed on undisturbed soil samples at various depths, and from test holes B96-1 to B96-5. The samples were isotropically consolidated and sheared undrained in compression at a constant rate of axial deformation (strain controlling). The total and effective stress on the test specimen was calculated by measuring the axial load, axial deformation, and pore water pressure. Generally, at least three specimens (in some cases four) were tested at different consolidation stresses to define the strength envelope.

The total and effective cohesion and the total and effective friction angles were calculated by drawing a straight line tangent to the Mohr's circles for each set of specimens. The Mohr-Coulomb total and effective failure envelope along with φ′, c′ obtained from the consolidated-undrained triaxial tests, are presented in Table 1. For the purposes of design calculations, the effective stress failure envelope φ'' and c'' were used.

Table 1, shows the variation of initial modulus of Elasticity with depth. In general, the test data shows a linear variation of modulus of elasticity with depth. The modulus of elasticity of the alluvium material is significantly higher than the embankment material.

4. Gmax Correlations

The seismic CPT was used to determine the incremental shear wave velocity, which was then used to determine the maximum shear modulus (Gmax) as follows:

\[ G_{\text{max}} = \rho \cdot V_s^2 \]  

Equation 1

A unit weight of 18.1x10^3 KN/m^3 (115 pcf) was used to calculate Gmax. Gmax values were also computed at each borehole location using an SPT-based correlation from Sykora (1987):

\[ G_{\text{max}} = 20000 \cdot (N_1)^{0.23} \cdot (\bar{f}_c)^{0.3} \]  

Equation 2
Where $Q_{max}$ and $\sigma_{n}'$ are in psf, and $\sigma_{n}'$ is the effective mean pressure.

\[ Q_{max} = 1634 \times (\sigma_{n}' / 10) - (p)_{a}^{0.75} \]  
Equation 3

Where $Q_{max}$, $\sigma_{n}'$, and $p_{a}$ are in kPa and $\sigma_{n}'$ is the effective overburden pressure.

$Q_{max}$ was also estimated using the laboratory testing results based on the following published relationships by Jamialikowski et al. (1991):

\[ Q_{max} = 625 \times e^{-5.5} \times (OCR)^{0.5} \times (\sigma_{n}' / 10)^{0.5} \]  
Equation 4

Where $e$ is the void ratio, OCR is the Over Consolidation Ratio (assumed OCR=1.0), $\sigma_{n}'$ is the effective mean pressure, and $p_{a}$ is the atmospheric pressure.

Figure 3 compares plots depicting the variation of $Q_{max}$ along the axis of the Dam based on field measurements versus empirical correlations from in-situ and laboratory testing. As shown in the plots, the measured and estimated $Q_{max}$ values correlate reasonably well. However, the SPT and CPT correlations tend to underestimate $Q_{max}$ values at shallow depths. Additionally, SPT and CPT correlations fit the measured values of $Q_{max}$ closer than laboratory correlations for this site.

5. SUMMARY & CONCLUSIONS

Laboratory and field test results for soil samples collected from the Dam site have been summarized and presented in this paper. It was shown that the CPT results are not only reliable but provide an added advantage over the SPT by providing a continuous profile of the soil stratigraphy. Furthermore, it was shown that the CPT data can be interpreted to provide a reasonable estimate of fines content, and $Q_{max}$ for in-situ soils.

According to the results of this study, blow counts estimated from CPT data closely match measured SPT blow counts. The comparison of measured versus estimated fines content and $Q_{max}$ values seem to differ at low overburden pressures. Nevertheless, reasonable agreement between measured and estimated values was observed at greater depths. In-situ correlations provide more accurate estimates of $Q_{max}$ than laboratory correlations.

6. ACKNOWLEDGMENTS

The authors would like to thank L. A. Jackson and Mark Olson for assisting in geophysical interpretations, Jenny Wu for drafting all figures and charts. Thayne B. Devors and Joachim W. Gunther of the LADWP Materials Testing Laboratory performed all soil testing. The support of the LADWP’s Water Supply Division managers, especially Wiaston K. Wu, is greatly appreciated.

REFERENCES


Prathar, Y., et al., 1997. Laboratory and Field Test Correlations for Timna River Dam, California, ASDSO Annual Conference, Pittsburgh.


Sykora, D.W., 1987. Miscellaneous Papers, GL 8732 US Core of Engineers Waterway Experiment Station, Experiment Station, Examination of Existing Shear Wave Velocity and Shear Correlations In Soils, Vicksburg, Mississippi.


An innovative foundation in difficult ground through detailed site characterization

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ABSTRACT: At a sinkhole-prone site in State College, Pennsylvania, detailed site characterization enabled the use of an innovative foundation system for a heavy, six-level parking structure. By performing a test boring at each of the 53 columns, it was found that only six locations had soils which were weakened by sinkhole activity. Moreover, the underlying Nittany dolomite had only minor voids despite its pinnacled surface. The design parameters of the predominantly favorable soils were determined by correlating triaxial and consolidation test results with standard penetration test data and many soil classifications. A geotechnical analysis which accounted for bearing on rock and on thick overburden revealed that the majority of soils could support the proposed loads with tolerable settlement. Having established this, selective compaction grouting was implemented to improve the weak soils. Post-construction foundation settlement monitoring was conducted for a period of 14 months beyond completion of construction. This showed negligible settlements.

INTRODUCTION
At State College in Centre County, the main campus of the Pennsylvania State University (Penn State), difficult subsurface conditions have led to a prevailing local design practice: the indiscriminate use of deep foundations to support all heavy structures. The underlying bedrock belongs to the Ordovician Nittany dolomite (Berg and Dodge 1981) which is characterized by sinkhole-prone instabilities in the local area. Thus in early 1991, as design for a new six-level parking structure was underway, the need for a deep foundation was preconceived and preliminary plans called for a system of drilled piers.

Valley Forge Laboratories, Inc. was initially retained by Penn State to determine the design criteria for this system. However, as the investigation progressed, it was determined that spread footings would not only be a safe alternative but a more economical one as well. This paper illustrates how an innovative, cost effective, shallow foundation system was achieved through detailed site characterization and selective compaction grouting.

PROJECT DESCRIPTION
Completed in 1992 at a cost of $8.5 million, the parking garage is a five-level, precast concrete structure with a design allowance for adding a future sixth level. It occupies a site which was previously a parking lot east of the Eisenhower Auditorium. The ground surface slopes downward toward the southeast with an average grade of 5%, resulting in a maximum relief of 3.5 m across the site. Fig. 1 portrays the foundation layout.

The parking garage has plan dimensions of approximately 31 m by 67 m and a maximum design parking capacity of 892 cars. Its typical bay size is approximately 9 m by 17 m and the corresponding column load range is between 1800 and 6200 kN, including dead and live loads. The total number of columns supporting the structure is 53, and they are shown and labeled in Fig. 1.

GEOTECHNICAL EXPLORATION AND LABORATORY TESTS
An initial investigation consisting of 28 test borings at every other column location generally revealed favorable conditions; the overburden was mainly composed of medium to stiff clay except for one location which was weakened by sinkhole activity. This finding indicated that a shallow foundation system was a promising alternative. However, further investigation was necessary to study the sinkhole activity at the remaining column locations and develop remedial measures, if needed.

The additional investigation started by performing a test boring at each of the remaining 25 column locations (Fig. 1). The standard penetration test
(SPT) was performed continuously to a depth of 3 m below the preliminary footing bearing elevations and every 1 m thereafter, until the rock surface was reached. The underlying Nittany dolomite was cored in every boring between 1.5 and 5 m depending on the core recovery, which averaged 73%. The rock quality designation (RQD) ranged between 0 and 68%, with an average value of 28%.

Shelby tube samples were also collected at 12 locations to enable a reliable study of the shear strength and consolidation properties of the clayey soils, which dominated the overburden.

Index Tests

Index testing was performed on six selected SPT soil samples and all twelve undisturbed Shelby tube samples. This testing provided a means to define the variations in gradation, water content, plasticity, degree of saturation, unit weight, and unconfined compressive strength of the site's soils.

Classifications were assigned according to the Unified Soil Classification System (USCS), which resulted in two predominant soil types at the site—fat clay (CH) and lean clay (CL). Table 1 summarizes the variation in index properties for these soils. The index testing, in combination with SPT data, greatly facilitated the selection of samples for consolidation and triaxial testing.

Unconfined compressive strength tests were performed on 11 intact rock core specimens. The rock strengths ranged between 30,000 and 67,000 kPa.

### Table 1. Variation in index properties of the site's clays.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Fractional composition</th>
<th>Plasticity</th>
<th>Degree of saturation</th>
<th>Compressive strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fat clay</td>
<td>27-33</td>
<td>26-37</td>
<td>76-46</td>
<td>89-97</td>
</tr>
<tr>
<td>Lean clay</td>
<td>27-30</td>
<td>22-34</td>
<td>70-23</td>
<td>89-98</td>
</tr>
</tbody>
</table>

Consolidation Tests

Fig. 2 presents the results of one-dimensional consolidation tests on undisturbed samples of the site's fat and lean clays according to ASTM D 2435 (1989). The consolidation properties are summarized in Table 2. As can be seen, both soil types are heavily overconsolidated with overconsolidation ratios (OCR) greater than 10. Therefore, the consolidation plots were not corrected according to the method proposed by Schmertmann (1955) to account for field disturbance because the virgin compaction properties of the clays were not applicable for geotechnical design purposes.

It is also worth noting that the preconsolidation stresses determined on samples from different borings were very close. This has provided confidence in these values and was a key to the success of the subsequent triaxial tests.

Triaxial Tests

In order to determine the shear strength and elastic properties for foundation analysis, selected undisturbed samples of the site's clays were
subjected to triaxial testing. Isotropically consolidated, undrained compression tests with pore water pressure measurements were performed according to ASTM D 4767 (1989).

Fig. 3 shows that the entire failure envelopes for both soils were determined by testing two specimens from each soil type. By knowing the preconsolidation stresses from the consolidation testing, the specimens were sheared above and below these stresses to establish the normally and overconsolidated envelopes, respectively.

Table 2 lists the resulting effective friction angle and cohesion values of the overconsolidated envelopes. The stress-strain plots of the triaxial tests yielded an average undrained elastic modulus of 48,000 kPa.

**IDEALIZED SUBSURFACE PROFILE**

The various soils encountered at the site were represented by an idealized subsurface profile which consists of localized fill, clay, decomposed dolomite, and dolomite bedrock. The overburden thickness ranged from 1 to 13 m. Fig. 4 shows five test boring sections which illustrate the different subsurface conditions across the site. (Refer to Fig. 1 for the column locations of the depicted sections.)

Although a clear distinction could readily be made between the fat and lean clays, they were grouped into a single stratum due to their random interlayering and very similar mechanical properties (Figs. 2 and 3). Clay was the predominant material, and its thickness varied from 0.5 to 11 m. Such an extreme variation in thickness is common to sinkhole-prone sites.

The shallow clays were significantly stiffer than the deeper ones at all columns locations, even those affected by sinkhole activity. Ultimately, six locations turned out to have pronounced sinkhole activity, where the deeper clays were very soft and wet (average SPT value of 3 blows per 0.3 m).

As shown in Fig. 4, a thin stratum of decomposed dolomite (dense sand and gravel) separated the clay and the bedrock in some of the borings. The average thickness of this layer was only 1 m and it was not important in foundation analysis.

The cores of the Niutany dolomite at the site indicated a broken, thickly-bedded rock mass having 3 to 6 fractures per 0.3 m. With an average unconfined compressive strength of 55,000 kPa, the intact rock was placed into Class III—hard rock (Horng 1969). Based on an average RQD of 28%.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Initial void ratio</th>
<th>Preconsolidation stress (kPa)</th>
<th>Preconsolidation ratio</th>
<th>Overconsolidation index</th>
<th>Effective friction angle (°)</th>
<th>Effective cohesion (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fat clay (CH)</td>
<td>0.75</td>
<td>577</td>
<td>16.1</td>
<td>0.033</td>
<td>26</td>
<td>26.8</td>
</tr>
<tr>
<td>Lean clay (CL)</td>
<td>0.68</td>
<td>577</td>
<td>10.2</td>
<td>0.030</td>
<td>26</td>
<td>22.0</td>
</tr>
</tbody>
</table>

Figure 3. Triaxial test results.
the rock mass quality was poor according to Conis and Morris (1970). Only a few minor voids were found in the cored rock. However, clay-filled seams caused low recoveries in some coring runs.

FOUNDATION ANALYSIS

While evaluating the feasibility of a shallow foundation for the parking garage, it was obvious that the weak clays at the six locations of pronounced sinkhole activity had to be improved before they could support structural loads. This aside, the geotechnical design calculations were performed using the overconsolidated clay properties.

In predicting total settlements, it was recognized that a reduced value of the consolidation component should be added to the immediate elastic settlement for the following reasons. First, spread footings do not undergo one-dimensional consolidation due to dissipation of vertical loads with depth. Second, the site's clays were not fully saturated (see Table 1), and consolidation would not occur until all of the air voids are completely compressed or dissolved—a situation unlikely to occur under actual foundation loads. Third, the addition of immediate elastic settlement to one-dimensional consolidation settlement is theoretically inaccurate and overestimates the total settlement (Lambe and Whitman, 1969). Fourth, the reduction of the one-dimensional component is analogous to the settlement ratio concept developed by Skempton and Bjerrum (1957).

In addition to the above general reasons, two project-specific factors support the conception of reducing the consolidation settlement. First, all of a parking structure does not have a sustained live load; the maximum vehicle loads would only be applied during weekday hours or football games. Secondly, the cyclic loads imposed on the structure would have a stiffening effect on the foundation soils due to strain hardening.

Considering all of the preceding points, only 25% of the live loads were utilized to model the perceived reduction in consolidation settlement. This is approximately equivalent to the percentage of time during which the parking structure would be occupied.

The maximum settlement was calculated to occur under column A-10. This was primarily due to the deep clay profile at that location compared to others, as depicted in Fig. 4. Under a net allowable bearing pressure of 300 kPa the total settlement was 35 mm. Of the total, 9 mm were immediate under dead load and 4 mm were immediate under live load. Also, the dead and live load consolidation settlements were 17 and 5 mm, respectively.

The maximum differential settlements would occur between footings on soil and adjacent footings on rock. There were nine footings on rock which were predicted to undergo negligible settlement under a
design bearing pressure of 1,000 kPa, relative to those on clay. Thus, the maximum calculated differential settlement and angular distortion were 30 mm and 1/300, respectively. The values of total and differential settlement were within tolerable limits for the parking garage. Thus, it would not be the case if the full consolidation settlement were taken into account.

COMPACTION GROUTING

Compaction grouting was selected as the optimum soil improvement technique for the six locations indicated in Fig. 1 due to its established success in similar areas (Welch 1988). Grouting of the rock was performed only to a limited extent. The grouting was executed after the building pad was prepared and the footings were laid out, to improve specific column locations. Selective grouting is far more efficient and cost-effective than conventional grid-type grouting of the entire building area, although it requires a more detailed site characterization. The operation was completed in three weeks.

A low-slump sand/cement mix was used. Grouting was based on a performance-type specification, i.e., a minimum SPT value of 8 blows per 0.3 m was to be achieved in the grouted clays. This value was derived from correlation of the triaxial and consolidation samples with the SPT data. Depending on the size of the footings and the severity of the subsurface conditions at the grouted locations, three, four, or five injection points were activated, for a total of 22. Fig. 5 depicts the grouting arrangement for two of the footings, A-11 and B-10.5/11.

The grouting injection holes were advanced to the top of rock using a rotary percussion tracked rig, and 51 mm steel casings were inserted into the holes.

Grouting then proceeded upward in increments of 0.3 to 0.6 m throughout the zones which required improvement (see Fig. 4). Each level was grouted until a pressure of 1,400 kPa was attained or an incremental ground heave of 3.0 mm was detected by surveying instruments set up about 70 m away. The grout had a maximum slump of 75 mm and a 28-day compressive strength of 10,000 kPa.

Upon completion, the degree of soil improvement was checked by SPT verification at each grouted location, as depicted in Fig. 5. The minimum SPT value of 8 blows per 0.3 m was achieved at all locations. Fig. 4 shows a comparison of SPT values before and after grouting for A-11 and B-10.5/11. In the end, 190 m² of low slump grout were injected to stabilize the site’s weak clays.

POST-CONSTRUCTION SETTLEMENT MONITORING

Following completion of the parking structure, a post-construction settlement monitoring program was initiated to obtain foundation performance data. Ten columns were selected to represent the diverse subsurface conditions and structural loading at the site. Three were grouted locations, while the remainder represented column footings on clay, bedrock, and decomposed dolomite. For 14 months after construction, monthly monitoring was performed using surveying equipment with an accuracy of 3 mm.

Only negligible settlements were recorded, as shown in Table 3. It is interesting to note that settlements on clay settled the most, about 6 mm. Conversely, settlements on rock did not settle at all. Footings on decomposed dolomite as well as footings on grouted soil settled halfway in

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**Figure 5.** Arrangement of compaction grouting injection points and verification tests.
between—about 3 mm. Although not shown in Table 3, all of the settlement measurements essentially stabilized after only one month of monitoring. It is also worth pointing out that certain monthly readings actually showed ground heave. This is explained as a structural response of the parking garage to uneven loading at various times during the monitoring.

CONCLUSIONS

In spite of difficult sinkhole geology and contrary to local design practices, a conventional shallow foundation supports one of the heaviest structures on Penn State’s main campus. In fact, the parking structure discussed in this paper was the first on campus not to be constructed on drilled piers. This valuable precedent was achieved through detailed site characterization and a scientific approach to the geotechnical investigation. Considerable savings were realized without sacrificing performance.

The foundation system has performed better than expected. In two ways, the monitoring data led to the conclusion that consolidation did not occur at the site. First, the readings stabilized almost immediately after construction, whereas rough calculations using a one-dimensional approximation would have predicted about 3 years of consolidation. Second, the recorded settlements were equivalent to the predicted elastic ones under live load.

The conclusion that consolidation settlement did not occur in a geologic setting in which it is typically presumed to take place is remarkable! The reduced consolidation conception qualitatively captured some of the underlying causes, such as three-dimensional effects, the partially saturated nature of the clays, and the lack of a sustained live load. Nevertheless, this approach was still too conservative because the 22 mm of predicted consolidation settlement did not occur. Only immediate elastic settlements took place. Without the reduced settlement approach, the calculated consolidation settlement would have been 37 mm, which would have eliminated spread footings from being considered altogether. All of this underscores the need for further study and understanding of the project-specific factors qualitatively discussed in this paper.

ACKNOWLEDGMENT

The writers extend their sincere thanks to Robert E. Cooper, the project manager with Penn State, for his support throughout all phases of the project. The cooperation of Theodore R. Newell of Ewing Cole Cherry, the architect, and Noli Aleacon of Walter Parking Consultants, the structural engineer is appreciated. Finally, the competence of the grouting crew of Hayward Baker, Inc. is acknowledged.

REFERENCES

Site characterization for reclamation project in North Jakarta

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ABSTRACT: Soil investigations were carried out on the well known very soft marine clays in the reclamation area. Several in-situ and laboratory tests were conducted. The study was to provide information on the characteristics of soil profiles and soil properties, comment on results, discussion and recommendation on the soil parameters for design use. Special attention were directed for the use of CPTU to facilitate the soil profiling work in the deeper water.

This paper summarize the results of soil investigations and correlate properties, discuss the comparison of soil parameters in laboratory and in-situ tests and their use for design. Soil properties were plotted with depth and several trend lines were drawn. The geology and formation of the Jakarta marine clays were also of interest. Method of analysis is by data collection for correlations of properties, plotting of properties versus depth, comment on consistency of data, comparison with published empirical correlations and use of engineering judgment to achieve the goals.

1. INTRODUCTION

Due to demanding area for business and residence, reclamation projects have been extensively developed and proposed in northern Jakarta area. The reclamation will be carried out by placing hydraulic sand fill on very soft marine clays. Depth of water at this present stage range from ~2.0 m to ~5.0 m.

The present soil investigation is for Axaol Bera reclamation project, mainly for phase I and phase II. Most boroholes were carried out to 30 m depth, as hard layer were found at this depth. This plan owed to the fact that previous soil investigations, mainly at the location of Tanjung Priok harbor, east of the project site, show consistent soil profiles, hence it is anticipated that both phase I and phase II will subsequently yield similar results, and the data will be well correlated.

The data for this study is based on actual soil investigation reports at phase I and phase II by PT. Sofeco, summary of soil profiles at Tanjung Priok by PT. Wierman, soil investigation at phase "O" and further soil tests by PT. Trisakti Indonicka. The project site is located in the Jakarta bay, west of Tanjung Priok harbor. Site location and locations of soil borings are shown in Fig. 1 and Fig. 2.

2. SOIL CONDITION AND SOIL PROFILES

Based on results of borings, CPT and CPTU, and field vane shear tests (VST) the soil profiles were established. The upper soil strata consists of very soft grey silty clays mixed with shell debris with thickness of 6.0 to 12.0 m. N-SPT values range from 0 to 3 and tip resistance of the CPT and CPTU 15.0 to 6.5 kN.

The underlying strata is brown and grey silty clays with medium to stiff and very stiff consistency. The thickness vary and sometimes interbedded with silt and sand lenses. This firm strata is frequently known as the Pliocene formation in the Jakarta coast.

The marine clays were deposited in a low energy environment and hence mostly dominated by fine grained material known as Jakarta marine clays which is highly plastic and very compressible. The process of sedimentation of the marine soil has been influenced by changes of sea-level during the end of Pleistocene and Holocene period.

Of particular interest is the existence of underlying stiff silty clays below the soft marine sediment. It was believed that the water level about 18,000 years ago was 140 m below its
present elevation (Nakagawa, 1977). The rising of sea water level after post glacial period was described by Fairbridge (1961) as shown in Fig. 4.

The sedimentation of Jakarta marine soils were started with the sea-water increase during glaciation. Underneath the soft clay deposit, stiff silty clays of 2.6 - 5.0 m thickness were found. This has been made possible due to the lowering of sea water level down 20 m below the present level. This stiff strata was formed when the marine soils were exposed due to regression of the sea water. Similar condition was also true in Singapore marine soils (Kobayashi et al., 1990). Generally, at the bottom stratification, sandy materials were found due to highly gradient during this geological process.

3. RESULTS OF CPT AND CPTU

In the soft layer, the use of SPT is not very useful. Several boreholes were drilled without SPT measurements in the upper 10 m, however at every boreholes, CPT and CPTU were conducted. The use of CPT and CPTU were to obtain continuous profiles. CPTU was mainly applied to get better interpretation of the soil stratifications and provide information on the excess pore water pressure during penetration. At some locations, dissipation tests were conducted to get the consolidation characteristics and coefficient of consolidation in radial direction.

The use of mechanical CPT show that no tip resistance were measured to a depth of 4.0 m below seabed because the soil were too soft to carry the weight of the penetrometer itself.
4. SOIL PROPERTIES WITH DEPTH

Due to its own weight, there is a tendency that the density of the soft marine clays increases with depth and on the other hand, the void ratio and water content decreases. The bulk density of the soils range 1.2 – 1.8 kN/m³ with average of 1.4
increasing depth. The swelling index is in general low, ranging 0.05 to 0.1 with average of Cr = 0.13. The results of consolidation tests for preconsolidation pressures were scattered but they give indication of slightly overconsolidated clay to depth of about 5.0 m. The overconsolidation ratio reaches about 8 near the surface and drops rapidly with depth.

3. SHEAR STRENGTH

The undrained shear strength of the Jakarta marine clays is an intrinsic function of the effective shear strength and hence is affected by the pore water pressure during loading. A different stress path will result in different pore pressure response and might give a different shear strength value. The most ideal situation is if one simulates the actual stress path expected to occur in the field. To characterize the shear strength of the Jakarta marine clays, field vane shear tests were conducted as well as laboratory triaxial and unconfined compression tests. Shear strength profiles were then plotted versus depth. Ratios of shear strength versus effective overburden pressure give an average value of 0.22. The sensitivity, defined by the ratio of peak versus remolded shear strength were measured. The higher values that were measured by vane shear may be due to existence of shell debris. This gives erroneous results of sensitivity of higher than 20. However the more appropriate sensitivity fall in the range of 4 – 5 (Rahardjo, 1995). This corresponds to the liquidity index of 0.7 – 1.1.

0.0 m at the seabed level to 1.6 m at a depth of 16 to 18 m. The void ratios of soft soils vary from 1.5 to 3.8. This variation is wider near the seabed level and were generally in closer range at deeper level (12.0 – 18.0 m below seabed).

The plastic limits were in narrow range of 30% - 50%, however the liquid limits were scattered from 50% - 120%. The natural water content of the soils were mostly closer to the liquid limits, indicating that soils were in very soft consistency, very compressible, low shear strength and sensitive. The compression index, Cc, was generally high at seabed level and range from 0.4 to 1.6, and then lower with
6. CORRELATIONS OF INDEX PROPERTIES WITH COMPRESSION INDEX

Since composition will be one of the main issues for reclamation, the use of proper compression index is of main importance. For the upper soft clays, a range of Cc = 0.6 – 1.6 were found. However the compression index for lower stiff clays range 0.2 – 0.8. The correlations were established with natural water content, void ratio, Plasticity Index and Liquid Limits of the soils. They show sufficiently good correlation for use in design.

7. UNDRAINED MODULUS

A series of settlement records were obtained from the phase "D" reclamation work consisting of 3.5 hectares of land fill. This record gave immediate as well as long term settlement. By use of the previous monitoring data, we can ascertain that the modulus of the soft clay is around 1.0 – 1.5 kPa. The laboratory tests gave the same range of values. This value will result in a 20% – 22% immediate settlement due to placement of sand fill.

8. COEFFICIENT OF CONSOLIDATION

Coeficient of consolidation had been interpreted from the results of laboratory consolidation test and a range of 8.2E-04 to 1.0E-03 cm/sec was resulted.

When acceleration of consolidation is to be achieved by use of vertical drain, the coefficient of consolidation for radial direction will be important. The use of dissipation test during CPTU has been utilized to infer this parameter. Only 4 dissipation tests were regarded as successful and may be interpreted properly. The results give an average higher radial coefficient of consolidation than in the vertical direction. The average ratio of Cb over Cv is 2.5, which is reasonable for use in the design of vertical drain for soil improvement.
9. CONCLUSIONS AND RECOMMENDATION

1. The upper soil strata consists of very soft silty clays with
thickness ranging from 6.0 m to 12.0 m. This layer is
compressible and sensitive that a slight disturbance will
cause reduction in shear strength. The sensitivity of the
soft clay will cause high excess pore water pressure upon
loading on this layer and might cause sudden failure.

2. The shear strength increases with depth as indicated in
the plot of strength vs. depth. For analysis, a ratio of
Sv/Sn = 0.33 may be used, other wise an average shear
strength of S = 5 kPa should be used.

3. The undrained modulus is very important to estimate the
magnitude of immediate settlement. A proper estimate is
necessary to anticipate the volume of additional sand fill
material during placement.

4. Due to high response of pore water pressure during
loading, care should be taken on sudden failure of the
soft clay during construction.

REFERENCES

Marine Clay at Changi East," Proceeding, XIV International
Conference on Soil Mechanics and Foundation Engineering,
Hamburg, Germany.

Kohayashi, Y., Todo, H., Wesslinghe, WAY., Chandra, P.,
"Comparison of Coastal Clays Found in Singapore, Malaysia,
and Indonesia," Proceeding, 10th Southeast Asian
Geotechnical Conference, Taipei, April, 16 - 20, 1990.

Nikalaga, H., 1997, "Chronology and Correlation of
Quaternary, The Quaternary Period: Recent Studies in Japan,"
University of Tokyo Press.

View Reclamation Project, Sunabaya," Testana Engineering

Rahmado, P.P., 1996, "Review of Soil Investigation for

Rahmado, P.P., "Characteristics of Marine Deposit in Jakarta
and Sunabaya," Proceeding Seminar on "Foundation Design &
Improvement Techniques.

Soefcu, P.T., 1996, Factual Report on Soil Investigation for
Ancol Baru Reclamation Project, East Ancol, Jakarta.

Testana Industriken, P.T., 1996, Soil Investigation for Phase
"A", Ancol Baru Reclamation Project, Jakarta.
Geotechnical investigations to characterize the Upper Quaternary Basin of Venice

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ABSTRACT: The design of the foundations of the mobile barriers protecting the city of Venice against flooding require an accurate knowledge of the geotechnical parameters of the quaternary basin and particularly of the cohesive formations, which are the most responsible of the settlements and displacements induced by the barriers in to the ground. This paper presents some preliminary results of a comprehensive geotechnical soil characterisation, which has been carried out by site and laboratory investigations.

1 INTRODUCTION

It is probably world-wide recognised that the city of Venice shows a precarious equilibrium and that the margin of security is being eroded annually at an increasing rate. The rate of deterioration is being accelerated by the increasing frequency of the flooding of the old city and by the reduction of the firebreak of the city as a result of the eustatic rise in sea level coupled with a subsidence of the general lagoon area (Ricceri and Butterfield, 1974).

In the early eighties, the Italian Government decided to finance the design of special mobile barriers - located at the three inlets of the lagoon - which should be able to protect the old city and the entire lagoon against the flooding due to the high tides.

The selection and design of the barrier foundations require an accurate knowledge of the geotechnical parameters of the upper quaternary basin, particularly of the cohesive formations, which are the most responsible of the settlements under the barriers.

To this purpose a comprehensive geotechnical investigation was carried out in two phases.

The first phase was considered as a preliminary investigation (to be used for the preliminary design of the barriers) and was performed in correspondence of the three lagoon inlets by using some standard investigations.

The second phase was decided in order to characterise more accurately the Venetian subsoil.

However, the high costs of boreholes and laboratory tests suggested a more extensive use of the less expensive in situ tests.

Unfortunately, in situ tests not always provide direct determination of soil parameters but require empirical correlations or interpretation methods in order to transform the in situ measured quantity (e.g. the tip penetration resistance in the case of cone penetration test) in the selected design parameters.

In order to establish reliable correlations or to validate available interpretation methods, a calibration between in-situ determined quantities and soil parameters, accurately estimated from laboratory tests on undisturbed samples, was considered as necessary.

Therefore, before beginning with the second phase, a special geotechnical investigation was planned: the so called "Geotechnical Calibration Station" (GECAST). In a limited area at the Malamocco inlet, deep boreholes together with piezocone, dilatometer, self-boring pressuremeter, and cross/drown hole tests were carried out on nearby verticals. In addition, in order to obtain high quality undisturbed specimens, it was decided to used also a new large diameter sampler.

This paper shows some preliminary results of the geotechnical investigations carried out through the investigation carried out at the GECAST. More particularly the paper considers the cohesive soils, which are the most responsible of settlements under the barriers.

2 BRIEF GEOLOGICAL HISTORY

The quaternary deposits of the Venice Lagoon, reaching a depth of approximately 900-950 m, have
been formed throughout the Pleistocene. They are composed by a complex system of interbedded sands, silts and salty clay sediments. Their accumulation took place in different phases, during which marine regression and transgression alternated and the rivers transported fluvial materials coming from the nearby Alps (Carri, 1995). In the twelfth century, when the first future citizens of Venice settled on the islands, the rivers Brenta, Bile, Piave and other discharges discharged waters and sediments into the venetian lagoon. In order to prevent this, the rivers were diverted into extensive canals around the lagoon periphery. After eighteenth century no further hydraulic works were carried out and, therefore, the lagoon was not subjected to any significant alteration.

3. THE GEOTECHNICAL CALIBRATION STATION

The Geotechnical Calibration Station was located in a small area at the Malaùocco inlet, where the following types of test have been carried out: standard boreholes (S), large diameter boreholes (g), piezocone (Pz), dilatometer (D), self-boring pressure-meter (Pa), down hole (DH) and cross hole tests (CH).

The geotechnical laboratory investigations were carried out by the University of Padova and ISMES of Bergamo in cooperation with the Magistrato alle Acque and the Consorzio Venezia Nuova (Magistrato Alle Acque, Ministero dei Lavori Pubblici, 1994).

Figures 1 and 2 show, respectively, the location of the GECASST at the Malaùocco inlet and the area of the investigations.
4 SOIL PROFILE

Figure 3 depicts a tentative soil profile, up to 60 m below mean sea level, determined from a borehole log, compared with the results of a piezocene test (q_c = tip resistance, u_w = pore pressure), carried out on a vertical close to the borehole.

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>q_c (MPa)</th>
<th>u_w (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 WET SUEDE SILT SAND</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2 HIGH C.C. CLAY</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>3 ALT-TENDING LEVELS OF SLIP CLAY AND CLAYEY SILT</td>
<td>40</td>
<td>60</td>
</tr>
<tr>
<td>4 FINE SILT SAND WITH HUMILIARY LAMINATIONS OR CLAYEY SILT</td>
<td>60</td>
<td>80</td>
</tr>
<tr>
<td>5 ALTERNATING LAYERS OF CLAYEY SILT AND CLAYY CLAY</td>
<td>80</td>
<td>100</td>
</tr>
<tr>
<td>6 FINE SILT SAND</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>7 CLAYEY SILT</td>
<td>120</td>
<td>140</td>
</tr>
<tr>
<td>8 ALTERNATING LAYERS OF SLIP SAND AND CLAYEY SILT</td>
<td>140</td>
<td>160</td>
</tr>
<tr>
<td>9 FINE SAND</td>
<td>160</td>
<td>180</td>
</tr>
<tr>
<td>10 ALTERNATING LAYERS OF CLAYEY SAND AND CLAY</td>
<td>180</td>
<td>200</td>
</tr>
<tr>
<td>11 FINE SAND</td>
<td>200</td>
<td>220</td>
</tr>
</tbody>
</table>

Figure 3. Soil profile at the GECAST.

On the basis of the comparison between the results of the piezocene test and the borehole log, 11 basic formation were selected.

Figure 4 sketches the variation with depth of some soil characteristics determined on samples coming from standard and large diameter boreholes: particle size composition, bulk density (P), Atterberg limits (LL, PI), natural water contents (w). It can be noted that the great majority of samples, other than silts, fall in the category of silts (P<10%, LL<35%) and very silty clay (10<P<25%, 35<LL<50%). From the moisture content profile it is evident that the subsoil has a water content slightly decreasing with depth but always above the plastic limit.

5 STRESS HISTORY

The stress history was reconstructed at the GECAST using both laboratory and in situ tests.

The trend of effective overburden stress σ_0' with depth, estimated using the values of bulk densities determined in the laboratory, is reported in Fig. 5. Fig. 5 also shows the values of preconsolidation stress determined from oedometer tests performed on standard and large diameter samples. They are compared with those estimated using the results of a dilatometer test carried out close to the large diameter borehole.

An appreciable decrease of OCR was clearly noted. The high OCR values (>10) in the formation 2 are characteristic of the well known curanto, an high OC oxidated clay on which most historical venetian buildings are founded.

The in situ stress state was estimated by considering the coefficient of pressure at rest K_s at

Figure 4. Variation with depth of some soil characteristics.

Figure 5. OCR and K_s trends against depth.
various depths in the ground. $K_s$ was determined from dilatometer and self-boring pressuremeter tests and from uni-axial recompression stage in computer controlled CKD/DU triaxial tests. The $K_s$ values from triaxial tests showed to be independent on depth and always lower than dilatometer and pressuremeter ones. This is probably due to the effects of laboratory recompression (Mayne and Kulhawy, 1982) and to the stress relief induced by sample disturbance, which is particularly important in silty formations (Ricceri et al., 1985).

6 STIFFNESS EVALUATION

The evaluation of soil stiffness can be performed using many methods, both in situ and in the laboratory.
Considerable differences are seen among the deformability values obtained from the different methods. In some cases the differences could be tenfold or even more. For example, the stiffness determined from shear-wave velocity measurement may be too large in estimating the settlement under static working loading conditions, whereas the one from pressuremeter tests may be too small. This is due to the highly non-linear nature of the stress-strain relation and to the dependence of the soil stiffness on the stress history of soil.

In order to estimate the deformability of Venetian cohesive soils at the GECAST several in situ and laboratory tests were carried out.

Here, the results of the following tests are reported:
- special strain/stress controlled triaxial tests,
- resonant column tests
- dilatometer test;
- down-hole test;
- cross-hole test.

6.1 Triaxial tests

Several standard compression and extension triaxial tests were carried out. Some tests were performed at the University of Padova on large specimens using an automated computed controlled stress/strain path triaxial system. In order to improve the precision of the measure, the triaxial system is equipped by a special triaxial cell, able to measure local deformations on soil samples.

The CGA triaxial tests with internal displacement measurements were carried out on the cohesive samples coming from formations 3 and 4, consolidated at vertical stress equal to the overburden stress. (Ricceri, Simonini and Cosa, 1997). The values of the maximum shear modulus $G_s$, secant to the stress-strain curves, are reported in figure 6.

6.2 Resonant column tests

Resonant column tests have been performed at ISMES laboratory on the samples coming from the standard and large diameter borings.

The maximum shear moduli, expressed as a function of overburden stress acting in the soil, are plotted on figure 6.

6.3 Dilatometer test

The results of the dilatometer test can be used to determine the oedometer modulus of the soil.

However, in order to compare the dilatometric stiffness values with those determined from other tests, the dilatometric modulus has been transformed in the shear modulus $G$ by using the well-known relation coming from the theory of elasticity $G = 3(1-\nu^2)/(2(1-\nu))$ with Poisson's ratio $\nu$ assumed equal to 0.2. The shear modulus (plotted in figure 6) shows a trend against depth similar to that given by the cone penetration resistance (plotted on the right side).

6.4 Down-hole test

At the GECAST a down-hole test has also been carried out. The measured shear wave velocity has been transformed in shear modulus and plotted again on figure 6.

6.5 Cross-hole test

The cross-hole test has been carried out on three verticals located at the three corners of a triangle circumscribing the GECAST. The measured shear wave velocity has been transformed in shear stiffness, which has been reported in figure 6.

As expected, the various trends of shear modulus are different: it can be noted that the cross-hole, triaxial (max value) and resonant column tests (max value) tend to the highest values of shear stiffness. On the contrary, the lowest values (calculated using the relation between shear modulus and elastic modulus due to the theory of elasticity) are those from dilatometer test.

7 SOME ADDITIONAL REMARKS

On the basis of the various stiffness trends observed at the GECAST a comparison between the dilatometric and cross-hole test results and the ones of piezocone test has been tentatively carried out.

To this purpose, the values of the difference $\Delta = \alpha G - \gamma_n$ measured during the cone penetration has been transformed into stiffness values using an empirical linear relation $G = \alpha G - \gamma_n$, where $\alpha$ is a multiplier strongly dependent on the type of soil. Such types of empirical relations have been proposed in the past by several researchers (e.g. Schmertmann, 1970; Denver, 1982), but these relations have been shown to provide, in most cases, very rough estimates of the stiffness for soils.

The $G = \alpha G - \gamma_n$ values have been plotted in figures 7 and 8 together with the results of cross-hole and dilatometer tests respectively. In the first case $\alpha = 10$ and $\alpha = 20$ were used whereas in the second case $\alpha = 2$ and $\alpha = 5$ were selected. The comparison has been limited to depths varying between 90 and 10 m below.
Figure 7. Stiffness measured with cross-hole and piezocone.

Figure 8. Stiffness measured with dilatometer and piezocone.

The $G$ values sketched in figure 7 show a very large scatter between the trends of the cross-hole and of piezocone test. Therefore, no direct correlation seems, at the present, to be possible between the maximum shear stiffness determined with the cross-hole and the tip resistance of the cone.

On the contrary, a very similar trend is noted for dilatometer and piezocone test results. Values of $\alpha$ between 2 and 3 seems to provide a satisfactory estimate of dilatometric stiffness, especially for the silty formations.

However, the above considerations must be considered as a first attempt to establish some possible correlations, which may be useful for design purposes. They also suggest that the preliminary results of the Geotechnical Calibration Station will help in the selection of the most economical site investigations to determine carefully the stratigraphic profiles, strengths and stiffness parameters of the soil at the inlets of the Venetian lagoon.

REFERENCES


Geotechnical investigations for the Great man-made river project, Libya

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ABSTRACT: Phase II of the Great Man-made River Project has been under construction since 1991. The geotechnical investigations which commenced in 1995 represent one of the most comprehensive ground characterizations undertaken for a civil engineering project.

1 INTRODUCTION

The Great Man-made River Project in Libya is one of the largest construction projects being undertaken in the world at the current time. The project was conceived in 1960 and is today reaping the benefits from the exploitation of the vast ice-age freshwater aquifers which lie trapped many hundreds of metres below the Sahara Desert. This ambitious project allows this fresh water to be pumped from the ground in the desert and then transported under gravity via a buried 4 metre diameter pre-cast concrete pipeline (PCCP) to the Libyan coastal strip. (Kim and Greinach, 1994). The first water was available to supplement Tripoli’s water supply on 1st September 1996.

This project has been constructed in two main phases to date, using two pipelines which run from separate aquifers in the Libyan Sahara desert northwards to Benghazi and Tripoli respectively (Figure 1). Further phases are planned to link the two pipelines and to take water to other areas of the country as part of irrigation projects. Phase II of the project takes water from the Jabal Hasauna wellfields to Tripoli (the “Central Branch”). Nearing completion of the investigation works for this section, an additional length of pipeline was introduced to the scheme providing a long diversion around the Jabal Nefzad mountains south of Tripoli. This became the “Eastern Branch”. Further substantial investigations were commissioned to investigate this route. This paper describes the geotechnical investigations which were undertaken for the Central and Eastern Branch of Phase II.

2 PROJECT BACKGROUND

Construction of the pipeline and all associated structures and facilities, including wellfields, has been undertaken by Dong Ah Consortium (DAC) of South Korea. As part of the contract DAC were responsible for the design of the system, including the performance and supervision of the geotechnical investigations. The design has subsequently been undertaken by Gibb (formerly Sir Alexander Gibb & Partners Ltd). Approval of design and construction work is undertaken by the Great Man-made River Authority (GMRA) and their representatives Brown & Root North Africa (BRNA).

At all stages of the project undertaken, GMRA has controlled the project through detailed project procedures. DAC are required to submit an hierarchy of documents covering planning, design philosophy, specifications, design criteria, investigation factual, interpretative and design reports and detailed design. Included in this philosophy is the need for conformance to BS 5750 (subsequently ISO 9001). The ground investigations were included in this requirement.

The geotechnical investigations themselves have been undertaken by Tencel/STFA of Turkey, who won the contract in competitive tender. Gibb have supervised and directed the geotechnical investigations since 1989 until fieldwork was completed in 1997.

3 GEOTECHNICAL INVESTIGATIONS

3.1 General

The basis of the geotechnical investigations was set out in the contract between GMRA and DAC. This required that fieldwork comprising intrusive investigations, surface geophysics, surface geological mapping, and laboratory testing be
undertaken. This was to be collated and interpreted in factual, interpretative and geotechnical design reports. Each of these reports was an important milestone in the overall construction programme managed by DAC. Full approval of the geotechnical reports allowed detailed design to proceed and be approved for that section. Delays in investigation and reporting therefore had the capability of generating compound delays in construction and commissioning of the scheme. The magnitude of the costs involved in such delays are on a scale unprecedented in general engineering construction. The management of the investigation and reporting process was therefore one of the most vital tasks in ensuring the timely completion of the project.

To supervise the ground investigations GIBB provided a team of up to a maximum of 5 staff in Libya. In the UK a team of up to 6 staff was used to collate the factual data and to produce the interpretative and design reports. The GIBB organisation for the ground investigations is shown in Figure 2.
between two adjacent pipes would open as the result of the differential heave or settlement and the type of joint seal used to accommodate such movements.

The investigation was therefore directed such that, for each section of the pipeline, the 4 metres below the pipe invert was classified into different subgrade types according to the encountered materials and their compressibility or swelling potential. This subgrade zoning was used by DAC who then carried out Subgrade Verification during construction at closer spacing to establish whether any subgrade improvement was necessary before pipe installation. The prediction of heave and settlement was based on the laboratory index tests and oedometer tests. This subject is covered in more detail in Safari and Mbach (1998).

2. Trench stability

The stability of the trench has to be considered for both short term and long term as the trench excavation was sometimes left open for long periods of time before the pipe was installed and backfilled. The basis of trench design was by classification of the materials in the sides of the trench along each section which varied from rock to poor quality soils. Conservative profiles were then based on the material classification and density. As construction progressed, the trench side slopes were modified on site based on the observed in situ conditions of the materials. Only two short sections of local trench instability were encountered during construction.

3. Pipe bedding and backfill material

The grading requirements for the bedding material, as specified in the Contract Specification required the material to be granular with a maximum size of 50 mm and with 90% smaller than 25 mm, and limited the percentage of material finer than ASTM no 200 sieve (0.074 mm) to less than 12 per cent. These materials were to be compacted to minimum relative density of 70%. The availability of suitable material within reasonable distance for transport had a significant influence on the construction programme. Where possible, excavation borrow pits were used; elsewhere additional sources had to be located.

An initial indication of areas of suitable bedding material was obtained during the investigation after the drilling of boreholes. Non-rock areas were investigated by test pits to obtain further samples for particle size analysis, index tests and compaction tests. The results of these investigations were used to ascertain where the materials from the trench excavation were suitable for pipe bedding and were reported in a Borrow Area Investigation report.

4. Design of foundations for structures

At the locations of structures the investigations for geotechnical design parameters were based on
routine in situ and laboratory tests and the geological mapping which provided valuable information on the geomorphology and structural geology of the area. In some cases, further information on the in situ condition of the founding strata was obtained after the excavation for the structure had been completed and prior to construction of the foundation. The results of the surveys were fed back to the design team in UK.

5. Corrosion

The assessment of the aggressivity of the ground and groundwater was one of the most important elements of the ground investigations, as this affected the pipe manufacturing programme which in turn played an important role in the construction programme.

According to the Contract Specification, in areas of chemically aggressive environment the PCCP was to be covered by an additional protective coating for corrosion protection. These pipes were commonly referred to as "black" pipes, whilst the pipes with no additional protective coatings were referred to as "white" pipes. The additional protective coating was applied to the pipe during manufacturing. Hence, well in advance of the commencement of the construction for a particular section an assessment of the chemistry of the ground and groundwater was carried out. This assessment was incorporated into the scope of the ground investigations. The corrosion criteria for the PCCP were specified in the Corrosion Control Strategy report, which identified pipe corrosivity to be dependent on the following factors:

- Presence of groundwater, capillary rise, or casual water within 2 m of pipe invert
- Resistivity of ground to a depth of 2 m below pipe invert
- Ground and groundwater chemistry (total and soluble sulphate and pH)
- Stray current interaction

The above factors were incorporated within the overall investigation programme by introducing sampling and testing strategies to obtain the required parameters. A preliminary assessment of the ground aggressivity was made on site and further investigation carried out, where necessary to establish more accurately the boundary between the black and white pipes.

6. Erosion

The protection of the pipeline and its appurtenant structures against erosion was an important element of the design of the system. The need for erosion protection was particularly critical in low lying areas, wash crossings, and where pipeline was aligned along the wash channels, utility crossings and in irrigation areas. The erosion protection measures varied according to the topography, size of the catchment area, geological and hydrogeological conditions and types and properties of the near surface materials surrounding the pipe.

The topographical maps of the area were used to estimate the size of the catchment area, and the gradients causing flow. Based on this data and the assumption of 1 in 1000 years storm, analyses were carried out to estimate flow velocities. The information from the geotechnical investigation was used to classify the in situ ground conditions according to their erodibility potential. Based on this, erosion protection measures were proposed which comprised one or a combination of the following:

- Additional cover to the pipe
- Gabions
- Rip-rap/mattresses

A survey of the route was carried out prior to and during construction to verify the parameters used in the assessment of the erosion protection and, where necessary, changes to the initial proposals were considered.

7. Aggregate Source

The selection of the locations of the aggregate source was important as vast amounts of good quality aggregate had to be located for concrete and other materials for the project. The investigation of the aggregate source were mainly by test pits to recover large bulk samples for laboratory testing. In addition to routine tests, tests relevant to the aggregate types found in Libya (alkali-silica and alkali-carbonate reaction tests) were undertaken.

3.3 Land reconnaissance survey

Initial route alignment was selected using normal terrestrial mapping at 1:50,000 and aerial photography at approximate scales of 1:10,000. At this scale the main linear lengths of the pipeline were readily located, particularly where major wide wash had been identified as a suitable alignment for route placement.

For the Central Branch, reconnaissance of the selected route was undertaken by the GIBB site team. For the Eastern Branch, due to the tight programme for the construction and delivery of water to Tripoli, the establishment of the centreline of the route of the pipeline and the reconnaissance survey were carried out by a joint task-force comprising GIBB, DAC, GMRA and BRNA.

An initial route was selected following a desk study of available literature. This was very limited in its extent as the study area was poorly documented. The topographical plans of the area were limited to 1:250,000 scale over about a third of the proposed route.

This route was set out by the DAC survey team. A drive-over Survey was then carried out by the task force to establish its suitability on the ground. During the reconnaissance the following factors were considered in assessing the initial route:
Constructability
- Minimising length of pipeline
- Hydraulic considerations
- Geotechnical considerations
- Erosion potential
- Avoiding properties and townships

Any changes to the initial route due to any of the above factors were agreed on by the task force. When the final route was agreed by all parties the GIBB site team produced a reconnaissance report for the chosen route. This report was reviewed and agreed by all the representatives on the task force. The route was then finally located by the DAC survey team.

3.4 Detailed Investigation Programme

Following the reconnaissance for a particular section, a second drive-over of the agreed route was carried out by the task force with the aim of selecting appropriate locations for the geotechnical investigations. This included the location of boreholes, test pits, electrical resistivity survey and seismic refraction surveys.

The selected locations were then incorporated in the Detailed Investigation Programme (DIP). This was a combination of the detailed scope of work, a brief methodology statement and a detailed time schedule for a particular area of the site. A statement on the purpose and scope of work, including the range and approximate quantities of in situ and laboratory tests, and particular requirements for the sequence of working to ensure timely completion of reports was included.

3.5 Geological Mapping

Geological mapping was carried out by Temel in progressive stages for each study area. The mapping for the conveyance line was presented at a scale of 1:10,000. Where topographical maps at a scale of 1:25,000 were available, then this scale was used. The geological mapping for the Central Branch was performed for a corridor of 2 km (one kilometre either side of the pipe centreline). For the Eastern Branch where the route has been selected and agreed on site by the task force a corridor of 500 m each side of the centreline was mapped.

3.6 Geophysical Survey

Electrical resistivity depth soundings were carried out at approximately 1.5 km centres at or near a borehole or a test pit location. If a significant topographical/geomorphological boundary occurred between the locations, further tests were carried out to define this boundary. These tests were mainly carried out to determine the aggressiveness of the ground for corrosion assessment and the design of cathodic protection. Electrical resistivity traverses, in conjunction with depth soundings, were successfully used to locate the extent of buried channels at some locations along the route of the pipeline.

Seismic refraction surveys were carried out at intervals of approximately 3 km. This information was mainly used to assess the excavatability of material for trench excavation.

3.7 Boreholes

Boreholes were rotary cored with water or air flush in rock and rock-like materials. Hollow stem continuous flight auger boring, in conjunction with air flush rotary coring, were used in other materials. Boreholes were generally at 1.5 km centres to a minimum depth of 10 metres (approximately 4 metres below pipe invert) and were used to define the ground conditions along the route of the pipeline. This was revised as necessary for local variation in the ground conditions during the progress of the investigation.

Over 1150 boreholes were constructed and approximately 14.5 kilometres drilled during the works.

3.8 Test Pits

Test pit investigations were carried out after the completion of boreholes. This allowed the location of the test pits to be selected based on the findings of the borehole investigation. Test pits were mechanically excavated to a depth of approximately 7 m below the ground level (approximate level of pipe invert) in superficial deposits. Difficulty in the excavation of test pits occurred when thick layers of cap rock or strongly cemented carbonate soils were encountered.

The purpose of the test pit investigation was for inspection and recovery of large bulk samples for physical and chemical testing, or for obtaining undisturbed block samples for oedometer and shear box tests. At the location of structures, where necessary, test pits were used for execution of in situ density and in situ plate loading tests. Test pits also provided valuable information on the excavatability of the material and on trench side slope stability.

Comprehensive test pit investigations were also undertaken at two locations along the route of the conveyance line for aggregate source investigations. The tests carried out in both the boreholes and test pits on the project were to a certain extent restricted by the availability of the equipment in country, and the feasibility of performing the tests in remote desert areas due to difficulty in
transporting the equipment and dealing with breakdowns and spare parts.

3.9 Laboratory testing

The majority of the routine laboratory testing were carried out at Tenel's physical and chemical laboratory in Libya. Tests which could not be performed in this laboratory were carried out at an other laboratory in the country or in the UK or Turkey. The laboratory tests were mainly performed in accordance with ASTM procedures.

3.10 Reporting

The results of the investigations were reported in the factual reports produced in Libya by Tenel immediately after the completion of the investigation for each section of the conveyance line. These reports were reviewed by GHBB before formal submission to the Owner for approval in accordance with the Project Quality Procedures.

Following the issue of the factual report for each section, a Geotechnical Interpretative Report (GIR) was produced by the GHBB team in the UK. A geological model of the area of the particular section under consideration was produced, using borehole correlations, geological mapping and geophysical survey results. This model was then used to classify the materials into groups for which engineering properties were determined for use in the design.

Geotechnical Design Reports (GDR) were produced for each section of pipeline and for each individual Major Structure (e.g. Flow Control Stations, reservoirs, etc.). The aim of this report was to provide simple conclusions and specific design recommendations and to provide summary information on predicted subgrade conditions. This allowed trench profiles to be set out and excavated and the necessary subgrade improvements to be planned, e.g. areas in swelling clays and areas needing dewatering. The GDR also provided information on the extent of the corrosion protection required along the length of the pipe.

4. GEOTECHNICAL DESIGN

With the geotechnical investigations necessarily being undertaken at an early stage in each section of the route, as detailed design and construction proceeded, some additional information became necessary. This was usually because of minor changes in alignment or location of structures for example, or where a specific parameter or measurement was required at a particular location. The fact that this occurred at only several locations indicates that the general investigation philosophy was sufficient for the majority of the needs of the project.

5. CONCLUSIONS

In conclusion, the geotechnical investigations for the Great Man-made River Project were amongst the most extensive ever undertaken for a civil engineering project. The philosophy adopted and the experiences gained in planning, performance and interpretation of these investigations will be of use to those clients, designers and contractors undertaking similar scale investigation works in the future.

REFERENCES AND BIBLIOGRAPHY


ACKNOWLEDGEMENT

The authors wish to acknowledge the significant contribution and co-operation of Dong Ah Consortium in the performance and management of the geotechnical investigation for the GMP II and thank the Great Man-made River Authority and Dong Ah Consortium for permission to publish this paper. The significant contributions of the many GHBB staff engaged in the management and supervision of the geotechnical investigations over the period of this work is also gratefully acknowledged.
Great man-made river project – Phase II: Heave areas and bedding material investigations

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ABSTRACT: The Phase II of the Great Man-made River Project (GMRP) conveys water from the Jabal Hasanna Wellfields near Fezzan to Tripoli and other coastal regions in the western Janahiba via a buried 4 m internal diameter prestressed concrete cylinder pipeline (PCCP). The ground investigations for the Phase II commenced in 1989 and were completed in 1995. This paper describes two aspects of this investigation which involve the methods of prediction of the subgrade heave potential based on the index properties of the clay and a small site trial carried out to investigate the settlement characteristics of the pipeline bedding materials which contain increased percentage of materials passing ASTM #200 sieve.

1 INTRODUCTION

The Great Man-made River Project in Libya exploits the fresh water aquifers which lie trapped many hundreds of meters below the Libyan Sahara Desert. Phase II of this project conveys water from the Jabal Hasanna wellfields near Fezzan to Tripoli and other coastal regions in the western Janahiba via a buried 4 m internal diameter prestressed concrete cylinder pipeline (PCCP). The Phase II of the project consists of two main sections referred to as the Central and Eastern Branch. The Central Branch runs from the wellfields to Tarhuna about 100 km south of Tripoli. The length of this section is approximately 550 km. The Eastern Branch separates from the Central Branch at approximate mileage 325 km and provides a long diversion around the Jabal Nefrah mountains before it ends near Bin Ghashir about 20 km south of Tripoli. The length of the Eastern Branch is approximately 400 km. From Tarhuna and Bin Ghashir the water is then distributed by secondary conveyance pipelines to other regions.

The ground investigations for the two main conveyance pipelines of Phase II commenced in 1989 and were completed in 1995. This paper is presented in two parts. Part A describes the methods of prediction of the subgrade heave potential based on the index properties of the clay and draws comparison with the estimated heave using laboratory oedometer tests. Part B deals with a small site trial carried out to investigate the settlement characteristics of the pipeline bedding materials which contain increased percentage of materials passing ASTM #200 sieve.

PART A HEAVE POTENTIAL

2 GENERAL

The heave phenomenon has been discussed by many researchers such as Ladd (1959), Seed et al (1962), Parcher et al (1965), Kair (1972) and Agarwal et al (1977). The swelling of expansive soils such as clay is the result of the interplay between physical and chemical properties of clay particles and those of the ground water. The factors that affect the amount of swelling are:

a) initial moisture content
b) confining overburden pressure
c) orientation of clay particles
d) clay mineralogy
e) density of the soil

Therefore, since the amount of heave is dependent both on chemical and physical properties of clay, the prediction of volume change of expansive clays is a difficult task and is usually based on correlations with index properties of the soil.

The heave of expansive clays may also be estimated from laboratory oedometer tests in which a sample of clay is inundated with water under small confining overburden pressure and is allowed to freely swell or prevented from swelling by increasing the overburden pressure (see ASTM 4546).
3. ESTIMATION OF POTENTIAL HEAVE ON GMRP II

During construction, the Subgrade Verification Procedures (SVP) of the project required that the material within a zone of 4 m below the pipe invert be investigated at close spacing along the length of the pipeline prior to placing the bedding material and installation of the FCP. This procedure was used to confirm that the foundation soils have an adequate bearing capacity and also to ensure that the subgrade vertical movements due to settlement or heave of the foundation soils are within the limits specified in SVP. The vertical movements are particularly critical at the interface between the clay and non-expansive materials along the pipeline where differential heave and settlement produce a gradient which may exceed the thresholds specified in the SVP.

The depth of the expansive clay over the length of the interface is accurately determined by frequent investigation points and disturbed samples of the clays are recovered. After performing laboratory testing to determine the index properties of the clay, the potential heave is then calculated based on simple correlations using index properties of the clay. This method of prediction provides a quick means of assessing the heave potential which is well suited for use during construction when compared with the odometer testing. Correlations for prediction of heave using index properties are described below.

The estimation of potential heave on GMRP II was based on the assumption that infiltration of water will take place to initiate and sustain the heave process and that full saturation of the swelling clay layer will occur. The magnitude of heave was therefore considered to be dependent on the following parameters:

a) swelling pressure
b) swelling index
c) initial voids ratio

Once these parameters have been defined then the amount of total potential heave will depend on the presence of water to initiate the swelling process, the permeability and thickness of the clay layer within the zone of influence and the confinement due to the overburden.

3.1 Swelling Pressure

Based on data from tests on more than 200 clay samples, Komornik and David (1969) proposed an equation to estimate the swelling pressure of clay soils using their index properties.

Komornik and David’s equation, when translated into SI units, is:

\[
\log P = 0.0208(LL - 0.269m + 0.0665\sigma_d + 0.132
\]

where:

\[
LL = \text{Liquid Limit (\%)}
\]

\[
\sigma_d = \text{Dry Unit weight (kN/m}^2)\]

\[
m = \text{Natural moisture Content (\%)}
\]

The swelling pressures measured from laboratory oedometer tests carried out in the earlier stage of the Phase II site investigations, were compared with swelling pressures predicted by Komornik and David’s equation. In order to obtain a closer agreement between the predicted and measured swelling pressures and using an average dry unit weight of 17 kN/m² from laboratory test results, equation (1) was modified to the following:

\[
\log P = 0.0208(LL - 0.03m + 1.263)
\]

(for LL \leq 60)

\[
\log P = 2.29 [(LL - 30)/(42 - w)]^{0.45}
\]

(for LL > 60)

3.2 Swelling Index

From laboratory data a relationship between swelling index and liquid limit was established as given by the following equation:

\[
C_s = 0.0023LL - 0.0372
\]

3.3 Void ratio

An average initial void ratio of 0.67 was obtained from laboratory test results.

3.4 Estimation of Heave

The equation proposed for the estimation of heave was based on the one dimensional consolidation equation:

\[
S = C H(1 + e_0)(\log \sigma /\sigma_0)
\]

If \(e_a\) is the swelling pressure and \(e_0\) is the available overburden pressure resisting the swelling pressure, then the above equation can be expressed as follows:

\[
H = \frac{2C H(1 + e_0) \log P - \log \sigma_0 + D_0 - D_{0.1}}{2}
\]

where:

\[
H = \text{Heave}
\]

\[
C = \text{swelling index}
\]

\[
e_0 = \text{Initial void ratio}
\]

\[
H = \text{Layer thickness (ft)}
\]

\[
P = \text{swelling pressure of the clay layer}
\]

\[
\sigma_0 = \text{Average bulk unit weight of the pipe}
\]
\[ D_n = \text{Depth to the top of the } n\text{th clay layer from original ground level.} \]

\[ D_{nt} = \text{Depth to the bottom of the } n\text{th clay layer from original ground level.} \]

The average bulk unit weight for the pipe fall of water, backfill and clay was calculated to be 18.2 kN/m³. Hence, assuming an average initial void ratio of 0.67, based on the results of the laboratory tests, and a bulk unit weight of 18 kN/m³ for the overburden, equation (6) can be re-written as follows:

\[ H_p = \sum C_i H_i / 1.67 \log P - \log [(D_n + D_{nt})] \]  

(7)

4 ASSSESSMENT OF HEAVE POTENTIAL

Equations (2) to (4) and (7) were used in the calculation of heave of the clay layers within the 4 m zone of the influence during the subgrade verification procedures for Phase II. These estimates were based on the moisture content and liquid limit of the samples of clays recovered during the SVP.

At locations where the swelling pressure and swelling index values had been measured in the laboratory during the ground investigations, the heave potential has been estimated directly from equation (7) for comparison purposes. The results of this comparison are presented in section 5.

5 DISCUSSION OF RESULTS

A plot of the ratio of the predicted heave to the measured heave against liquid limit is presented in Figure 1. It can be seen that for liquid limits greater than 60%, the calculated heave is generally greater than the values based on the laboratory oedometer tests. For liquid limits below 60%, the method of prediction slightly underestimates the expected heave as calculated from the oedometer test results. However, the measured heave for these clays was generally small and are unlikely to cause any foundation problem. The assumption of full saturation of clay within a zone of 4 m below the pipe invert for swelling to take place is very conservative and may never take place as the source of water along the route of the pipeline is generally scarce and the water for swelling is assumed to be from utility crossings, irrigation systems and flowing wadis during storms.

PART B BEDDING SITE TRIALS

6 GENERAL

The grading requirements for the bedding material, as specified in the Contract Specification required the material to be granular with a maximum size of 50 mm and with 90% smaller than 25 mm and limited the percentage of material finer than ASTM #200 sieve (0.074 mm) to less than 12 percent. This material was to be compacted to a minimum relative density of 70%. The availability of suitable material within reasonable distance for transport had a significant influence on the construction programme.

For some sections along the Eastern Branch Conveyance line the majority of the borrow areas investigated contained materials with percentage finer than ASTM # 200 sieve of between 12% and 25%. Therefore, site trials were carried out in order to demonstrate that the settlement characteristics of the bedding materials containing percentage finer than ASTM # 200 sieve between 12% and 25% would be satisfactory and that these materials could be used for the PCCP bedding. The bedding site trials comprised placement and compaction of the bedding materials and the performance of in situ density and plate load tests and laboratory tests.

7 SITE TRIALS

7.1 Bedding material properties

Two types of bedding materials with percentage finer than ASTM #200 sieve of 16% and 24% were used in the trials.
The results of the laboratory tests carried out on the above materials to determine their particle size distribution and index properties are shown in Table 1. All laboratory tests were carried out in accordance with the relevant ASTM Standards.

The particle size analysis distribution for both materials are given in Table 1. The grading of the materials indicates that both materials are classified as silty SAND (SM). However, it is noted that the material with 16% fines contains 77% fine grained sand whilst the material with 24% fines is relatively a more well graded material. The Atterberg limit tests carried out on these materials indicated both materials to be non-plastic.

The compaction tests carried out on the materials indicated maximum dry densities of 1.81 g/cm³ for the material with 16% fines and 1.918 g/cm³ for the material with 24% fines. The respective optimum moisture contents were 11.5% and 10.25%. Maximum and minimum density tests were carried out to determine the relative density of the compacted material when the field density is known. These indicated values of 1.629 and 1.373 g/cm³, respectively for the material with 16% fines and 1.782 and 1.479 g/cm³, respectively for the material with 24% fines.

7.3 Test areas and preparations

Three test areas were excavated to a depth of 1 m on the south side of the haul road at chaining E560,000. The base of the excavated trench was cleaned and all loose soilrock and debris were removed prior to placing the fill. The material for the ICB was moisture conditioned and placed in a 500 mm thick layer. This was subsequently compacted by a 10 tonne vibrating roller using 6, 8 or 10 number of passes in different test areas to study the effect of compaction effort on density. The material for the HZB was placed at its natural moisture content and compacted by a Hoeapac with 8 seconds vibration time in one area and 12 seconds vibration time in another. A layer thickness of 800 mm was used.

After placing and compacting the material samples were taken for moisture content determinations on site using the speedy moisture apparatus.

7.4 In situ density tests

A total of 22 in situ density tests were carried out. All tests were carried out by the sand replacement method in accordance with the procedures specified in ASTM D1556. At least one in situ density test was carried out near each plate load test to determine the initial in situ density of the compacted material prior to the execution of the test and to calculate the relative density using the maximum and minimum densities calculated in the laboratory for each material. The initial density of the compacted bedding materials were also intended to investigate any relationship between the plate settlement and density of the compacted material.

Further in situ density tests were carried out in the materials compacted with different compaction effort to study the compactability of the bedding materials with increased amount of fines.

7.5 Plate load tests

A total of six plate load tests were carried out. Four tests were carried out in the ICB materials with two tests performed in the material with 16% fines and two
in the material with 24% fines. Two plate load tests were carried out in the HZB with one performed in each material with 16% and 24% fines.

The test procedures consisted of loading a 300 mm plate at increments of 40 kPa up to a required load (P). For each load increment the plate settlement was monitored until the difference between consecutive readings was less than 0.03 mm. At load (P) the plate was flooded with water to approximately 50 mm above the base of the plate to simulate the conditions where the base of the trench becomes saturated due to infiltration of water from various sources. Keeping the load (P) and the water level above the plate constant, the plate settlement was monitored for a period of at least one hour until the time-settlement curve commenced to tail off or the difference between the consecutive settlement readings taken after one hour was less than 0.03 mm. The load on the plate was then increased to a final value which varied from test to test and the plate settlement monitored.

8 DISCUSSION OF RESULTS
8.1 In situ density test results

The results of the in situ density tests are shown in Figures 2 to 4. From these results the following observations are made:

1. The results of the tests on material with 16% fines confirm that increasing the moisture content towards the optimum moisture content (OMC) and the compaction effort increases the density of the compacted material. The maximum density of 101.2% standard Proctor was obtained for the maximum compaction effort of 10 passes with a moisture content of 10% which is close to the OMC of 11.5% for this material. The minimum density of 91% Standard Proctor was obtained for the minimum compaction effort of 6 passes and moisture content of 7.8%. For the same compaction effort of 6 passes, increasing the moisture content to 10.2% produced a density of 96.9% Standard Proctor.

2. In the material with the 24% fines, generally higher densities were achieved for the same compaction effort when compared to the densities obtained in the material with 16% fines, as shown in Figure 2. This is in agreement with observations of Finley and Mesch (1992). However, it is noted that this material with 24% fines was compacted at higher moisture contents very close to its OMC. This may have resulted in achieving higher densities.

3. For the HZB, generally lower compactions were achieved due to their lower moisture contents (Figure 3). Increasing the compaction effort from 8 to 12 seconds produced only slight improvement in the density.

4. From Figure 4 it can be seen that for 70% minimum relative density, specified in the Specification for the bedding material containing less than 12% fines, an equivalent percentage Standard Proctor density of about 85% is obtained. From this figure it is suggested that for the bedding material containing more than 12% fines the use of Standard Proctor density is more appropriate than relative density for specifying the desired degree of compaction.
8.2 Plate load test results

For the ICB a total of four plate load tests were carried out. Two of the tests were carried out in the material with 16% fines and two on the material with 24% fines. In all these tests the load on the plate was increased in increments of 40 kPa to 120 kPa. This load was calculated to be the load applied on the bedding by the pipe and the backfill in the trench. At this load the plate was flooded with water and the plate settlement monitored. After the plate settlement was complete the load on the plate was increased to 160 kPa and the plate settlement monitored.

For the HZB a total of two plate load tests were carried out on this type of bedding. One of the tests was carried out in the material with 16% fines and the other on the material with 24% fines. In both tests the load on the plate was increased in increments of 40 kPa to 160 kPa. This load was calculated to be the load applied on the bedding by the pipe and the backfill in the trench. At this load the plate was flooded with water and the plate settlement monitored. After the plate settlement was complete the load on the plate was increased to 120 kPa and the plate settlement monitored.

The results of the plate load tests are presented in Figures 5 and 6. From these figures the following observations are made:

1. For ICB, the settlement values before and after saturation were very small and no trend of increasing settlement with percentage fines content is indicated.
2. The total settlement prior to saturation for HZB was 2.96 mm for the material with 16% fines and 1.80 mm for the material with 24% fines. This larger settlement may be due to the slightly lower initial density of this material.
3. For HZB a Total settlement after saturation of 25.56 mm and 16.47 mm was obtained for the material with 16% and 24% fines, respectively. The respective settlements for these materials due to saturation were 22.6 mm and 14.67 mm. The large settlement on saturation is due to the lower initial moisture content of the HZB material.
CONCLUSIONS

The results presented in Parts A and B are drawn:

1. For liquid limits greater than 60%, the calculated value is generally greater than the values based on the laboratory oedometer tests.

2. For the initial compaction, the results indicate that the bedding material with increased percentage of fines of 16% and 24% can be placed and compacted to achieve densities of well in excess of the minimum value specified in the Contract Specifications.

3. The relationship between the percentage Standard Proctor density and the relative density for all the tests carried out showed that for the bedding material containing more than 14% fines the use of Standard Proctor density is more appropriate than relative density for specifying the desired degree of compaction.

4. The observed settlements for both the ICB and HZB are very small and well within the equivalent specified threshold values.

5. The settlement due to saturation is insignificant for materials compacted at moisture contents close to their OMC.

ACKNOWLEDGEMENT

The authors wish to acknowledge the significant contribution and co-operation of Doug Ah Consortium in the performance and management of the geotechnical investigation of the GMRP II and thank the Great Man-made River Authority and Doug Ah Consortium for permission to publish this paper.

REFERENCES


ASTM Standards 1983. Volume 04.08. Annual Book:


Geotechnical Site Characterization, Robertson & Mayne (eds) © 1996 Balkema, Rotterdam, ISBN 90 5410 939 4

The site investigation program for the Oslo Airport, Gardermoen, Norway:
I. Site investigation strategy

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ABSTRACT: A new main airport is presently being built at Gardermoen, a small community about 50 km north of Oslo. The airport will, upon completion in 1998, cover an area of about 13 km², and is built on a complex system of several glaciifluvial deltas, forming an almost horizontal plateau. Due to the relatively complex geological history, the soil conditions in the area show large variations, containing soil types ranging from soft sensitive clays to very dense, morainic materials. This variation in soil conditions, and the geotechnical and hydrogeological problems associated with development of the area, required a thorough knowledge of the soil conditions. A variety of tests, involving more than 700 locations, were carried out. The field test program included various types of soundings, piezocene tests (CPTU), field vane tests, screw plate tests, undisturbed piston sampling, isotopic soundings, hydrogeological soundings and various geophysical investigations. A comprehensive laboratory test program was also carried out.

This paper presents some aspects and results from the planning and performance of the site investigation program, and focus on a presentation of the investigation strategy and evaluation of the investigation methods used.

1 INTRODUCTION

A new main airport is presently being built at Gardermoen, a small community about 50 km north of Oslo. The new airport will be completed in 1998, and will become the new main gateway to Norway as well as the most important airport for domestic flights.

The airport will upon completion cover an area of about 13 km².

The airport is built on a complex system of several glaciifluvial deltas, forming an almost horizontal plateau. Due to the relatively complex geological history, the soil conditions in the area show large variations, containing soil types ranging from soft sensitive clays to very dense, morainic materials. The new airport is located in the area of Norway’s largest precipitation fed groundwater reservoir. Modelling of the groundwater table and the groundwater flow to protect the reservoir against contaminating agents from the airport, was one of the major subjects for planning and design.

The variation in soil conditions, and the identified geotechnical and hydrogeological problems associated with development of the area, required a thorough knowledge of the soil conditions. A variety of tests, involving more than 700 locations, were thus carried out.

The test program included various types of soundings, piezocene tests (CPTU), field vane tests, screw plate tests, undisturbed piston sampling, isotopic soundings, hydrogeological soundings and various geophysical investigations. A comprehensive laboratory test program was also carried out, including index testing, triaxial, endoscope and permeameter tests.

The total soil investigation program covered an area of more than 50 km², and was carried out in two major phases. The main introductory investigations were carried out in 1991 to establish a geotechnical background for the choice between various alternative locations. The detailed site investigation program was performed in 1992 - 93, after the principal location was settled and design of the airport could be carried out.

The total costs of the site investigation program exceeded 15 MNOK (about 3 mill US$). The major part of the site investigations were carried out by the Norwegian Geotechnical Institute (NGI) in Oslo.

This paper presents some aspects of planning and
performance of the site investigation program, including a discussion of the investigation strategy, and evaluation of the investigation methods used.

2 PROJECT DESCRIPTION

The new Oslo Main Airport is now being built at Gardermoen, a small community located about 50 km north of Oslo in a countryside dominated by acres and green fields. The new airport will upon completion in 1998 become the new main gateway to Norway, and also the most important airport for domestic flights. The developer of the project is Oslo Main Airport Association (Oslo Havneflyplass A/S OSL) and the Norwegian Aviation Administration. These organizations have permitted the use of data presented herein.

SINTEF Civil and Environmental Engineering, Department of Geotechnical Engineering, in cooperation with The Department of Geotechnical Engineering at the Norwegian University of Science and Technology (NTNU) in Trondheim, acted as geotechnical advisors to OSL in the project. The majority of the site investigations, including the CPTUs, were carried out by the Norwegian Geotechnical Institute (NGI) in Oslo. Results presented in this paper refer to evaluations and interpretations carried out by NTNU/SINTEF, not necessarily being a part of the design recommendations.

Figure 1 shows a principal overview of the new airport, which includes two parallel runways, a traffic terminal with railway station and the apron area with its necessary infrastructure. The new airport will cover an area of about 13 km².

The total site investigation program covered an area of more than 50 km², and was carried out in two phases. The main investigation was carried out in 1991 to establish a geotechnical background for the choice between various alternative locations. The detailed site investigation program was performed in 1992 - 1993 after the principal location was settled, and design of the airport could be carried out.

2.1 Brief description of the geology in the Gardermoen area

The new airport will be located at an almost horizontal plateau at about 205 mnd. This plateau was formed as a complex system of several glacial drifts during a period of about 50 - 100 years in the Preboreal period, some 10,000 years before present (Watts et al (1995)). The deposition of this system took place in contact with the glacial front in a shallow marine fjord, to the north of the planned airport. The deltae have later aggraded, prograded and coalesced into a large complex, including a relatively thick outwash plain.

In the northern parts of the area, the Garderfjell hill rises up to about 225 mnd, whereas in the western and southern parts, the plateau is bordered by ravined terrain. In this area, which partly correspond to the alternative western location of the airport, the soil conditions are dominated by partly sensitive clays and silts. To the east, the terrain drops gradually to a landscape with several kettle holes and other dead ice formations. Sands and gravels generally dominate the geology, with finer materials encountered in lenses in a matrix of sands and gravels. Clay strata may be found at larger depths.

3 GEOTECHNICAL PROBLEMS RELATED TO THE PROJECT

Several geotechnical problems had to be solved in order
to arrive at a safe design of the airport. Even if the overall quality of the soil masses was good with respect to bearing capacity and compressibility, the large variations in soil conditions were one of the challenges met. An important task in this project was therefore to carry out a thorough mapping of the soil conditions, and the subsequent construction of a representative stratigraphical model for the area (Wain et al. 1995).

In the following, the design of different installations and structures, along with the corresponding need of geotechnical information, is briefly discussed.

3.1 Foundation of runways

This work included improved foundation of an existing runway and design of a new. Very strict limits for acceptable total and differential settlements were set for the runways, and settlement calculations required reliable values of deformation modules and preconsolidation stress levels. In the upper sand and silt layers, these parameters were mainly found from CPTU interpretations. From the expected content of fines, the soil classification also indicated the frost susceptibility of the masses.

3.2 Foundation of buildings and structures

The engineering works included foundation of a 90 m high flight control tower, hangars, office buildings and similar structures. Design of these structures and buildings represented conventional geotechnical analyses, including evaluations of bearing capacity and settlement of the structures. Analysis of shallow foundations in the upper sand and silt layers relied very much on CPTU interpretations.

3.3 Construction of the railway culvert

The construction works for the 165 long and 112 m wide terminal building included a 12 m deep excavation for a railway culvert. To control and maintain the groundwater balance, it was chosen to install an impervious sheet pile wall in this area. In addition, a well-point system was introduced for dewatering of the culvert pit.

From the CPTU results it was obvious that the vertical coefficient of permeability was considerably smaller than the horizontal, mainly due to the presence of horizontally oriented silt and clay lenses in the top sand matrix. These lenses would reduce the vertical flow of water considerably, and the use of a sheet pile wall was hence recommended. The use of this stabilization method made it easier to maintain the groundwater balance in the area, a solution made feasible by the very detailed mapping of the soil conditions obtained from the CPTU results.

The design of the railway culvert also required reliable information about the soil conditions in both the sand and clay layers, such as strength properties for design of sheet pile walls and stability evaluations, deformation parameters for settlement calculations and permeability properties for evaluation of the drainage and groundwater control systems. Use of CPTU data was an important part of all these design considerations.

The majority of the CPTU results presented and discussed herein refer to the area of the railway culvert and the traffic terminal.

3.4 Protection of groundwater resources

The airport is constructed on a 79 km² coarse-grained, precipitation-fed groundwater aquifer (Wain et al. 1995). To simulate the groundwater balance in the area of the new airport, a sophisticated model for groundwater flow was developed, based upon the stratigraphical model for the area. The aim of this model was to evaluate the risk of preliminary or permanent movements of the groundwater table, caused by the building activities. The model could also be used to evaluate the risk of contamination of the groundwater aquifer, for example from leaking fuel tanks or leaching liquids percolating from the surface.

As indicated, the design of the airport and its installations required knowledge of the whole specter of geotechnical and hydrogeological parameters.

4 SITE INVESTIGATION PROGRAM

The site investigation program is the largest of its kind in the whole of Norway, stretching over a period of almost 2 years, with more than 7 effective months of field operations. The total costs of the site investigations exceeded NOK 15 mill (~3 mill USD). The site investigation program included the following tests:
- 697 rotary pressure soundings
- 81 total soundings/bored control
- 27 simple/weight soundings
- 206 CPTUs
- 16 field vane tests
- 5 screw plate tests
- 144 locations with undisturbed sampling.
Figure 2 Identification chart proposed by Robertson et al (1990), including CPTU classifications from the railway culvert area.

Figure 3 Example of output from the stratigraphical model, including identification of soil strata.
- 149 excavation pits
- 6 isotopic soundings
- 581 hydrogeological investigations (piezometers, observation and pumping wells)

In addition, a comprehensive geophysical investigation program, including reflection seismic and georadar measurements was carried out.

In the laboratory, various index and special tests, such as oedometer, triaxial and permeability tests were carried out on undisturbed samples of clays and fine silts. Several tests were also carried out on disturbed samples of coarser materials with relevant porosities.

4.1 Site investigation strategy

A main investigation was carried out first to establish a geotechnical background for the choice between different principal alternatives. Both a western and an eastern location related to the existing runway were discussed, each principal location with several alternative designs. After choosing the eastern location, the detailed soil investigation program was carried out.

The general scheme beyond the site investigation program was to carry out a large number of simple and quick rotary pressure soundings to obtain a rough impression of the ground conditions. At selected locations, the rotary pressure soundings were coupled with CPTU and laboratory classifications to obtain an overall verification of the soil types and stratigraphy. This information created the basis for the stratigraphical model representing the ground conditions in the area.

It was also important to couple CPTU profiles, soil sampling and other in situ methods at some locations to obtain a calibration and control of the CPTU interpretations in various soil types. The CPTU results also gave a very good basis for choosing levels for undisturbed soil sampling, as well as locations for other in situ tests.

4.2 Stratigraphical modelling

The identification of soil types and soil strata was mainly based on results from CPTUs and rotary pressure soundings. The overall interpretations of these two tests were verified by results from various sampling and classification methods. In the location of the airport, the soil conditions may roughly be characterized by an upper sand layer, overlaying strata of silt and clay. The bedrock is usually covered by morainic material, locally with considerable thickness. This rough description is made somewhat more sophisticated in the stratigraphical model presented later.

Even the presence of thin lenses of contrasting materials, such as clay or silt in a dominating sand matrix or vice versa, was revealed through the CPTU results. This was very important when evaluating the drainage properties of the soils, a major topic in this project.

The chart proposed by Robertson et al. (1986, 1990) was used to evaluate encountered soil types from CPTU results. This chart incorporates values of q, Br, and Rb in the classification of soil type. Figure 2 shows CPTU results from the railway culvert area in this diagram, using the normalized relationships of the CPTU recordings. The diagram includes the possibility of addressing features like overconsolidation, aging and sensitivity into the soil classification.

According to the obtained results, clayey soils show large variations in sensitivity, an indication which is confirmed by other test results. This demonstrates that CPTU classification charts may give valuable indications on engineering properties of the soils, not only on soil type and composition. Moreover, the figure shows the variation in soil composition in the selected area, ranging from dense sands to very soft, sensitive clays.

The aim of the stratigraphical model was to provide information of the detailed stratigraphy and encountered soil types at a given location, based on geotechnical soundings, in situ tests and laboratory classifications (Waag et al. 1995). Hence, at any chosen location in the area, the stratigraphical model provides information of surface geometry, soil stratification, position of the groundwater table and depth to the rock surface.

The model is based on evaluation of the soil stratigraphy in altogether 711 investigation points, whereof 206 are CPTU profiles. As shown in Figure 3, the soil stratification is represented by a total of 7 strata, including the bedrock and the groundwater table. Roughly, one may characterize the ground conditions by an upper system dominated by sands, with lenses of both coarser and finer materials. Below this top stratum, the soils are dominated by clays and silts, overlaying scattered morainic material and the bedrock. The stratigraphical model does not fully reflect the detailed geological conditions at the site. However, the main stratigraphical patterns of the model has been confirmed by on-site inspections in excavated pits and cuts in the area.

Further use of CPTU data in the Gerdnersøen site investigation program is presented by Sandven et al. (1995, 1998).
5 ACKNOWLEDGMENTS

Financial support was provided by the Norwegian Aviation Administration for the sedimentological and hydrogeological field and laboratory investigations. Oslo Hovedflyplass AS (OSL) has kindly permitted the use of data from the soil investigations. We also appreciate the work of M.Sc. Håkon Kulleberg and the valuable contributions from professor emeritus Nilsmar Jardal at the Department of Geotechnical Engineering, NTNU. Thanks are also due to Jan Ove Banken (SINTEF), Eirik Fyltn and Randi Sæther (NTNU) for excellent drawing assistance.

6 LITERATURE AND REFERENCES


Sandven, R., Watz, A. and Westerdal, G.J. (1998). The site investigation program for the Oslo Airport, Gardermoen, Norway. II. Use of in situ test results in design. To be presented in Proc., ISEC’98 First International Conference on Site Characterization, Atlanta, USA.


The site investigation program for the Oslo Airport, Gardermoen, Norway: II. Use of in situ test results in design

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ABSTRACT: A new main airport is presently being built at Gardermoen, a small community about 50 km north of Oslo. The airport will upon completion in 1998 cover an area of about 13 km², and is built on a complex system of several glaciological deltas, forming an almost horizontal plateau. Due to the relatively complex geological history, the soil conditions in the area show large variations, containing soil types ranging from soft sensitive clays to very dense, morainic materials. This variation in soil conditions, and the geotechnical and hydrogeological problems associated with development of the area, required a thorough knowledge of the soil conditions. A variety of tests, involving more than 700 locations, were carried out. The field test program included various types of soundings, piezocene tests (CPTU), field vane tests, screw plate tests, undisturbed piston sampling, isotopic soundings, hydrogeological soundings and various geophysical investigations. A comprehensive laboratory test program was also carried out.

This paper shows how advanced in situ tests, particularly CPTU, may be used to reduce the amount of sampling and laboratory testing in a rational site investigation program. The paper also includes comparisons between in situ and laboratory test data in selected soil types.

1 INTRODUCTION

A new main airport is presently being built at Gardermoen, a small community about 50 km north of Oslo. The new airport will be completed in 1998, and will become the new main gateway to Norway as well as the most important airport for domestic flights. The airport will upon completion cover an area of about 13 km².

The airport is built on a complex system of several glaciological deltas, forming an almost horizontal plateau. Due to this relatively complex geological history, the soil conditions in the area show large variations, containing soil types ranging from soft sensitive clays to very dense, morainic materials. The new airport is located in the area of Norway’s largest precipitation fed groundwater reservoir. Modelling of the groundwater table and the groundwater flow to protect the reservoir against contaminating agents from the airport, was one of the major subjects for planning and design.

The variation in soil conditions, and the identified geotechnical and hydrogeological problems associated with development of the area, required a thorough knowledge of the soil conditions. A variety of tests, involving more than 700 locations, were thus carried out.

The test program included various types of soundings, piezocene tests (CPTU), field vane tests, screw plate tests, undisturbed piston sampling, isotopic soundings, hydrogeological soundings and various geophysical investigations. A comprehensive laboratory test program was also carried out, including index testing, triaxial, oedometer and permeability tests.

The total soil investigation program covered an area of more than 50 km², and was carried out in two major phases. The main introductory investigations were carried out in 1991 to establish a geotechnical background for the choice between various alternative locations. The detailed site investigation program was performed in 1992-93, after the principal location was settled and design of the airport could be carried out. The total costs of the site investigation program exceeded 15 MNOK (about 3 mill. US$. The major part of the site investigations were carried out by the Norwegian Geotechnical Institute (NGI) in Oslo.

This paper aims to show how advanced in situ...
2 THE ROLE OF CPTU IN NORWAY

Cone Penetration Tests (CPT) with measurement of pore pressures during penetrations (CPTU) has been the dominating site investigation method on the Norwegian Continental Shelf for more than 20 years. The method is an obligatory part of any site investigation program offshore, with penetration carried out from the seabed or by down-the-hole testing. This activity involves piezocene testing in a wide range of soil types, from soft clays in the Norwegian Trench to stiff, overconsolidated clays and sands in the shallow bank areas (Lunne & Sandven (1993)).

In this period, much work has been carried out to develop solid and reliable test equipment, a large part of it tailor-made for offshore operations. Along with the development of CPTU equipment, the oil companies have also encouraged research on interpretation and direct use of CPTU results in practical design.

In Norway, CPT has also been used onshore since the beginning of the 1950s, with separate pore pressure measurements introduced as early as 1972 (Aasbu & Sannet (1974)). However, except from a few projects, CPT and CPTU has not gained the same status in onshore practice until recently. A major reason for that may be the complexity of the equipment, which requires careful calibration and maintenance of the equipment, and the need for a time consuming preparation phase in the field before the actual test can be carried out.

However, more rational piezocene equipment has now made the method a more cost-effective alternative to other field investigation methods. The combination of an outstanding soil mapping ability, and the potential of estimating soil parameters from the test results, explain the increased use of CPTU in Norwegian site investigation practice.

This paper describes the role of CPTU in the major site investigation program carried out for the new Oslo Main Airport, Gardermoen. This investigation program represents in many ways a breakthrough of CPTU in Norwegian onshore practice, involving very successful and versatile use of CPTU results for both soil classification purposes and design evaluations. The work presented herein addresses both these features, related to the many geotechnical and hydrogeological challenges encountered in the planning and design of the airport infrastructure.

3 USE OF IN SITU TEST RESULTS IN THE PROJECT

The CPTUs were carried out according to the ISSMFE recommended test procedures for CPT (ISSMFE, 1989). The tests were carried out by the Norwegian Geotechnical Institute (NGI), using hydraulic drill rigs pulling about 1 m in each stroke before a new rod had to be added. An ENVITECH Mencocone cableless penetrometer system was used for all piezocene tests at the site. The utilised system consists of the Mencocone penetrometer itself and a corresponding Geoprinter data acquisition unit.

In the ENVITECH Mencocone, a steel ring just behind the conical part provides a 0.3 mm opening slot into the cone. This ring replaces the conventional porous filter and reduces the ringing of the test, compared to the other piezocene. The cavities of the cone are saturated with silicone grease or similar, whereas the pressure chamber is filled with de-aired water or an antifreeze liquid. The saturation procedure is very efficient, and parallel tests with conventional piezocene have shown good agreement in measured pore pressure values.

The results and interpretations presented herein refer to two selected areas:

- the area of the railway culvert and the traffic terminal at the chosen location, represented by the dotted line in Figure 1. The geology is dominated by sandy soils.

- an area some kilometers west of the chosen location (western location), once considered an alternative location of the new airport. The geology is dominated by clayey and silty soils.

Some examples on the use of CPTU results in the design of airport infrastructure are presented herein. The examples include both aspects of soil identification and stratification, as well as interpretation and correlation of soil parameters. The examples intend to show some of the potential use of CPTU results in such a major project, but some of the shortcomings are also discussed.

4 INTERPRETATION OF SOIL PARAMETERS FROM CPTU

In this section, the basic principles of some selected interpretation methods are outlined. A few
comprehensive presentation of methods and results is
given by Sandven and Wain (1995). The following
parameters are presented here:
- undrained shear strength, \( s_u \)
- preconsolidation stress, \( c'_{p} \)
- deformation moduli in clay, \( M_k \), \( M_s \)
- deformation moduli in sand, \( M_s \)

The soil parameters determined from laboratory tests
were used to calibrate the values interpreted from the
various in situ tests.

4.1 Undrained shear strength

The determination of a unique value of the undrained
shear strength \( s_u \) for a clay is not a single task. This
parameter is influenced by several factors such as
anisotropy, stress history, rate of straining and mode of
failure. The undrained shear strength may be found from
CPTU, either by using a cone resistance based or a pore
pressure based approach. In this context, only the latter
approach will be presented.

By the use of cavity expansion or stress field
theory, the undrained shear strength can also be
expressed by:

\[ s_u = \Delta u \rho N_{c} \]  

(1)

The bearing capacity factor \( N_{c} \) may be written as
\( N_{c} = N_{k} N_{a} \) where \( N_{k} = 6 \sim 9 \) = theoretical bearing capacity
factor. This approach is based on the use of stress field
theory. Using the cavity expansion theory, solutions for
\( N_{c} \) are developed both for pore pressures at the face of
and immediately behind the cone (e.g. Mazzarachi and
Broms (1981)).

According to both these approaches, the bearing
capacity factor \( N_{c} \) may vary between 2 and 20.
However, with typical values of \( B_k = 0.8 \sim 1.0 \) in soft
clays, the range in \( N_{c} \) values becomes 5 \sim 9. In
overconsolidated, stiffer clays, a representative range
becomes 4 \sim 6, developed from a range in \( B_k \) values
from 0.5 to 0.7. Lamme et al (1989) report \( N_{c} = 6 \sim 10 \)
as an estimated range for most Norwegian clays.

This approach has gained increasing popularity in
Norway, particularly in soft clays, where pore pressure
based interpretation is believed to be less influenced by
errors in the measurements.

Reference values of the undrained shear strength were
in this project found from laboratory triaxial and shear
box tests and in situ vane tests. Results from triaxial
tests refer to samples consolidated to the present in situ
effective stress level. The reference values are indicated
as results from local tests, combined with an \( s_u \) -
envelope including all values from the western location.
However, the number of such tests are limited, and
obtained values vary considerably over the site.
Moreover, samples taken from large depths may be
somewhat disturbed due to swelling effects, so that
local correlations may be uncertain.

Figure 1 shows results from pore pressure based
interpretations of a CPTU profile from the western area.
In the example, \( N_{c} \) values between 6 and 8 have been
chosen for the interpretation of CPTU results.

The obtained results indicate that this approach may
slightly overestimate the undrained shear strength, at
least if the selected reference values are representative
for the true material strength. In this context it is fair to
mention that the obtained reference values show a
significant scatter, making it difficult to obtain unique
correlations and draw certain conclusions. There is
however no doubt that the CPTU interpretations
provided valuable input to the assessment of design
values for the undrained shear strength.

4.2 Evaluation of stress history

Knowledge of the preconsolidation stress \( c'_{p} \) is impor-
tant in Norway, since a large part of our soil deposits
are overconsolidated. In situ tests, such as CPTU, may be an alternative for evaluation of stress history, particularly in soils where it is difficult to obtain adequate sample quality. In the following, some simple principles for evaluation of stress history in clays are outlined.

The theoretical bearing capacity expression for a clay in terms of total stresses can be written:

\[ q_t = \gamma z - N_c s_u \]  

(2)

where:
- \( q_t \): theoretical cone resistance
- \( \gamma \): unit weight of soil
- \( z \): penetration depth
- \( N_c \): bearing capacity factor (= 6 - 12)
- \( s_u \): undrained shear strength

For a normally consolidated clay, the undrained shear strength can further be expressed as:

\[ s_u = \alpha_s (\gamma - \gamma_w) z = \alpha_u \gamma z \]  

(3)

where:
- \( \alpha_s \): 0.2 - 0.25 (≈ ½ sin(θ))
- \( \gamma_w \): unit weight of water

By combining Eqs. 2 and 3, one gets the following expression for the theoretical cone resistance in a normally consolidated clay (Sennens et al. 1989):

\[ q_t = K_c (\gamma z - \alpha_u \gamma z) \]  

(4)

where:
- \( q_t \): theoretical cone resistance
- \( K_c = N_c - \alpha_u \gamma z / (1 + \alpha) \) (= 1.5 - 2.25)

The stress history of a clay deposit may hence be evaluated by plotting a straight line with slope \( 2\gamma \) on the \( q_t - z \) record. If \( q_t \) is considerably larger than the theoretical value given by Eq. 4, the clay is most likely overconsolidated.

A similar approach may be utilized for approximation of the preconsolidation stress \( \sigma_{cp} \). The undrained shear strength may be expressed by Eq. 1:

\[ s_u = \frac{\Delta u}{N_u} \]  

(1)

For an overconsolidated clay, the following relationship between \( s_u \) and the preconsolidation stress \( \sigma_{cp} \) exists:

\[ s_u = \alpha_u (\sigma_{cp} + s) \]  

(5)

Figure 2: Interpretation of preconsolidation stress from CPTU results, western alternative. Total stress approach.

where:
- \( \alpha_u \): 0.22 - 0.30
- \( a \): attraction (≈ c/\gamma z)
- \( c \): cohesion

By inserting Eq. 5 into Eq. 1 and rearranging, one gets:

\[ \sigma_{cp} = \left( \frac{\Delta u}{N_u} - a \right) \]  

(6)

With \( \alpha_u \approx 0.22 - 0.30 \) and \( N_u = 6 - 10 \), the value of the product \( \alpha_u N_u \) may range from 1.5 to 2.25. Eq. 6 represents a very simple relationship between undrained shear strength and preconsolidation stress in slightly to moderately overconsolidated clays. Figure 2 shows interpretation of a CPTU profile in clays and silts from the area of the western alternative. Interpreted values indicate a linear relationship between consolidation stress and depth, except from the upper few meters close to the terrain surface.

As shown in the figure, a straight line through the interpreted values indicates a terrain level approximately 10 m above the present level. This should indicate that the most plausible causes for real overconsolidation could be erosion of previous surface layers and/or, more likely, fluctuations of the groundwater table. However, the distribution of laboratory results from undisturbed oedometer test samples indicate that local...
preconsolidation effects such as previous surface loading, capillary effects, and weathering could be additional reasons.

Similar interpretations were also carried out for 10 CPTU profiles in the railway culvert area, as shown in Figure 3. Using a total stress approach with interpretations restricted to the clay layer, obtained results indicate an overconsolidation equivalent to a groundwater level about 20 m lower than today's level.

Unfortunately, very few results are available from other tests in the area. Neither oedometer tests on samples from the clay layer, nor screw plate tests in the sand show any indication of the preconsolidation stress level, even if they indicate overconsolidated behaviour. The exact stress history of the sediments is hence unknown. Despite these difficulties, CPTU was the only method that gave indications on the preconsolidation stress level. When more experience is gained with these interpretation methods, the quantitative predictions may improve further and increase the potential of CPTU evaluations.

4.3 Deformation moduli in clays

Correlations between CPT results and the constrained deformation modulus presented herein, refer to the tangent modulus $M$ as found from an oedometer test. However, there is no established analytical solution relating CPTU data to any kind of deformation modulus. The soil outside the plastified region around the cone during penetration will not be in a state of failure, even if the stress level may be higher than for the at rest conditions. The stiffness of this soil is hence of importance, and will influence the obtained resistance. Physically, the deformation properties of the soil and the cone resistance $q$ should hence be related. In the interpretations included herein, values of $M$ are estimated from empirically based relationship based on the net cone resistance (e.g. Semmens et al. (1982)):

$$ M = m \cdot q, \alpha $$

(7)

where:
- $m_0$: modulus number
- $q_0$: net cone resistance
- $\alpha$: stress exponent

The stress exponent $\alpha$ is represented by the value 1.0 for clays and fine silts. In the overconsolidated stress range, $m^* + \Delta q < m$, the modulus $M$, can be estimated from the expression:

$$ M = m_1 \cdot q_1, \alpha $$

(8)

where:
- $m_1$: modulus number = 10 ± 5
- $q_0$: net cone resistance

For the normally consolidated stress range, a similar expression is suggested for the modulus value at $q_0$:

$$ M = m_2 \cdot q_0 $$

(9)

where:
- $m_2$: modulus number = $m_{u0}/N_0 = 6 \pm 2$
- $q_0$: net cone resistance

4.4 Moduli in sands and coarse silts

In sands and coarse silts, a value of $\alpha = 0.5$ (square-root adaption) is more appropriate. The corresponding modulus expression reads:

$$ M = m_s \cdot q_0, \alpha $$

(10)

where
The modulus value interpreted from this expression corresponds to an average modulus value in the design stress range \((\sigma_{at} + \Delta\sigma/2)\).

The CPTU interpretations for the sands in the railway culvert area are shown in Figure 4. The CPTU interpretations are compared to oedometer test values on disturbed, recompacted sand samples, with a porosity matching as good as possible the in situ conditions.

The laboratory modulus values are developed for a stress level of \(\sigma_{at} + 0.5\Delta\sigma\).

Based on experience in sand, an average value of \(m_0 = 10\) has been chosen for the CPTU interpretations. The laboratory values range from 9 to 30 MPa with an average of 21 MPa. This corresponds very well with the CPTU predictions in this area, but one may discuss if the recompacted laboratory samples correspond to the real in situ conditions.

In clays, predictions of the modulus in the preconsolidated stress range, \(M_0\), has been carried out using an average value of \(m_0 = 10\). Since no clear indication of \(\sigma_{at}\) is present in the laboratory curve, reference modulus values are developed for a stress level \(\sigma - \sigma_{at}\). It is believed that the real values of the modulus in the preconsolidated stress range are slightly higher, but the preconsolidation effect may have been reduced by swelling effects or sample disturbance.

In Figure 5, the CPTU interpretations are compared to obtained values of the oedometer modulus. The laboratory values range from 20 to 30 MPa, with an average of 23 MPa. CPTU predictions seem to underestimate the modulus for \(m_0 = 10\). This may indicate that the upper limit of the recommended range in \(m_0\) would have been a better choice if the laboratory values are correct.

It is probably more interesting to compare the laboratory reference values with \(M_0\), which corresponds to the modulus value at \(\sigma_{at}\). This would have led to even larger discrepancy, with CPTU interpretations being on the conservative side.
5 FURTHER DEVELOPMENT IN USE OF IN SITU TESTS

The last 5 years have represented a significant and welcome development of more rational in situ equipment and improved test procedures. The aim has been to shorten necessary time for preparation and mobilization of the equipment. The parallel development of more effective data acquisition systems has also contributed to more rational test procedures. The optimization of the field operations should however not interfere with the required accuracy of the measurements. The recommended test procedures should emphasize careful performance of critical operations, such as calibration procedure, temperature stabilization and saturation of the pore pressure system. Shortcomings in these procedures may seriously reduce the quality of the measurements, and may lead to misleading conclusions.

In the future, CPTU and other in situ equipment will probably include a larger number of sensors for measurement of supplementing properties of the soil. This will further improve the versatility and applicability of this test, but also increase the complexity of equipment and test procedures. There may hence be a need of separating in situ tests in various performance classes, each with a specified recommended accuracy depending on the purpose of the test. Such thoughts are already introduced in the Swedish and Norwegian CPTU guidelines.

6 CONCLUSIONS AND EXPERIENCES MADE

The recent development of modern and rational test equipment, improved test procedures and interpretation methods have increased the popularity and use of CPTU considerably. One recent example is the site investigation program for the new Oslo Main Airport, Gardermoen, which represents a milestone for the use of CPTU in offshore investigation practice in Norway. From a general soil characterization based on recordings of cone resistance (q), pore pressure (u) and sleeve friction (f), CPTU results were used along with other information to establish a general stratigraphical model for the area. Test results were also interpreted in order to develop strength and deformation parameters for the various strata, which made very valuable contributions to the recommendation of design values. Use of CPTU and other in situ tests in larger soil investigation programs can be a very cost-effective alternative, since it may reduce the necessary amount of soil sampling and laboratory testing. However, there is still a need for local calibrations of soil parameters developed from in situ tests. In this project, in situ test results were particularly useful in soils where undisturbed samples were difficult to get. In these soils, design values of various soil parameters were mainly based on CPTU interpretations.

The overall experiences made and the conclusions drawn from this project can be summarized as follows:
- CPTU provided excellent soil mapping capability, a must in this project.
- A rational site investigation program was obtained, due to reduced sampling and soil testing, more cost-effective field operations with modern and reliable test equipment.
- Soil parameters were interpreted for all strata to large depths, and a large database of test results was established.

The soil investigations at Gardermoen have proved the usefulness of combining different types of equipment and methods in the determination of soil properties, with CPTU playing a very important role. The basis for engineering design and the potential for a depositional environment understanding, has been greatly improved by the use and interpretation of CPTU results.

7 ACKNOWLEDGMENTS

Financial support was provided by the Norwegian Aviation Administration for the sedimentological and hydrogeological field and laboratory investigations. Oslo Hovedflyplass A/S (OSL) has kindly permitted the use of data from the soil investigations. We also appreciate the work of M.Sc. Håkon Kolberg and the valuable contributions from professor emeritus Nilsur Jadhun at the Department of Geotechnical Engineering, NTNU. Thanks are also due to Jan Ove Busklien (SINTEF), Eirik Fyn and Randi Settner (NTNU) for excellent drawing assistance.

8 LITERATURE AND REFERENCES

Testing of Soils (TC 16).
Reference Test Procedures.
Swedish Geotechnical Society, SIGF, Information, 2. Linkoping.
Robertson, P.K. (1998)
Soil Classification Using the Cone Penetration Test. Canadian Geotechnical Journal, 27 (1): 151-158.
Sandven, R. (1990)
Strength and deformation properties of fine grained soils obtained from piezocene tests. Dr.ing. dissertation 1990.3. Department of Geotechnical Engineering, Norwegian Institute of Technology (NTH), Trondheim, Norway.
Sandven, R. (1992)
CPT '95. Soil classification and parameter evaluation from piezocene tests. Results from the major site investigations at Oslo Main Airport, Gardermoen. Thesis lecture. Proc., CPT'95 International Symposium on Cone Penetration Testing, Linkoping, Sweden. 3.35-55.
The site investigation program for the Oslo Airport, Gardermoen, Norway. I. Site investigation strategy. To be presented in Proc., ISDR'98 First International Conference on Site Characterization, Atlanta, USA.
SINTEF Geotechnical Engineering (1993)
Site investigations, Gardermoen. Evaluation of ground conditions. SINTEF report no. 31/489-33023, Trondheim, Norway.
Stratigraphical and sedimentological modelling at Oslo Main Airport, Gardermoen. Proc., 12th European Conference of Soil Mechanics and Foundation Engineering, Copenhagen, Denmark. 6:221-227.
Site characterization by means of the natural slope methodology (NSM)

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ABSTRACT: This paper is a synoptic presentation of those NSM features which enable a variety of site characterization applications in areas with natural slopes in rocks and/or soils, and which are not limited only to geotechnical applications, but can also be used for geological, mineral and petroleum exploration. NSM allows estimates of a sizeable number of geomechanical, slope stability and other parameters, whose validity stems from 1. the mapped trends of the NSM results correspond to the disposition of geostuctural features; 2. their satisfactory degree of identity with the results of conventional tests; and, 3. the deterministic causality relations underlying the "nearly perfect" correlation between the L and H measurements of natural slopes. These measurements are the only basic NSM requirement, and are easily made on adequate quality and scale topographic maps; and unlike the conventional site characterization methods, NSM does not require field work, borings, nor laboratory or in-situ tests.

1. INTRODUCTION

This paper deals with those features of the Natural Slope Methodology (NSM) that enable site characterization applications in study or project areas where natural slopes in rocks and/or soils are present.

The first text touches succinctly on the main pertinent features of the sizeable field of knowledge of NSM: its operational and analytical levels, its measurement and sampling systems, its data base, its analytical system, the mapping of its parameter's results and the causality relations which underpin the satisfactory confidence levels of its quantitative results; and then it proceeds to describe synoptically the range of site characterization possibilities offered by NSM, not only within the geotechnical exploration field, but also in other fields somewhat far removed from NSM's early initial purpose of simply serving as an economical and rapid slope stability analysis method.

Given the tight length limitations for this text, and the amplitude of NSM's subject matter, it is not possible to enter into any degree of detail on any one of the subjects presented herein; and this necessarily leads to an introductory and descriptive type of text, and a very summarized exposition of all the topics here treated. However, the references cited in the text can provide from some to many of the particulars lacking in this paper.

2. MAIN PERTINENT FEATURES OF NSM

The key elements of NSM are described in some detail in relatively recent papers (Shuk, 1994a, 1995, 1996a and 1996b), and also in an extended and comprehensive text (Shuk, 1998). It is an empirical and analytical methodology developed in two stages: from 1953 to 1970 (Shuk, 1968 and 1970), and then from 1978 to the present; and it is inscribed within the not so recently conceptualized and operationalized framework of Macrogeotechnics (Shuk, 1999).

In its present state, NSM is an original and novel methodology, which in some ways is different from the conventional geotechnical approaches. The basic starting premise of NSM is "nature is the best in-situ test", and its research and practical application efforts center around the possibility, at present verified to a large extent by means of its use in a number of different types of practical projects of various sizes, of a quantitative interpretation of the resistions to these massive, exogenous and endogenous in-situ tests applied by nature during very long periods of (geologic scale) time. The multitude of these different and varied reactions can all be lumped together under the term "adaptation process".

The visible, interpretable and quantifiable features of this adaptation process are the presently existing geofoms, and in the particular case of
NSM, their natural slopes, two of whose geometrical dimensions (L and H) are easily measurable on adequate scale and quality topographic maps. These measurements have led to quantitative outputs of a satisfactory confidence level, as determined by comparisons with conventional empirical and analytical results in those projects where these had also been carried out (for example: Shuk, 1994a, Table 2; Shuk, 1994c, Table 2).

Unlike the conventional site characterization methods, which in many instances require costly and prolonged field tasks, borings, and laboratory and/or in-situ tests, NSM only requires the availability of topographic maps of an adequate quality and scale, of a general type of geological information, of personnel knowledgeable in the NSM natural slope measurement systems, and commonplace office equipment.

Accordingly, the ratio of NSM's costs with respect to those of the prevailing conventional ones, under the assumption of an identical density of information coverage, ranges between 1/1000 and 1/10000; and the completion time of a study using NSM may be a fraction of that required by the conventional methods.

However, whenever the conventional site characterization methodologies are irreplaceable, as can occur in a number of study or project situations, NSM can be used either before the conventional exploration, so as to pinpoint problem areas (and, thus, effectively guide this exploration); or in its later stages, as a valuable complementary tool.

2.1 Operational and Analytical Levels of NSM

NSM has three basic operational and analytical levels: family (the set of individual natural slopes which are present within a family delimitation covering a part or the whole of a hillslope); population (all the families within a given geological formation, or geological member); and universe (all the populations within a given study or project area). Depending on the objectives of a study, and the size of its area, in certain cases the population level has to be subdivided into a number of subpopulations.

2.2 Measurement of Natural Slopes and their Sampling

At the family level, the NSM Sequential Cumulative Contour Curve Interval (SCCCI) system is used. This system is illustrated in Figure 1, and consists of cumulative 1, measurements starting from the bottom of the family delimitation, of the maximum (minimum L) and minimum (maximum L) topographical gradient contour intervals, until the top of the family delimitation is reached. An H (height) dimension corresponds to each L. result and, at the top, the maximum limiting dimensions are represented by H_D and H_L. Depending on the shape of the family delimitation, sometimes these measurements have to be started from its top.

The description of the various natural slope measurement systems, and the rules and criteria for the family delimitation are presented briefly in other papers (Shuk, 1995, Section 3.5; Shuk, 1996b, Section 3.1 and 3.2), and in a more detailed form in the NSM basic manuscript (Shuk, 1998, Chapter 8). Figure 1 also illustrates superficially the concept of family delimitation and one of its basic rules: the direction (or azimuth) of the measurements which differentiates family ESQI-4 from family ESQI-5s.

There types of information coverage are used by NSM: 1. biased sampling, which is used when required by specific objectives of a study or project (Shuk and González, 1994, figures 1 and 3); 2. random sampling, used, for example, when the speed required to obtain slope measurement data in a large study or project area is of the highest importance; and, 3. total coverage, which implies the measurement of all the natural slope families within a targeted area. Total coverage cases are shown in other references (Shuk, 1996b, Figure 2, Shuk, 1998, Chapter 8). Notwithstanding its designation, this type of coverage can result in blank spaces on the family location maps of a given study area, which correspond to zones that do not comply either with the study's objectives, or with the family delimitation rules.

![Diagram](image-url)

**Figure 1.** The SCCCI natural slope measurement system used on the Bogotá-Villavicencio highway.
2.3 NSM Data Base

The presently available NSM data base is voluminous and consists of 3162 families, 153 populations, and 32 universes of practical studies and research projects, in rocks of all types of origins (igneous, extrusive and intrusive, sedimentary, metasedimentary, and metamorphic), and in a great variety of lithologies and textures; and in volcanic, alluvial, glacial and marine deposited soils, plus all the possible types of parental rock derived soils (rock-roll transitions, saprolites, mature residals and colluviums).

Most of this data base consists of natural slope measurements in Colombia, but it also includes the results of natural slope information gathered from the geotechnical literature dealing with cases in Canada, France, Hong-Kong, Panama, Spain and the United States. A number of these information items also include conventional geotechnical empirical and analytical results, which constituted the initial steppingstones of NSM's development process.

Extrapolating from the statistical average results of the total number of individual natural slopes (6107) which were identified within 194 measured family delimitations, the total number of families included in the presently available data base may well be equivalent to some 100000 or more individual natural slopes.

2.4 Analytical System of NSM (ASNSM)

This system consists of the following components:

A. At the family level: a correlation function based on the average results of the measurements made on topographic maps of two geometrical dimensions (L and H) of natural slopes, and which corresponds to a simple power function (\( H = a L^b \)).

This function evidences a "nearby perfect" correlation coefficient, whose overall NSM data base average value is 0.992.

At the population (subpopulation) or universe level: a simple power function (\( H = a L^b \)) resulting from the logarithmic average slope trend of the Triangular Pattern. The functional parameters \( a \) and \( b \) of these simple power functions are expressions of the following geomechanical parameters: pressurization, unit weight (\( \gamma \)) and strength (\( c, \phi \)) (Shuk, 1994b-p.258). The Triangular Pattern is established by means of the limit values (\( L_{U2} \) and \( L_{L2} \)) of all the families belonging to either one of these two levels. The upper and lower limits of this Triangular Pattern correspond respectively to the minimum and maximum slope trends, and they are governed by deterministic causality relations (Shuk, 1996a-Section 3.1; and Shuk, 1996b-Section 5.3.1), as briefly described in the next section of this text.

Although the Triangular Pattern also exists at the family level (see lower right - hand side box of Figure 1) and is, in fact, a universal law for all the natural slopes existing in geologic materials, it is not taken into consideration at this primary NSM analytical level.

B. The Transformed Shear Stress Envelope (TSSE), obtained by means of a dimensional one-to-one transform factor (whose value is always 1.0 in the MKS dimensional system) from the L vs. H simple power function of component A; and it is represented by a vertical stress (\( \sigma_H \)) vs. shear stress (\( \tau \)) plot (Shuk, 1998-Chapter 13).

C. A matrix layout system whose nodes represent a large number of so-called Hypothetical Stress Envelopes (HSE). Each one of these envelopes originates from systematically consistent and arbitrarily predetermined (but different for each HSE) mathematical manipulations of some of the TSSE's parameters, particularly those related to strength and pressurization (seepage or pore pressures, etc.), and, thus, they all have a common origin in the previous ASNSM component (Shuk, 1998-Chapter 14).

D. The computerized Algorithm for the NSM Geomechanical Parameter Estimates which, at present, estimates 118 of these parameters for four different kinds: pressurization, density & phase (specific gravity, porosities, unit weights, etc.), strength and deformability.

A number of the estimates of the first two kinds of geomechanical parameters originate from correlations between pairs of HSE parameters among fixed groupings of some of the hypothetical envelopes of the previous ASNSM component (Shuk, 1998-chapters 17 to 19). These pairs and groupings are different for each one of these estimates.

On the other hand, the strength parameters, which can be estimated in terms of total or effective stresses, are obtained from geometrical relations between vectors placed in the TSSE according to geotechnical logic and criteria (Shuk, 1998-Chapter 20). Both the strength parameters, as well as the deformability parameters (Shuk, 1998-Chapter 21), which also conform to those logic and criteria, can be given in terms of either a "Condition of the Mass" or the so-called "Laboratory Condition".

The results of this second condition as well as those of some density & phase parameters pertaining to the Constitutive Element of the mass (specific gravity, dry unit weight, primary porosity, etc.), all simulate laboratory situations, and permit comparisons between the simulated NSM and the real conventional test results.

Up till now, the differences between these two types of results usually ranged between 2% and 8%, and in a few cases they were of the order of 10% (or...
somewhat more) and, in others, of less than 1% (for example: Shuk, 1994a-Table 2).

F. The NSM Slope Stability and Related Parameters: safety factor, failure probability (hazard) and economic decision (risk, expected failure magnitudes and their probabilities, and expected costs, etc.) parameters, whose initial estimates originate directly from the L vs H plots, and whose results are relative to the NSM level in which they are made (Shuk, 1989 - p.728 to 741; Shuk, 1996b-Section 4).

G. The NSM Mass Characterization Parameters: degradation, disaggregation and variability parameters, which characterize the present evolutionary state of a given mass of geologic materials, and are obtained from the relative position of the simple power function within the Triangular Pattern of the population (or subpopulation) trend and from this pattern's amplitude (Shuk, 1996a-Section 3.4; and Shuk, 1996b-Section 3.3.9).

2.5 Mapping of NSM Parameters

Apart from the fact that the NSM parameters can lead to discrete and specific types of quantitative results, which are geared towards the given and predetermined objectives and needs of a given study or investigation, its massive quantification capabilities allow continuous types of results such as the arithmetical isolo value contour curve maps for any one of these parameters within a given study area, thus enabling an appreciation of their variations; and the residual isolo value contour curve maps, which highlight even more these variations and their trends. Accordingly, these NSM originated mapping tools can be of great value in a number of different types of site characterization studies or projects.

An example of the applicability of the NSM quantitative mapping tools is illustrated in Figure 2, which shows the distribution trends of the specific gravity within a study area, and which highlights the association of the high value anomalies of this parameter with many of this area's faults, thus suggesting conditions of fracture or (to a lesser degree) of secondary mineralization. This particular application does not only have some relevance for underground works, but also points to its usefulness in the exploration for heavy minerals bodies.

Examples of the results of this type of maps for other NSM parameters (pressurization, porosity, strength [c, q], deformability, and safety factor) are shown in other writings (Shuk, 1994c - figures 2 to 4; Shuk, 1996a - figures 10, 13, 16, and 18 to 20; Shuk, 1996b - figures 20 and 21; Shuk and Hernández, 1996 - Figure 3).

The previous NSM mapping possibilities would enable it to be included in a GIS software package. However, for this to occur, a few subjective decision items which still remain in the NSM family delineation criteria (Shuk, 1998 - Chapter 8), will have to be converted into decisions of an objective nature. Thus, the whole NSM quantification process could become totally automatized, starting from digitized topographic maps (or aerial photographs), and ending in either discrete type and specific results, or continuous type arithmetic and/or residual isolo value contour curve maps of the parameters needed for a study or investigation (within anyone of the NSM fields of applicability) of any study area which evidences natural slopes.
Table 1. Site characterization applications of the Natural Slope Methodology (NSM).

<table>
<thead>
<tr>
<th>Target</th>
<th>Analysis items, main parameter(s)</th>
<th>Ref. (2)</th>
<th>Usable for</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stability of cat slopes and excavations.</td>
<td>Direct NSM method; Slope stability parameters; Maximum potential height; Cohesion anisotropy; Differentiation of derived soils.</td>
<td>T 94b/262-265; T 94a/248-249; T 94a/249-253; T 94a/243-248</td>
<td>Rock and excavation in civil engineering projects of all types; Open-pit mining.</td>
</tr>
<tr>
<td>Underground works.</td>
<td>Zones of high infiltration risk (high η anomalies). Activation or dissolution tendencies of faults (u). Design of roof supports (c, q). Design of temporary or permanent supports (Md). Determination of relative excavation rates (κ, υ, others).</td>
<td>T 94c/274; T 94c/272, 278; 96a/25, 26; T 94c/274-276; T 94c/274</td>
<td>Tunnels, caverns, chambers, etc.</td>
</tr>
<tr>
<td>Foundations.</td>
<td>Bearing capacity; pertinent geomechanical parameters (c, q).</td>
<td>U</td>
<td>Buildings, bridges, other types of structures.</td>
</tr>
<tr>
<td>Construction of civil works.</td>
<td>Rapp赏ibly and other characteristics based on geomechanical parameters.</td>
<td>U</td>
<td>More rational use of excavation equipment and explosives.</td>
</tr>
<tr>
<td></td>
<td>Detection of geotechnical features,* High η anomalies and others.</td>
<td>U 96a/20-29; T 96a/1-38</td>
<td>Exploration for heavy minerals.</td>
</tr>
<tr>
<td></td>
<td>Detection of geotechnical features,* Excess pressurization anomalies (u).</td>
<td>U 94c/274; T 96a/1-38</td>
<td>Petroleum exploration.</td>
</tr>
<tr>
<td></td>
<td>Detection of geotechnical features,* Artesian water sources.</td>
<td>T 96a/1-38</td>
<td>Geohydrology.</td>
</tr>
</tbody>
</table>

(1) (T): tested application; (U): untested application (see Section 3 of text).
(2) See references after the end of the text (year's last two numbers/page numbers).
* not detected by conventional exploration methods.

2.6 Causality Relations Undermining NSM

The high confidence levels of the NSM geomechanical parameter results obtained by means of its algorithm, and by its other types of quantitative parameters, are undermined directly by both deterministic and chance causality relations. The deterministic ones are based on the following geomechanical characteristics of a given mass of geologic materials: its strength (cohesion, basic friction angle and, in the case of a rock mass, by the friction angle between the minerals composing it), its pressurization; and other characteristics related to its evolutionary stage within the adaptation process, and which are quantifiable by means of the NSM mass characterization parameters (component F of the ASNSM).

The same as in the case of the Brownian Movement of gases, this deterministic type of causality is evidenced concurrently with a chance (probability) type of causality relation represented by the joint probability distribution of the simultaneous occurrence of the two geometrical dimensions (I. and H) measured by NSM in natural slopes (Shuk, 1996b- Section 3.3 and Figure 4).

The previously described direct causality relations are complemented by indirect ones, such as the already mentioned satisfactory degree of identity between the NSM simulated "Laboratory Condition" results and those of real laboratory tests; and the NSM arithmetical and residual isovalue contour curve map results, which always evidence a correspondence between the trends of the quantitative NSM results within a given study area, and the alignments of those physical geology features (geostuctural, tectonical, lithological) which mold them (Shuk, 1994c and 1996a).
3. SITE CHARACTERIZATION APPLICATIONS OF NSM

The present scope of tested and unstressed NSM site characterization applications are synoptically shown in Table 1. There are indications that their number may increase somewhat in the future. The unstressed ones are clear possibilities pointed out by the results of one or more past NSM practical cases, but their specific applicability has not yet been corroborated.

4. CONCLUSIONS

The advantage of the Natural Slope Methodology (NSM) as a site characterization tool lies in that it only requires natural slope measurements on adequate quality and scale topographic maps, whereas the conventional methods require significantly more cumbersome, costly and time consuming field tasks, borings and testing.

At first sight, the methodological simplicity implied by this advantage may seem to be illusory. But a prolonged and profound analysis, such as the one whose outstanding resulting features are presented summarily in this paper, points to the mild and deeply rooted validity of the NSM fundamentals and, hence, of its practical results.

In turn, these results lead to a sizeable scope and variety of NSM site characterization possibilities, the majority of which have been tested in practical project applications; and which overflow the mere area of Geotechnics into other important fields, in which NSM can be used as an auxiliary or complementary tool. The same complementary type of use can be given to NSM in the cases where the conventional methods are irreplaceable.

In spite of its satisfactory present state, NSM (including its unstressed application possibilities) still continues requiring a strong and persistent research effort, until it reaches its full maturity.

REFERENCES


Pressuremeter deformation modulus for dynamic compaction at Öresund bridge between Sweden and Denmark

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ABSTRACT: The deformation characteristics of urban soils and land fills are possible to predict under certain circumstances. The combination of various soundings, pressuremeter tests and large scale field tests, combined with historical survey, can provide sufficient information. The results from this case support the concept that deformation stiffness growth vs. applied compacting energy can be predicted also in land fills using the pressure meter, and is also comparable with the average, measured large scale surface settlements.

1 INTRODUCTION

In 1991, the agreement to construct the Öresund bridge was made between Sweden and Denmark. To represent the Swedish part and to integrate this project to existing infrastructure, the organization SVEFAB was set up, owned 50/50 % by the Swedish National Road Administration (Vägverket) and the Swedish National Rail Administration (Banverket). Their task includes some 40 bridges, 10 km of freeway and 20 km of railway tracks.

1.1 Background

The first serious proposal for a permanent bridge over Öresund, connecting Denmark and Sweden came in the early 1970’s. A serious attempt to build the bridge was made in the early 1970’s, but it took some 20 years more until the decision was made to build the bridge. The city-planner on the Swedish side in Malmö and the Swedish National Road Administration had although known for many years that the bridge decision would eventually come, so corridors in the landscape had been left unexploited, as well as existing infrastructure been planned, to allow the traffic from the bridge easy access to the Swedish freeway system. Over the years, land required has been purchased, except, among other areas, the planned eastern abutment of the bridge. One reason for this is that the area was in use for dumping of residual material from cement manufacturing nearby since almost a century.

1.2 Preliminary site investigations

Man-made landfill is obviously difficult to characterize since it is not a naturally sorted material that we have knowledge or experience about from geological processes or previous successful constructions. Therefore, a historical investigation is a good start to build the model of expected filling materials. The historical survey included city archives, documentation from the cement manufacturer, reports to local authorities, maps, air-photos, interviews with former employees and previous geotechnical investigations. The survey showed that the area, called Lemmeken, consisted of some 13,5 million m³ of landfill, of which approx. 1 million m³ was expected to be various waste-material. The rest of the material was expected to be clay-till and limestone which probably could be used for construction purposes.

The geotechnical drillings confirmed the previous reports of a large variation of the ground strata. The profile shows some 11 m of soft fillings, overall with high silt contents, drained to sea-level approx. 10 m below surface. The natural contents of flint-stone in this limestone were the equipment heavily and caused considerable drilling difficulties. Waste material and organic soils were only encountered in the eastern central part and would either be excavated for building the road and railway corridor or be capulated along its vicinity.
The preliminary investigation concluded that it was of major interest to locate the approx. 1 million m³ of possibly contaminated waste material and to investigate to what extent the existing landfill could be used for construction purposes.

2 SITE INVESTIGATION

Due to the degree of contamination of the site, and to a certain extent the legal situation, the site investigation was divided into one environmental and one geotechnical part.

The environmental part involved mostly fillings that needed to be excavated and handled in a safe manner. But since these masses will not be a part of the actual construction works, this part will be omitted from this article.

The geotechnical investigation is divided into two parts due to planned construction activities: expected geotechnical conditions from the preliminary investigations as well as for topographical reasons.

In the western part, a planned access ramp to the bridge rises up to 25 m above sea level with a base width of 75 m on top of 10 m of loose filling. It was recognized that proper ground reinforcement in this area would yield considerable savings.

In the eastern part, the railway tracks and roads will pass over fillings which have been consolidated by overloads of 12 to 16 m of existing fillings. The excessive overburden combined with only limited expected ground reinforcement, investigations were executed during excavations.

The site investigation for the western part involved cone penetration tests, dynamic penetration tests (Swedish standard H(h)), percussion drillings and auger drillings for sampling. Acquired soil samples were sent to laboratory for examination. The water contents and grading were of interest since the possibility of heavy dynamic compaction had appeared early as an interesting form of soil reinforcement in this area. The area was also surveyed using resistivity measurement for metal contamination which also could possibly detect deposits of organic soils. Areas with low resistivity were auger sampled, but no organic soils were detected in the low resistivity areas.

The investigation concluded that most of the fillings were weathered limestone, deteriorating to silt fraction, well mixed with clay-till. Overall water contents were low, in the range 15-35%. The top layer of 3 m was dense, but below this layer the fillings were very loose. The modulus of elasticity was estimated at 20 MPa in the top layer and 1-3 MPa in layers below. Two deviating deposits were located. One of these containing approx. 20,000 m³ filtered dust from cement manufacturing; the other was a larger area where the filling consisted mostly of clay-till. The filter-dust were located where the embankment was 4-6 m high and the area of high clay till contents was assumed to be within the normal fillings variation, on the less favorable side. It was also concluded that the large variation of the filling appeared to be persistent, i.e. there was no area showing significant better nor worse conditions from a geotechnical point of view.

The geotechnical site investigation concluded that ground reinforcements with heavy dynamic compaction and pre-consolidation with overloads probably would provide acceptable ground conditions. Therefore, piling works could be limited.

2.1 Large scale field tests

To assess the characteristic parameters for detailed design, large scale field tests were required. Two areas of 30×30 m with less favorable ground conditions were chosen. In each of them, four point were investigated using CPT, auger sampling and pressuremeter. Due to the harsh conditions for pressuremeter, steel sheeted 54 mm French Menard probes were used. It was not possible to pre-drill, the probe was installed by ramming. Typical results from CPT in the test areas are shown below.

![Figure 1: Principal cross-section in the western part.](image-url)
Figure 2: CPT result from test area 1, pore pressure not shown.

Figure 3: Settlement of the filling after heavy dynamic compacting, total 1089 drops, compacting 393 m³.

Figure 4: The outcome of performed pressuremeter tests before / after compacting test area 1.

Test area 1 was used for heavy dynamic compacting using a free falling weight of 16.5 tones, bottom size 2 x 2 m, from 17 m height with acceleration of 8 m/s² limited due to the wire. First, the area of 30 by 30 m was evened and prepared with a dynamic compaction roller. Thereafter, a terrain-model was established covering approx. 40 by 40 m to allow monitoring of the surface settlements. The heavy dynamic compacting was carried out in rounds of three drops, spaced 3 m. After each round the surface was evened off and roller compacted in the same manner as before. The terrain model was then re-established to allow calculations deformations. These operations also provided a resting period of 2 weeks for the soil, allowing eventual pore over pressure to decline. The compaction was stopped when the compacting potential of the filling was fulfilled and shear failure appeared as heave movements around the test area.

Two weeks after completing the compacting, the four original investigation points were re-investigated with an offset of 1.5 m to determine the compacting effect.

In average per 9 m³, 10 drops were performed. Average settlement was 0.44 m. Evaluated compaction depth was 10 m. The pace of compacting was in this test approximately 12 drops per hour, limited due to heat problems of the brakes to the lifting wire drum.

Performed CPT soundings did not indicate any significant difference after the compaction, neither did the dynamic penetration tests.
Test area 2 was used for testing pre-consolidation with overloaded embankments. Two embankments were constructed, 3.5 m and 7 m in height, both equipped with a high precision settlement measuring system designed to measure only the deformation of the 12 m thick filling. The top area of the embankments were 20 by 20 m.

The time-related deformations followed very well a logarithmic scale with 14 / 23 mm per log(10) step. After 3 months, the top 1.5 m of the 7 m embankment was removed and the settlements seized completely.

2.2 Analysis

It was early recognized that the commonly used calculations for determining the deformability of soils using the pressuremeter was not applicable. This is probably due to the combination of installation method and history of the filling material giving steep foliation. So, the analysis started with assessment of the correlation factor between the pressuremeter modulus $E_v$ and the characteristic elastic modulus of the filling, $E_v$.

For the stress distribution of the embankments overload, the elastic theory was used. The pressuremeter test values were evaluated by level and separately, omitting extreme value possibly caused by faulty installation of the probe or extreme local conditions around it.

The analysis resulted that:

$$E_v = k \times E_v$$

where $k = 3.15$

whereas the expected range was $k = 1.5 - 2$.

The design value for elastic modulus of the filling was then calculated as:

$$E_{d} = E_v / \chi_e$$

Where $\chi_e$ is the material uncertainty factor based on 95% probability that the design value will be more favorable than the expected characteristic value. For Test Area 1 where this is of interest, $\chi_e$ could be calculated to 1.25 using a normal distribution with 95% confidence interval. This was unexpected low with regard to the crude installation method and the expected variation of the material. For design purposes, the value of 1.5 was used in accordance to regulations.

The next question to analyze was if the accomplished, heavy dynamic compaction in test area 1 was sufficient to allow foundation of the access ramp. Due to the layout of the traffic system with the railway track in a concrete tunnel on concrete piles, see figure 1, partly under the road structure, the road surface is quite sensitive for differential settlement. The criteria for differential settlement in the road surface is given by the Swedish National Road Administration’s regulations. For the stress distribution of the load of the embankments, elastic theory was used. Also, the assumption was made that the settlement of the bank material is equal to or less than the settlement developed in the filling during the construction period of the bank. This is a conservative assumption since the settlement in the embankments are likely to be very small and to develop quickly, see figure 5 above.
Under the circumstances stated above the expected settlement in the filling could be calculated to be 64 mm in the most unfavorable cross-section while the code criteria stated that up to 84 mm was acceptable. The time related deformations were also verified to be within the criteria.

For the ground reinforcement of the whole area, two acceptance criteria for the heavy dynamic compacting were formulated.

Energy acceptance criteria:
When the energy compacting factor has exceeded the required value according to the results from Test Area 1, the compaction could be stopped. In this case the required value was 15 MNm/m² calculated as applied energy (MNm/m³) divided by compacted volume (m³/m²).

Developed settlement criteria
The compaction could be stopped, when developed the settlements due to compacting exceed the design settlements, calculated for the case that the embankment had been constructed without any ground reinforcement.

Values for result control using pressurometer were calculated to require a pressurometer modulus of minimum 4 MPa from the level 4.0 m a/s 1 and a minimum of 2.5 MPa below this level.

3 HEAVY DYNAMIC COMPACTION
The heavy dynamic compaction was carried out in the spring of 1997, covering approx. 18,000 m³ and using a falling weight of 16.7 tones dropped from 16.5 m. The crane used was a Liebherr 512 specially equipped for compacting works. The acceleration was measured to 8.1 m/s². The whole area was before compaction prepared by a dynamic roller compactor and a terrain model was established and readings were taken after each complete round in the same manner as performed during the testing. The compaction sequence was altered, compared to the test program, in the manner that spacing in the first two rounds was 6 by 6 m with a geometric centric offset. The third round covered the points in-between, leading to a 3 by 3 m grid. The 3 by 3 m grid was subsequently used.

The compaction was completed after some 3 months with a production rate of up to 60 drops per hour. Initial production problems involved primarily uncontrolled flying of the twin cable going out from the drum after the weight had hit the ground, causing the cables to fall off the top wheels. This could be handled by covering the cable stretch with a wide rubber band. Flying rocks also caused a minor safety problem.

Evaluating the incoming production results caused some concern as the settlement stiffness growth vs. applied compacting energy was slower and showed no tendency to level off as was expected. It was later concluded that this phenomenon probably had two reasons. First, the soil in the test area had a higher content of stiff clay-till with a smaller compaction potential; secondly some energy in the first compacting rounds was lost due to soil surface after rains, "soils splashing".

The compaction were stopped after meeting the developed settlement acceptance criteria. Performed compacting was controlled in 9 points measuring the pressurometer modulus on 4-5 levels at each point. The results are graphically presented below, see figure 7. The control points were chosen in areas with small or large surface settlement. But the investigation could not verify any significant correlation between surface settle and pressure meter modulus profile for the individual investigation points. This is probably due to both the method of preparing for measuring the settlements as well as actual local variations.
Measuring results proved that the compaction in most points exceeded the stated requirement. Three measurements did not meet the requirement. Calculations for the overall area resulted that expected settlement of the embankment is 34 mm while the statistical 5% and 95% values are 14 and 164 mm. Since these design values were higher than expected, all inclined piles had to be verified to hold the bending moment induced by settlements of the soil. Holding in mind the fast settlement development, Fig. 5, and including the proposed settlement monitoring program, is was concluded that the ground reinforcement was sufficient.

4 CONCLUSION

Land fills of various residual materials will increasingly be of interest for construction purposes. Projects of this kind will gain very much in confidence if it is possible to reccrate the filling operations so that the risk of embedded organic soils and domestic waste can be handled. Given this information, it is possible to formulate a sufficient investigation program and to analyze the outcome so that reasonable design parameters are accessed.

ACKNOWLEDGEMENT

The author is grateful to SVEADAB for their permission to use field data and other information necessary to produce this paper.

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A review of artificial intelligence systems for site characterization

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ABSTRACT: Site characterization has become one of the most extensively developed areas of application for artificial intelligence systems in geotechnical engineering. A total number of 33 knowledge-based systems and 14 neural network systems have been reported in the literature. Systems have been developed for site investigation planning, interpreting ground conditions, classification of soil and rock and the interpretation of geotechnical parameters. Techniques used have involved a wide range of techniques: simple rule-based approaches; probabilistic methods; fuzzy sets; object-oriented programming; case-based systems. Many systems are only developmental prototypes, with much more work needed before they would prove useful in geotechnical practice. Nevertheless, they do demonstrate the potential of both knowledge-based systems and neural networks in this field. Hybrid systems which combine different artificial intelligence techniques and other techniques (eg database systems, Geographical Information Systems) will probably prove to be the most useful type of system for site characterization in the future.

1. INTRODUCTION

One of the earliest reported geotechnical knowledge-based systems (SITECHAR) was developed for site characterization in 1985. Since that date, site characterization has become one of the most extensively developed areas for geotechnical applications of both knowledge-based systems and neural networks. A total number of 33 knowledge-based systems and 14 neural network systems have been reported in the literature.

Toll (1996) has reviewed artificial intelligence system applications in geotechnical engineering, updating and extending the review of knowledge-based (expert) systems by Moula et al (1995). Details of systems for applications other than site characterization can be found in the review by Toll (1996).

The systems developed for site characterization have been developed for site investigation planning, interpreting ground conditions, classification of soil and rock and the interpretation of geotechnical parameters. Techniques used have involved a wide range of techniques: simple rule-based approaches; probabilistic methods; fuzzy sets; object-oriented programming; case-based systems.

Systems for classification and parameter assessment of soils and rocks are not addressed in the present paper but are described by Toll (1996). Neural networks have been found to be useful for classification applications, particularly in rocks. They have also been used to predict stress-strain behaviour of earth materials. Knowledge-based systems have been used for interpreting CPT data, for estimating values for geotechnical parameters and for rock mass classification purposes.

2. SITE INVESTIGATION PLANNING

SOILCON (Wharry and Ashley, 1986; Siller, 1987) was one of the earliest KBSs to address the problem of determining the required level of geotechnical investigation. This is based on the requirements of a proposed structure and the level of information known about the site. The aim is to reduce the risk involved with the subsurface to an acceptable level. The knowledge base contains 24
investigation techniques, ranging from preliminary (e.g. reviewing topographical maps) to more sophisticated (e.g. pressuremeter), that are used to make the ultimate recommendation. The complexity of the recommended investigation increases when there is a large amount of site data available. One of the limitations of the system is that it does not handle geometric descriptions of the problem and site quantitatively.

Akin and Munroe (1987) present a very simple prototype KBS for soil investigation. It offers guidance on soil identification based on visual and physical observation of soil characteristics. It combines information about the soil and loading conditions to suggest the most suitable sampling and drilling techniques for the particular investigation scheme. However, the knowledge in the system is rather simplistic textbook knowledge. Davey-Wilson et al. (1988) identify some of the limitations of the system.

Probabilistic analysis was used by Halin et al. (1991) in a KBS to assist engineers in making site exploration decisions. The system generates an inference of prior estimates of soil anomaly characteristics (such as lenses or pockets of soft soils within the regular soil deposit) and uses probabilistic analysis in the selection of the most appropriate exploration program.

Smith and Oliphant (1991) describe a KBS to assist with the planning stages of a site investigation. The system provides suggestions as to the next stage of the site investigation (e.g. desk study, site reconnaissance, ground investigation etc.). The information obtained from the subsurface exploration stage is also used to create a 2-D visual representation of the soil layers. This has been developed further by Oliphant et al. (1996) as part of the ASSIST system which is described later.

CESSOL (Magunan, 1992) is a more advanced system that can give qualitative advice on the type of investigation needed, and what sort of testing would be required. It can also give quantitative advice on the number of boreholes and piezometers and amount of testing required.

The system for site investigation of trunk road projects described by Thomas et al. (1993), Winter and Matheson (1992) and Winter et al. (1996) is similar to the approach used by Smith and Oliphant (1991). In this case an activity log of an investigation is produced for comparison against a list of mandatory and advisory procedures contained within the system. The system can therefore be used to highlight omissions in the way that an investigation has been carried out.

A KBS for assisting in the selection of appropriate field tests is presented by Moulah (1993) and Moulah & Toll (1993). The system can be used to advise which test can provide a required parameter (with a specified reliability) and how applicable it is to the specified ground conditions.

Fung and Kay (1996) have used a probabilistic approach to the selection of an appropriate field testing strategy, thereby extending the approach used by Moulah (1993). Their system (Soil Exploration Planning System, SEPS) provides quantitative information in addition to qualitative advice.

3. INTERPRETING GROUND CONDITIONS

One of the earliest geotechnical knowledge-based systems was SITECHAR (Norkin, 1985; Rehak et al., 1985). This is a KBS which uses geometrical reasoning to develop inferences about the depositional patterns of the subsurface materials and their physical properties. It uses field and laboratory data but also takes into account existing experience of geology and geomorphology at a specific site or at similar ones. The initial SITECHAR system incorporates knowledge of geometry and trends, matching soils by description, proximity (such as "near", "above", etc.), geomorphology (such as erosional surfaces, channel cutting, etc.), geology (such as faults, folds, etc.) and searching for marker beds.

CONE (Mullarkey, 1986; Mullarkey and Fennes, 1986) is a KBS that interprets raw data from the cone penetrometer (CPT) in order to perform an input and validity check of the raw data, classification of the soil types (including the profiling of layers) and inference of design parameters with respect to the shear strength of sands and clays.

The rule-based approach used in SITECHAR (Rehak et al., 1985) was further developed as LOGS (Lok, 1987, Adams et al., 1989). LOGS treats information from several boring logs and provides the user with two dimensional subsurface profiles. The system tries to identify marker beds, lenses (wedge-shaped deposits) and faults (areas with boundaries within the confines of the site).
The approach used by Carpaneto and Cremonini (1991) for geotechnical site characterisation uses the concept of a 'site pattern' i.e. a simplified general soil profile. The depths and field description of soil layers, results of laboratory and insitu penetration tests, are compared against the 'site pattern' and a measure of certainty calculated.

SAGITARE (Vergobbia et al., 1992) can be used to merge data from soil descriptions, classification data from laboratory testing and results from insitu tests to form a final borehole log. It has knowledge bases for identification of soil types (based on the Unified Soil Classification Scheme) and for processing results from the Cone Penetration Test (CPT). Having identified layering from these different sources, it then tries to simplify the log by eliminating insignificant beds and attempts to identify the major formations.

Another system which uses the integration of different types of data is described by Kovalevsky & Kharchenko (1992). Their system is used for classifying seabed soils based on an integration of geophysical and geotechnical data, e.g. compressional- and shear-wave velocity sections and borehole profiles.

Toll et al. (1992), Toll (1994), Toll (1995) describe SIGMA, a KBS for interpreting ground conditions from borehole logs. It can also assist with the derivation of design parameters from laboratory or field test results. The approach used for correlating soil layers between boreholes (based on soil descriptions) is described by Vaptivas & Toll (1993). A similar approach was also used by Oliphant et al. (1996). Their system (ASSIST) can also generate graphical representations of the ground conditions.

Kinnicut et al. (1994) describe a system called NOMAD which can be used for three dimensional stratigraphic characterisation. NOMAD can use the functionality of KRHS (Kinnicut, 1995) to create ground profiles from borehole data. This is done by combining geostatistical and knowledge-based approaches. The geostatistical interpretation can be combined with subjective data entered by the user. Inazaki (1994) combines a dynamic depth warping algorithm for borehole correlation with a KBS for combining layers into geotechnical zones.

Adams & Bosscher (1995) describe the integration of geographical information systems (GIS) and knowledge-based systems for subsurface characterization. Thomaz & Altschaedl (1994), in their sketchy outline of GeoSYS, also suggest that a combination of tools is necessary to support the site investigation process. These developments are the logical extension of the idea of a 'geotechnical site characterisation workbench' suggested by Rahak et al. (1985).

Al-Giarr (1995) describes the use of a KBS in the desk study phase of a site investigation. This system uses data from aerial and space imagery to assist in evaluating potential sites. The KBS uses features abstracted from the images (topography, drainage, gullies, tone, land cover and land use) to assess a site.

SITECLAS (Wong et al., 1989) is a KBS used to classify a site (according to the Australian Standard AS2870.1) rather than to generate an interpretation of the ground conditions. The site is identified as one of six classes based on information about the soils found at the site (using Australian soil groups), presence of fill, footing type, whether the site is subjected to landslip, erosion, collapse etc.

The methods so far described for analysis and interpretation of geotechnical site investigation data make use of either geometrical reasoning or statistical techniques. Zhou & Wu (1994) describe the use of neural networks for this purpose. Their neural network system is used to characterize the spatial distribution of rockhead elevations. Similar applications relevant to ground water characterisation are described by Rizzo & Dougherty (1994) and Basheer et al. (1996). Basheer et al. describe how neural networks can be used to map the variation of permeability in order to identify boundaries of a landfill.

4. DISCUSSION

The application of artificial intelligence techniques in geotechnical engineering as a whole is still relatively new. Many systems are only simple developmental prototypes, with much more work needed before they would prove useful in geotechnical practice. Nevertheless, they do demonstrate the potential of both knowledge-based systems and neural networks to this field.

Systems for site investigation planning that have been developed to date are exclusively knowledge-based systems. There are examples of systems to
consider different site investigation strategies. The aim is normally to reduce the risk involved in investigating subsurface conditions to an acceptable level. Recommendations are made about numbers of boreholes or trial pits, amount of testing etc. Other systems can be used to highlight omissions in the way that an investigation has been carried out. Some systems can assist in decisions about the most appropriate type of in situ testing.

Systems for interpretation of ground conditions are largely knowledge-based systems although a small number of neural network approaches have been developed. The main aim of most systems is to generate a model of the ground conditions based on information from borehole records. Both geometrical reasoning and statistical techniques have been used. Some systems only attempt to generate a simplified general soil profile whereas others involve three-dimensional stratigraphic characterisation.

There is a growing trend for site investigation data to be generated and/or stored in digital form. Advances in instrumentation have resulted in much larger quantities of data being generated than was the case in the past. A further development (which is already underway) will be the adoption of surface modelling packages such as Geographical Information Systems (GIS) within geotechnical engineering. Since more data is available, more useful data processing options will be needed. Therefore, artificial intelligence systems that can assist with data processing will become more important, if not vital, to future generations of geotechnical engineers.

Hybrid systems which combine different artificial intelligence techniques and other techniques (eg database systems, Geographical Information Systems) will probably prove to be the most useful type of system for site characterisation. In this way the strengths of each computing technique are harnessed to their best advantage. Some examples of these combined systems are now emerging, and they form a useful model for future developments.

6. ACKNOWLEDGMENTS

In compiling this paper I have drawn heavily on an earlier review by Moulou, Toll & Vagias (1995). I acknowledge the work of Dr Marina Moula and Dr Nikitas Vagias in producing that paper.

7. REFERENCES

Adams T.M., Christiano P. & Hendrickson C.


Site characterization for a land reclamation project at Changi in Singapore

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ABSTRACT: The Changi East reclamation project in Singapore consists of 1500 hectares of land reclamation over the Singapore marine clay and soil improvement works for the future expansion of Changi International Airport. An extensive site characterization program was implemented to determine the physical, strength and consolidation parameters of the foundation soil for land reclamation control and to evaluate the feasibility of the proposed soil improvement method. A program combining laboratory testing and in-situ testing has been found to be effective and useful for site characterization in land reclamation projects.

1 INTRODUCTION

Reclamation of foreshore on compressible marine clay foundation often requires extensive site investigation to characterize the foundation soil. The consolidation properties of foundation soils are needed prior to land reclamation activities in order to predict the magnitude and rates of settlement with the expected fill load and future service load. These properties are also needed for the design of soil improvement works. The shear strength properties are required for foundation slope stability analyses during reclamation and for the short and long term stability analyses of shore protection works.

Figure 1 shows the site plan of the Changi East reclamation project in the Eastern part of Singapore. The reclamation work involves primarily the hydraulic placement of 7 to 16 meters of sandfill over the sealed existing of the Singapore marine clay which is as thick as 40 meters in certain areas. A preliminary survey inclusive of a desk study of existing data and geophysical seismic reflection surveys was carried out prior to the detailed soil investigation. Based on the geophysical survey results, the soil investigation program was planned.

Extensive soil investigation works consisting of marine boreholes, in-situ tests and laboratory tests were subsequently carried out. Undisturbed samples taken from boreholes were tested in the laboratory to determine the physical, mineralogical, strength and consolidation characteristics of the marine clay. The in-situ tests included the field vane shear test, cone penetration test, dilatometer test and self-boring pressuremeter test.

Two main objectives of this paper are: (1) to describe the planning and use of various site investigation techniques for the characterization of foundation soils and (2) to present a summary of comparison between soil parameters measured from the various in-situ, laboratory tests and direct field measurements.

2 PRELIMINARY INVESTIGATIONS

A desk study was first conducted on existing marine boreholes which had been carried out earlier by other parties and site profiles and geology of the underlying soil were then derived. Additional investigations for clarifying the stratigraphy, geology and the soil characteristics and stress history of the soils were found necessary.

Marine bathymetric surveys together with marine geophysical seismic reflection surveys of the project area was carried out with the use of a water surfaced towed boomer profiling system. The elevations of the bases of the compressible layers and the distribution of soft marine clay pockets deposited in submarine valley cuts were determined.
3.3 Laboratory testing

Conventional oedometer, UU triaxial tests and specialized laboratory tests such as the CKoU triaxial test and Rowe consolidation test were conducted on undisturbed soil samples retrieved from the site. The objective of these tests was to establish the characteristics of the foundation soil and to obtain the soil parameters needed in the design.

The results obtained from these laboratory tests were compared with those interpreted from the in-situ tests.

4 GEOLOGICAL PROFILE

The preliminary site investigation and geophysical survey revealed that the Singapore marine clay at Changi consists of two marine members locally known as the upper and the lower marine clays. These soft to medium stiff clay members are recent deposits of estuarine origin. The upper and lower marine clays are separated by a layer of medium stiff to stiff clay 2 to 5 meters in thickness. This layer, is reddish in color and is believed to be the desiccated crust of the lower marine clay resulting from the exposure of the seabed to the atmosphere during the rise and fall of the sea levels in the geological past. At some localities, sand layers which are alluvial deposits laid at river mouths during the pause in marine deposition, were found instead of the desiccated clay. The lower marine clay in turn is underlain by the old alluvium comprising of a cemented silty clayey sand.

The marine clay deposits found in the northern part of the project site are significantly thicker than that in the southern part. The strength of the clay at seabed differs across the site in that the material was moderately overconsolidated in the southern part and normally to lightly overconsolidated in the northern part.

The high overconsolidation ratio (OCR) of the southern half of the project area could be explained by considering a postulated geological history. Based on a seismic survey carried out in 1993, it was found that there was a discontinuity of buried channel in the southern part of the site which was possibly an old river mouth at which sand deposits have laid over the lower marine clay during the pause in marine deposition. In the 1970's, during the initial phases of reclamation for the existing Singapore Changi Airport, the sand deposits originally overlying the marine clay were extracted for the reclamation. As a...
result, the seabed marine clay in the southern part appears to be overconsolidated. The age of the existing marine clay in the southern part of the project is believed to be the same as lower marine clay in the area.

The typical geological profile and soil conditions of the project site are shown in Figure 2.

5 PHYSICAL AND MINERALOGICAL CHARACTERISTICS

The typical soil profile, strength and consolidation parameters of the marine clay found in Changi is shown in Figure 3. The marine clay can be described as high to very high plasticity silty clay, except for the intermediate layer as shown in the plastic chart in Figure 4. From clay mineralogy tests using both the X-ray diffraction and the electron microscope methods, the clay mineral assemblages were found to be dominated by kaolinite with smectite, mica and chlorite are present in minor amounts.

6 STRENGTH CHARACTERISTICS

6.1 Shear Strength

The undrained shear strength of the marine clay was investigated at site using the FVT, the CPT, DMT, and SBPT.

From the FVT the empirical correlations of the shear strengths and the normalized shear strength ratios of the marine clay was found to be different for the northern and southern parts of the project as shown in Table 1. The results indicate different stress histories of the clay. The much higher normalized shear strength ratio for the marine clay in the southern part confirms the higher overconsolidation ratio of the clay.

By referencing in-situ test measurements to the undrained shear strengths as determined from the FVTs, the following empirical correlations between the undrained shear strength and measurements from CPT, DMT and SBPT respectively for the Singapore marine clay were established.

\[ CPT: s_u = \frac{(q - d - N_k)}{N_k} \]  \hspace{1cm} (1)

where \( N_k = 23.8 - (0.263 L_3) \)

\[ DMT: s_u = 0.22 \sigma_{00} (0.5 K_o) \]  \hspace{1cm} (2)

where \( K_o = (P_c - u_0) / \sigma_{00} \)

In the authors’ assessment, the value of \( m \) is 1 for upper marine and intermediate clays and 0.7 for lower marine clay.

\[ SBPT: s_u = \frac{(P_L - \sigma_{00})}{N_p} \]  \hspace{1cm} (3)

![Figure 2: Geological profile at the project site.](image)
where $N_p = 1 + \log_6 (C / c_v)$, $P_i$ is limit pressure, $c_{vis}$ is total horizontal stress, $G$ is shear modulus and $N_p$ is the pressuremeter constant defined by Marsland and Randolph (1977). The values of $N_p$ are 6.6, 6.4 and 7.2 for the upper marine clay, the intermediate clay and the lower marine clay, respectively.

A comparison of the values of undrained shear strength of the marine clay as interpreted from in-situ tests using the established correlations with those determined from triaxial $C_{kol}$ tests is shown in Figure 5.

7 CONSOLIDATION CHARACTERISTICS

7.1 Stress history of clay

The stress history or the OCR of the marine clay was evaluated using both the in-situ and laboratory tests.

Results of the FVTs, CPTs, DMTs and SBPTs were used to estimate the OCR values.

The estimation of OCR from the CPT penetration test was based on the net corrected cone resistance normalized by the overburden pressures. The estimation of OCR from the DMT test used the cone resistance. From the SBPT, the OCR is related to the normalized undrained shear strength calculated from SBPT test. The estimation of OCR from the FVT was based on the normalized undrained shear strength and the plasticity index.
The one-dimensional oedometer test was used to determine directly the OCR of the clay in the laboratory.

By referencing to the OCR, as obtained from oedometer tests the following empirical correlations between the OCR and measurements from CPT, SBPT, DMT and FVT respectively for the Singapore marine clay are established:

CPT: \[ \text{OCR} = \alpha \left( q_t - \sigma'_{v0} \right) / \sigma'_{vo} \] (4)

Where \( \alpha \) is a constant and is of value 0.32 for the Singapore Marine Clay at Changi.

DMT: \[ \text{OCR} = (0.5 \text{ K}_d) \] (5)

where \( \text{K}_d = (P_s - u_r) / \sigma'_{vo} \)

\( a \) is 1.0 for upper and lower marine clays and 0.8 for intermediate clay, compared to a \( a \) of 0.84 as reported by Chang (1991) for on land Singapore marine clays.

SBPT: \[ \text{OCR} = \log\left( s_u / \sigma'_{vo} \right) / 4 \] (6)

Where \( s_u \) is the undrained shear strength obtained from the self-boring pressuremeter test.

FVT: \[ \text{OCR} = \log\left( s_u / \sigma'_{vo} \right) * (1 / 0.25) / 0.8 \] (7)

Where \( s_u \) is the undrained shear strength obtained from the field vane shear test.

A comparison of the OCR values estimated from various in-situ tests using the established correlations with those from oedometer tests is shown in Figure 6.

7.2 Coefficients of consolidation

Disipation tests using the cone penetrometer, the dilatometer, the self-boring pressuremeter and the BAT permeameter were carried out to determine the coefficient of consolidation for horizontal flow. Several one-dimensional consolidation tests were also carried out with the Rowe cell on specimens which measures 75.7 mm in diameter and 30 mm in height for determining the coefficient of consolidation for horizontal flow (Cv) in the laboratory.

The coefficient of consolidation for vertical flow (Cv) of the marine clay was determined by one-dimensional oedometer tests the results of which are tabulated in Table 2 for these distinctive clay layers. The coefficients of consolidation for horizontal flow (Cv) as obtained from various in-situ tests and of laboratory tests are shown in Figure 7. The Cv value obtained from vertical flow (Cv) as obtained from oedometer tests are also shown in Figure 7. The Cv value obtained from laboratory is about 0.5 m\(^2\)/year.

The Cv values measured from the Rowe cell is comparable to that measured from the oedometer. The Cv values as interpreted from in-situ tests are higher than the laboratory measured values. Among the in-situ tests the Cv values from SBPT’s are the highest.

7.3 Primary and secondary compression indices

The primary compression and secondary compression characteristics of the marine clay were determined from oedometer tests and results are also shown in Table 2 for the three clay layers.

The upper marine clay with an average compression index (Cv) of 1.0 is found to be more compressible than the lower marine clay. The average Cv value of the lower marine clay is in the order of 0.5. The intermediate stiff clay has a Cv value between 0.2 to 0.3.

Both the upper and the lower marine clay are lightly overconsolidated by nature while intermediate clay is moderately overconsolidated due to desiccation. The secondary compression index of the upper and for the lower marine clay averages 0.019 and 0.018 respectively. The Cv/Cv ratio averages 0.019 for the upper and 0.025 for the lower marine clay respectively. The intermediate clay has a very low secondary compression index averaging 0.0137 and has a Cv/Cv ratio averaging 0.0613.
Table 2. Range of physical and consolidation characteristics of Singapore Marine Clay at Changi.

<table>
<thead>
<tr>
<th></th>
<th>UMC</th>
<th>ISC</th>
<th>LMC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Density (kN/m³)</td>
<td>14.23</td>
<td>18.64</td>
<td>15.7</td>
</tr>
<tr>
<td>W.C. (%)</td>
<td>70.88</td>
<td>10-35</td>
<td>48-60</td>
</tr>
<tr>
<td>LL (%)</td>
<td>80-95</td>
<td>50</td>
<td>65-90</td>
</tr>
<tr>
<td>PL (%)</td>
<td>20-28</td>
<td>18-20</td>
<td>20-30</td>
</tr>
<tr>
<td>Cb</td>
<td>1.8-2</td>
<td>0.7-0.9</td>
<td>1.1-1.5</td>
</tr>
<tr>
<td>G</td>
<td>2.6-2.72</td>
<td>2.68-2.76</td>
<td>2.72-2.75</td>
</tr>
<tr>
<td>Cg</td>
<td>0.6-1.5</td>
<td>0.2-0.3</td>
<td>0.6-1.0</td>
</tr>
<tr>
<td>Cc</td>
<td>0.012-0.025</td>
<td>0.003-0.023</td>
<td>0.012-0.028</td>
</tr>
<tr>
<td>Cc/Cg</td>
<td>0.008-0.042</td>
<td>0.0076-0.015</td>
<td>0.012-0.038</td>
</tr>
<tr>
<td>Cc</td>
<td>0.09-0.16</td>
<td>0.08-0.15</td>
<td>0.14-0.2</td>
</tr>
<tr>
<td>Cc (m/yr)</td>
<td>0.47-0.6</td>
<td>1.45</td>
<td>0.8-1.5</td>
</tr>
<tr>
<td>Cc (m/yr)</td>
<td>3.7</td>
<td>10-30</td>
<td>4-10</td>
</tr>
<tr>
<td>Cc (m/yr)</td>
<td>2.3</td>
<td>5-10</td>
<td>3-5</td>
</tr>
<tr>
<td>OCR</td>
<td>1.5-2.5</td>
<td>1.4</td>
<td>2</td>
</tr>
</tbody>
</table>

*Note: UMC=Upper Marine Clay, ISC=Intermediate Silt Clay, LMC=Lower Marine Clay, Cc=C for Reconspression.

Typical oedometer test results and a family of void ratio versus effective stress curves can be found in Figure 8 for the marine clay layers at the site.

8 CONCLUSIONS

A site investigation program consisting of survey, direct boring and sampling, laboratory testing and in-situ testing has been found to be effective and useful for site characterization for the Changi East reclamation project in Singapore.

9 ACKNOWLEDGEMENT

The authors would like to express their thanks to Win Maw of SPECs Consultants for his help in computer graphical works and report preparation.

10 REFERENCES


Public Works Department (Singapore) 1996. The Geology of Singapore.
Drilling and sampling
Application of horizontal directional drilling for contaminated site characterization

Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Alta., Canada

ABSTRACT: Traditionally, sampling and monitoring of potentially contaminated soil and ground water has been achieved using vertical drilling technology. However, vertical drilling presents several technical limitations including the need to position the drilling rig directly above the target zone and the risk of cross contaminating laterally stacked aquifers. These limitations may be overcome using horizontal directional drilling technology.

This paper gives a short introduction to horizontal directional drilling equipment and installations techniques, followed by an overview of existing horizontal sampling tools. A proposal for the development of a horizontal multiple port soil sampler is outlined. The future role of horizontal sampling and horizontal logging in site investigation is discussed. It is concluded that horizontal directional drilling technology can find many applications in site investigation, offering an efficient and cost-effective method of collecting geotechnical and geo-environmental data, both as an alternative and as a complement to vertical drilling and sampling technologies.

1. INTRODUCTION

Horizontal directional drilling, an emerging industry since the early 1980’s, is currently associated primarily with the oil, gas and utilities industries. In recent years, horizontal drilling technology has been used in the remediation of contaminated ground water and soils (Alouche et al., 1997). A new promising application of this powerful technology is in the sampling of contaminated sites.

Traditionally, sampling and monitoring of suspected contaminated soils and ground water has been done using vertical drilling technology. However, vertical drilling presents some technical limitations, as the drilling rig must be located directly above the location of interest. Thus, collection of soil samples below surface/subsurface obstacles, such as rivers, structures or highways, is restricted. In addition, collection of samples in unstable soil conditions or environmentally sensitive areas may be proven difficult and expensive. Finally, vertical drilling is associated with the risk of penetrating impermeable layers, potentially causing cross-contamination between aquifers. Recent developments in horizontal directional drilling provide a means to overcome these limitations.

Section 2 of this paper is devoted to directional drilling technology. Drilling equipment, steering and navigation capabilities as well as boring techniques are outlined. Section 3 provides an overview of existing horizontal sampling tools, which are commercially available in North America. A joint industry/university research and development effort of a new generation of horizontal sampling and characterization tools currently in progress at the University of Alberta is presented in Section 4. The paper is concluded with a discussion of the possible applications of horizontal drilling technology in the characterization of contaminated sites and in other areas of site investigations.

2. HORIZONTAL DIRECTIONAL DRILLING - TECHNOLOGY AND TECHNIQUES

Horizontal directional drilling (HDD) makes it feasible to drill horizontal boreholes of 50 mm (2 in.) to 1220 mm (48 in.) in diameters beneath
surface/subsurface obstacles, over extended horizontal distances (Kamloops, 1996). HDD is used in the oil, gas and utility industries for the installation of pipelines and conduits, and more recently, for the remediation and sampling of contaminated soils and ground water. Other applications of horizontal drilling include de-watering, pipe-seaming, cathodic protection, and slope stabilization.

In contrast to conventional vertical drilling rigs where the self-weight of the drilling assembly provides the downward force on the bit, in horizontal directional drilling, the load on the bit is provided solely by the drilling rig. Typical HDD rigs use a chain drive, rack and pinion drive or a hydraulic cylinder to push or pull a carriage to advance and retract the drill string as illustrated in Figure 1. A carriage slides forward on a frame pushing the pipe into the borehole. The carriage is then retracted, and a new pipe segment is attached. HDD rigs can be classified as mini, midi or maxi rigs, depending on their thrust/pull-back and torque capacities, as shown in Table 1. Regardless of their class, most drilling rigs operate in a manner similar to that described above.

![Figure 1: Horizontal drill rig](image)

During the boring process drilling fluid is injected under pressure ahead of the advancing bit. The drilling fluid is used to reduce friction and stabilize the borehole. While the use of drilling fluid extends the boring distance of the drilling rig, avoiding their use may be a preferred alternative in the case of horizontal sampling. The bent-sub assembly is located at the front of the drill string, has a larger diameter than the drill string, and is at an angle to it. To bore a straight hole the drilling string is rotated while pushed forward. When a change in direction is desired, rotation stops and the bent-sub assembly is preferentially oriented in the borehole and is then pushed by the drill rig. As the slant on the face of the wedge is pushed against the soil, the entire assembly is deflected in the desired direction. After the steering correction is completed, rotation is resumed until another correction is required.

During the drilling process the bore path is traced by interpretation of signals sent by electronic sensors located near the drill head. At any stage along the drilling path the operator receives information regarding the position, depth and orientation of the drilling tool, allowing him to navigate the drill head to its target.

A horizontal borehole is defined using three parameters, an approach angle, radius of curvature and step-off distance. The approach angle is the angle between the drill stem and the ground surface at the entry point. Depending on the type of rig used this angle may vary from 7° to 90° to the horizontal. An angle of less than 30° is most common. The deeper the borehole, the larger the approach angle.

The radius of curvature defines the curved portion of the borehole. The greater the borehole radius of curvature the greater the total borehole length. The benefit of a higher radius of curvature (i.e. reduced stress on drill string and well product) must be balanced against the extra cost of drilling a longer borehole.

<table>
<thead>
<tr>
<th>Specifications</th>
<th>Mini Rig</th>
<th>Midi Rig</th>
<th>Maxi Rig</th>
</tr>
</thead>
<tbody>
<tr>
<td>Throat/Pullback</td>
<td>&lt; 20,000 lbs.</td>
<td>20,000 - 80,000 lbs.</td>
<td>&gt; 80,000 lbs.</td>
</tr>
<tr>
<td>Maximum Torque</td>
<td>&lt; 2000 ft. lbs.</td>
<td>2000 - 20,000 ft. lbs.</td>
<td>&gt; 20,000 ft. lbs.</td>
</tr>
<tr>
<td>Drilling Speed</td>
<td>&gt; 120 RPM</td>
<td>110 - 190 RPM</td>
<td>&lt; 190 RPM</td>
</tr>
<tr>
<td>Carriage Speed</td>
<td>&gt; 100 ft/min.</td>
<td>90 - 100 ft/min.</td>
<td>&lt; 90 ft/min.</td>
</tr>
<tr>
<td>Carriage Drive</td>
<td>Chain or Chain</td>
<td>Chain or Rack &amp; Pinion</td>
<td>Rack &amp; Pinion</td>
</tr>
<tr>
<td>Drill Pipe Length</td>
<td>5-10 ft.</td>
<td>10 - 30 ft.</td>
<td>30 - 40 ft.</td>
</tr>
<tr>
<td>Drilling Distance</td>
<td>&lt; 200 ft.</td>
<td>200 - 600 ft.</td>
<td>&gt; 600 ft.</td>
</tr>
<tr>
<td>Power Source</td>
<td>&lt; 150 HP</td>
<td>150 - 250 HP</td>
<td>&gt; 250 HP</td>
</tr>
</tbody>
</table>

Table 1: Specifications of Directional Drilling Rigs (After May, 1994)
The step-off distance is the horizontal distance between the entry hole and the beginning of the horizontal section of the borehole. This is usually determined by the open area available for setting up the drilling equipment. The selection of any two of these variables determines the third.

The bore is launched from the surface and the pilot bore proceeds downward at an angle until the necessary depth is reached. Then the path of the bore is gradually brought to the horizontal, and the bore head is steered to the designated exit point where it is brought to the surface by following a curved path. A directional monitoring device, located near the head of the drilling string, is used to track the position of the drilling head. After the pilot string breaks the surface at the exit location, the bit is removed from the drill string and replaced with a back-reamer. The pilot hole is then back-reamed, enlarging the hole to the desired diameter while simultaneously pulling back the well product behind the reamer. The above description is typically referred to as a "continuous" borehole. In some instances it is desired to drill a single-ended borehole, commonly named a blind borehole. Here construction, reaming and all other activities take place at the location of the entry pit. Blind boreholes are used only in environmental applications for either collecting soil samples or installing horizontal wells beneath structures.

Accuracy of tracking the drilling head varies according to the method and type of equipment used. Typically, depth and position readings can be assumed to be accurate within ±5% in terms of the drilling head true depth (Digital Control Inc. 1997), however, tracking systems which allow accuracy of ±2% in both plan and profile, regardless of the borehole's depth, are available (Shawwell, 1996). Current commercial HDD equipment can operate in a wide range of soil conditions, from extremely soft soils to full face rock formations with unconfined compressive strengths of 21 MPa (30,000 psi).

3. HORIZONTAL SAMPLING

Horizontal drilling technology provides the ability to recover undisturbed, high quality, samples from areas that cannot be reached using vertical drilling technology, such as beneath structures as illustrated in Figure 2. Multiple target points, at different depths, distances and directions can be collected without a need to reset the rig. Also, this highly automated, remote access drilling technique offers an elevated safety level to field personnel since exposure to contaminants may be dramatically reduced (Langseth, 1980). Finally, the risk of penetrating impermeable layers, potentially increasing the extent of contamination is significantly reduced.

3.1 Eastman Soil Sampler

Perhaps the first to develop a sampler to be used in conjunction with horizontal directional drilling rigs for the purpose of characterizing contaminated sites was Eastman Chemical Environmental systems in the early 1990’s (Karlsson, 1993). The sampler, designated PunchMaster 2000® shown in Figure 3, is capable of cutting a two inch diameter by five feet long undisturbed sample from soft to medium soils. It can be used for vertical and horizontal sampling through a minimum 100' radius curve in a 5'-6" diameter borehole. The sampler works on a principle similar to a split-spoon or a Shelby Tube core sampler. First a "blind" borehole is drilled up to the target area. The drill string is then withdrawn from
the borehole and the boring head is replaced with the sampling tool. The PunchMaster 2000™ is advanced into the borehole to the target area while the load on the outer tube is kept constant with an applied hydraulic pressure. At a pre-determined location an inner tube is accelerated into the formation by hydraulic pressure. The sample is then drawn back into the outer tube while pressure on the outer tube is maintained to prevent drilling muds from contaminating the sample, and the PunchMaster 2000™ is brought to the surface. This process is repeated for each sample. PunchMaster 2000™ is suitable primarily for relatively large ("maxi") drilling rigs.

3.2 Ditch Witch Soil Sampler

In the mid 1990’s Ditch Witch™ developed a core sampler significantly smaller and lighter than the PunchMaster 2000™ and capable of use with medium and small sized drill rigs typically employed in the utility installation industry. The sampling process is as follows: the drilling head is navigated below ground to a distance of 1 to 2 ft from the target area, the drill string is then retracted, the cutting head removed, and a soil sampler is connected to the end of the drill string. The sampler is pushed through the bore with the face of the sampler blocked by a piston. At the sample location, the drill string is retracted approximately 18°, and the sampler tube is locked in an open position. The sampler is pushed forward 1 to 2 ft, filling the tube with soil. This device can be modified for use with most Jet-Trac® and some Navigator™ horizontal boring units.

Aside from core samplers, several other devices were developed for use with horizontal drilling rigs, as engineers and designers expanded the boundaries of sampling technology. An example is SEAMIST™, an evening membrane fluid-sampling system developed by Eastman Cherrington Environmental for the US Department of Energy (Eastman Cherrington, 1992). The SEAMIST system can be used to carry instruments (i.e. sensor) in and out of a wellbore or collect fluid samples from a horizontal or vertical borehole quickly and inexpensively.

Another promising application is the use of geophysical methods in conjunction with horizontal boreholes. By drilling a horizontal borehole through the alignment of a proposed underground excavation, and examining the strata around the borehole’s wall using geophysical logging devices, it is possible to characterize the entire excavation alignment using a single borehole. Geophysical methods may include Magnetic Induction (EM), Magnetic Susceptibility (MS) and Ground Penetration Radar (GPR). The data collected can greatly aid the planning and risk/opportunity assessment of underground excavations.

In a recent project, 10:1 Solution Group, a Colorado based company, used geophysical logging technology together with a directional boring technology to identify the location of steel tie-back cables that might have been encountered during boring of a new sewer alignment (Miller, 1997). Other applications of horizontal boring include mineral exploration and continuous geochanical sampling and characterization of contaminated sites.

4. A NEW GENERATION OF HORIZONTAL CHARACTERIZATION TOOLS

Soil characterization and sampling technologies described in Section 3 provide new tools for engineers, designers and planners for collecting subsurface data. Nevertheless, these tools have their disadvantages. For example, in order to collect a core sample using either the PunchMaster 2000™ or the DitchWitch™ soil sampler the drilling string...
must be withdrawn twice for each sample. First to remove the drilling head and attached the sampler, and then to recover the sampler and reattach the drilling head in order to drill to the next target location. In addition, the sampling tool needs to be de-contaminated between successive samples to avoid cross-contamination. This repetitive process is time-consuming and consequently expensive. In order to minimize their risk many contractors will not quote cost per sample but rather charge a flat rate which ranges between $4,000 to $8,000 per day (Wilson and Layne, 1995).

In this section, a new approach to horizontal sampling, which takes advantage of the special features of horizontal directional drilling is presented. The proposed horizontal sampler will be pulled-back through a completed borehole rather than pushed ahead of the drilling head, as is the case with vertical drilling. Furthermore, instead of collecting a single sample the device will be capable of collecting multiple samples from the borehole's wall at various locations along the borehole's alignment. Eliminating the need to re-enter the borehole and withdraw the drill string for each sample collected will result in significant reduction of sampling costs.

4.1 Multiple-Port Soil Sampler

The multiple-port soil sampler illustrated in Figure 4 is designed to be pulled-back through a continuous borehole, collecting samples from the borehole's wall at pre-determined locations. A horizontal drilling rig will be used to drill down to the target area using normal drilling procedures. Then the drilling fluid supply will be shut down and a "dry" boring procedure will be conducted across the target area. Finally, normal drilling procedures will be used when boring up towards the exit location. The drill head will then be removed, the multiple sampler placed on the end of the drill string, and the assembly will be pulled back through the borehole, stopping at predetermined locations to take soil samples from the bore's wall.

The design of the sampler will include a 19 cm (7½") diameter, 102 cm (40") long outer casing in which the sampling mechanism will be placed. A number of 3½" diameter, 8" long sampling tubes will be used to take individual samples. The sampling tubes will be arranged in a carriage which will rotate into a position where they will be pushed from the sampler at an angle and into the formation.

The multiple-port soil sampler is currently under development at the University of Alberta as a part of a joint University/Industry research project with SubTerra Corporation, an Edmonton based directional drilling contractor. A prototype is scheduled to be completed by the end of fall 1997, and initial testing of the new system will be conducted shortly after.

5. DISCUSSION

Horizontal sampling and horizontal logging technology presents several advantages over the traditional vertical sampling and down-hole geophysical logging methods. The greatest advantages are the ability to reach otherwise inaccessible target areas, remote access to hazardous, or environmentally sensitive areas, and the ability to characterize soil profiles horizontally for horizontally oriented projects such as tunnels. Most of the tools developed to date for horizontal sampling were adopted directly from the vertical drilling industry. As a result, samples are taken ahead of the cutting head. The use of this approach together with current horizontal directional drilling technology results in a tedious and expensive sampling process. A new generation of horizontal sampling tools taking advantage of the features of horizontal drilling equipment and installation methods, can lead to significant reduction in the cost of horizontal sampling.

The proposed multiple-port soil sampler can reduce the time required to collect a soil sample by as much as 70%, while the use of geophysical tools with horizontal drilling can provide significant
information along a horizontal line using a single borehole.

Horizontal sampling presents a new dimension in site investigation, offering new tools and added flexibility for collecting geotechnical and geo-environmental data regarding subsurface conditions or contaminant concentrations. In some applications, horizontal sampling presents an economical alternative to vertical drilling methods, while in others data collected from horizontal boreholes may be used to complement data collected from vertical boreholes, providing a more comprehensive description of subsurface conditions.

During our technical review and development, the idea of horizontal sampling generated significant interest among potential users, both contractors and owners. Therefore, it is expected that as geotechnical professionals become aware of the capabilities of sampling and geophysical logging technologies, we will see increased use of these technologies in all aspects of site investigation.

6. CONCLUSIONS AND RECOMMENDATIONS

1. The need to locate a vertical drilling rig directly above the location of interest restricts the use of this technology to applications where such access exists. Also, in environmental investigations, vertical drilling is associated with the risk of penetrating impermeable layers, potentially causing cross-contamination between aquifers.

2. Many of the limitations detailed in (1) can be overcome using horizontal directional drilling technology.

3. A few horizontal soil sampling devices are currently available. However, adopted directly from the vertical drilling industry, they fail to take advantage over the special features of horizontal directional drilling technology. Consequently, the use of these tools is time consuming and rather expensive.

4. A new generation of soil samplers may significantly reduce the cost of collecting geotechnical and geo-environmental data using horizontal boreholes.

5. Another promising application is the adoption of geophysical methods to be used in conjunction with horizontal directional drilling technology.

6. Horizontal site characterization tools present a new exciting dimension in the field of site investigation, and new applications for this technology are likely to arise as geotechnical and geo-environmental professionals gain insight into this powerful technology.

REFERENCES


On the use of drilling parameters in rock foundations

Paulo Colosimo
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ABSTRACT: The Author discusses the advantages of the use of instrumented non-coring drills for rock geological characterization and for controlling the consolidation of grouted rocks. The drill machine employed was a crawler drill, operating with a tricone bit and with a down-the-hole percussive hammer. The perforation parameters were recorded and processed by the Pa.Pe.Ro. system. The instrumented drills (1530) was calibrated on low number of core-drills (10), seismical sounding and 3257 (5576 m) grouting boring with controlled grouting absorption executed for the design and construction of three viaducts of the new Taranto railway, in Italy. Rocks were represented by a calcarenite lying on a limestone, both affected by karst erosion.

1 INTRODUCTION

In geotechnical exploration the objective is to recover core samples either using standard (“diamond”) rotary drilling techniques or using special core barrels for undisturbed samples. In each cases the priority is not to obtain a high rate of penetration but rather to recover quality samples with minimum cost recovery. Besides, the core drilling process itself dictates low rate of penetration because of the time required for the following operations: a) round-trips of the end string, discharging of the core barrels and placing of samples in the core boxes; b) clearing of the hole by circulation on bottom before continuing to core; c) setting of casing across unstable formations, since only clean water can be used, being mud forbidden.

As an intermediate response to these issues, instrumented SPT drills were tested. Combining geological information was proposed (Barr and Brown, 1978, 1983; Brady and Brown 1994).

The definitive response would use instrumented heavy duty rotary down-the-hole percussive techniques leading to lower cost per meter drilled and short drilling times; these advantages increasing with depth.

The following paragraphs illustrate the results obtained using such instrumented drilling procedures, without resorting to the design and construction of three viaducts carrying the new 86 km-long Bari–Taranto line of the Italian State Railways, in Puglia region (Italy).

2 THE TEST SITES

A schematic geological section of a tract of the new Bari–Taranto Railway is illustrated in Figure 1. Deep canyons (“gravines”) due to extreme karst phenomena are located along faults. The famous Castellammare's Caves are only 20 km away. The viaducts and bridges rest on the following formations, from surface:

a) Gravina Calcareous (Pleistocene): soft and medium hard calcarenite, unweathered, irregularly fractured and faulted, with caves, filled or not by residual clay (“terra rossa”); the thickness varies from 6 to 20 m; it is absent in the middle and the lower parts of the “gravines”;

b) Murgia Limestone (Cretaceous): hard arenaceous limestone, in layers 0.03 – 0.3 m thick; strongly faulted and jointed, with fissures and cavern filled or not by “terra rossa”, that are present especially at the unconformity below the calcarenite.

Instrumented drill rigs were employed at all the foundation sites of the three viaducts. In the first two (Gravina Minor of Castellana bridge and S. Lucia bridge, on the Gravina Maggiore of Castellana) for the following purposes:

a) in trial sites, to optimize drilling work, to calibrate the system and establish correlation between drilling parameters and rocks characteristics; D.T.H. percussive drilling was also calibrated with rotary tricone bit, both being calibrated with core-drill;

b) to complete the geotechnical site characterization made by traditional methods (surface investigations, core drilling, P.Q.D. measures, compression tests on samples, Lugeon and pressometer tests, surface and down-hole seismical survey) to obtain Rock Mass Rating (Bieniawski, 1988) and to estimate the bearing capacity of foundations, to define the mixture, geometry and pressures of grouting injection for rock consolidation, with special attention to karst fissures and caves;
c) to survey the rock foundations after the grouting.

Figures 2 and 3 show the geological situation and grouting extension below the foundations.

For the Serra Pizzuta viaduct (1750 m, 70 foundations), crossing the Gravina of Palagianello Instrumented without coring was employed in substitution of core-drilling for the design of the foundations and consolidation by injections. The construction has not yet started.

Many troubles were encountered: a) constant loss of circulation and frequent core burning; b) frequent hole instability need to case, cement, and to re-drill; c) interruptions of drilling due to circulation losses greater than the water supply; d) frequent blocking of the core barrel in inclined drillings due to hole instability.

The gross R.O.P. was less than 15 m for a day - work.

3 COMPARISON OF DRILLING PERFORMANCE

3.1 Coring boreholes

For the design of the entire new Bari - Taranto railway more than 450 core drillings were performed by various contractors to depths varying from 25 to 140 m. The core barrel was a double tube swivel type, mounted on a crown of tungsten-carbide steel inserts, with diameters varying from 76 to 100 mm, boring above the water table.

Drilling of the consolidation injections boreholes were performed by seven hydraulic crawler mounted unit (C-8 Italian model), designed to perform both heavy rotary drilling and D.T.H. percussion drilling, for injection boring. One unit was instrumented, and was also used for exploration.

After trials, two bit types were selected:

a) Tricone roller bit, VTH model, 101.6 mm diameter, moderately spaced strong steel teeth with one flank hard
4 MEASUREMENT AND CONTROL

4.1 Standard drilling instrumentation

Hydraulic drill-rigs are equipped as standard with gages measuring various data:

- a) drilling parameters: revolution indicator, pressure gages for measurement of the thrust produced by the drill-head and for measurement of torque; pressure gages for measurement of circulating and/or working fluids;
- b) pressure in the circuits of the services (manometers of booms, raising and lowering, coupling and uncoupling the drill pipes, stabilizers, etc.);
- c) pressure gages and speed indicator of the motor supplying the primary energy to the drill-rig.

When drilling parameters are maintained constant, study of rate of penetration allows the detection of change in lithology and of the rock compactness. This parameter is of paramount importance and needs to be meticulously recorded in order to capture all significant lithological informations. Depth of rotary table drill is equipped, as a standard, with a device for recording the rate of penetration (the "penograph"). In the standard hydraulic drill, accurate penetration measures can be estimated only if pipes are marked at intervals by grease.

The rotation speed, the thrust and the circulating fluid pressure can be read on the gages. Their values are fixed by the operator; variations are due to the pipes tension and rock condition. Torque should vary nearly instantaneously with rock condition; therefore, torque should be recorded continuously.

In percussive drill, the pressure of working fluid producing the hammer impacts can be directly read on gages; the frequency of the impacts depends on this pressure.

4.2 The PaPe.Ro. instrumentation system

The PaPe.Ro. system (= Parameters of Perforation by Rodio t.g.r.) comprises, as shown in Figure 4, in:

1) rig sensors, measuring the standard drilling parameters:
   - rate of penetration;
   - speed of rotation;
   - thrust on the bit;
   - torque;
   - pressure of circulating or working fluid; rig accelerometer; proximity switch; encoder;
   - portable unit of acquisition data Pa.pe.ro., with video - controlling production, an energized digitizer;
   - office or portable computer; software and printer.

Data are energized at every cm of bit advance. When a run has been completed or the rod drills are lifted for flushing, the proximity switch stops the energization.

4.3. Information and results by Pa.pe.ro. system

The system was described by De Paolo et al., 1988, Fortunati F. & Stella C. 1992. The use in controlling the consolidation by grouting is reported by Fortunati, 1996 (for soils), and by Colosimo, 1996 (for rocks). The system produces a log of depth versus R.O.P. (rate of penetration), E_q (specific energy) and I_a (acceleration index).

- Penetration rate (R.O.P.). The Pa.pe.ro. energization are made at every cm of the advancing bit.

Figure 5 shows Pa.pe.ro. records in Murgia limestone. It is possible to interpret the following rock characteristics:

- from 0 to 3 m, R.O.P. = 3 - 8 m/h: highly fractured medium hard rock;
- from 3 to 13 m, R.O.P. = 1,5 - 2,5 m/h: medium hard-rock;
- from 13 to 25, R.O.P. = 2,5 - 3,5 m/h: highly fractured hard rock.

- Specific perforation energy (E_q). The R.O.P. depends on the drilling parameters. As written above, the thrust...
and the rotation speed can be fixed, but the torque varies with rock condition. Therefore the energy spent to produce the advance must be considered a more accurate indicator of the rock condition. The specific energy ($E_{sp}$) can be calculated by the following expression (Teale, 1965):

$$E_{sp} = \frac{F \cdot V \cdot T}{A \cdot B}$$

(1)

where:

- $E_{sp}$ = specific energy, GJ/m³
- $F$ = thrust on the bit, kN
- $V$ = speed of penetration (m/min)
- $T$ = torque, kN•m
- $A$ = area of bit, m²
- $B$ = speed or rate of penetration (R.O.P.), m/s

$F$ represents the work needed to bore a unit volume of rock. The first member of the equation (1) represents the contribution of the thrust (as in the Dutch penetrometer) producing no advance.

The best bit produces the highest speed of penetration at minimum energy, therefore it behaves the drill operator to select the optimum bit type and optimum drilling parameters.

The pressure of circulating fluid was slowly increased with depth in order to clear the bottom hole, to refrigerate and to clean teeth and cones of the bit and stabilize mylonite layers, but not to participate in the excavation of the rock, as is used high fluid pressure in jet roller bits.

The video control in the Papiro system allows to “see” the working parameters. Indeed, it is possible to adjust the pull force on rods are added and to increase the fluid pressure with depth.

The application of the above $E_{sp}$ concept to the D.T.H. drill should consider the contribution of:

- a) the thrust;
- b) the rotational abrasive action of the buttons of the bit;
- c) the impacts of the hammer-bit.

As high rotational speed is also required to lift the cuttings.

The equation (1) can be rewritten in the following form, introducing a third member representing the pure hammering work, without rotation (derived from Wells, 1949, Nox, 1965):

$$E_{sp} = \frac{F \cdot k \cdot V \cdot T + k \cdot E_{DTH}}{A \cdot B}$$

(2)

where, in addition to (1):

- $k$, $k_1$, $k_2$ = empirical factors
- $F$ = mean air pressure in the cylinder, kN/m²
- $S$ = area of the cylinder piston, m²
- $n$ = number of impacts per minute
- $L$ = stroke of the piston, m
- $W$ = weight of the bit, kN

The selected rotation speed (80 rpm) was high, but inferior than that employed for the roller bit (120 rpm). The D.T.H. hammer is more sensitive than roller bit to thrust. If the thrust is too low, the energy is reflected through the pipes as a tension wave; if it is too high no advance is produced and the energy is reflected as a compression wave, with consequent troubles in pipes joints and rig. Indeed, thrust must be adjusted continuously.

In general, the roller bit worked better in the more abrasive calcite-crystal; D.T.H. hammer worked at low costs in the limestone.

Statistical distributions of $E_{sp}$, based on the data from the trial drills are presented in Figure 6, for calcite-cryst.
in Figure 7 those for the Murge limestone of Gravina Minore and in Figure 8 those of Murge limestone of Gravina Maggia.

The following characteristics have been obtained:

a) The energy fields are different for the calcarenite and the limestone: 95.1% of the $E_p$ of the calcarenite are comprised between 219 and 1790 MJ/m³; for the limestone, the values in this field are only 1.3%. The values of 1800 MJ/m³ can be assumed for the very sound calcarenite. Higher values can be produced by harder fissures filled by secondary calcite.

b) In the limestone four field $E_{eq}$ can be recognized:

- from 0 to 1200 MJ/m³: open cavities and fissures ($<500$) or filled by clay, this field is shown in Figure 6;
- from 1200 to 3000 MJ/m³; this field is recognizable in Figure 5 and represents heavy jointed rock and clay;
- from 3000 to 8000 MJ/m³; clay joined limestone ($<5400$) and thick layers of sound rock ($>5400$); more than 8000 MJ/m³: exceptionally hard thick layers.

The $E_p$ characterizes the rock conditions better than R.O.P. In the Figure 6 the medium fractured very hard limestone is characterized by $E_{eq} > 10^5$ MJ/m³ extending from -3 m to -13 m, while by the R.O.P. the lower change is estimated at -12 m.

The Figure 6 shows that coves detected by the R.O.P. was filled by hard soil material (dry "terra rossa" or high broken limestone) because they require a low, but not insignificant, $E_{eq}$ to be bored.

- Acceleration index $J_a$ An accelerometer placed an a stirrup on the top of the hill-rig mast allows to register the vibrations due by drilling, between the frequencies from 40 to 2000 Hz and their power peaks. By trials, the following frequency fields are recognized:

Figure 6. Perforation. DHT percussive drill. From 0 to -3 m: calcarenite; below: limestone. Caves: from -10,80 to -11,40 and from -14,20 to -14,60.

Figure 7. Distribution of $E_p$ in trials drilling in the limestone of S. Lucia bridge. Rotary drilling, tricone bit.

Figure 8. Distribution of $E_p$ in trials drilling in the limestone of Gravina Minore bridge. Rotary drilling, tricone bit.
<table>
<thead>
<tr>
<th>Rock condition</th>
<th>$R_{OP}$</th>
<th>$R_{QD}$</th>
<th>$I_a$</th>
<th>$V_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravina calcarenite</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
<tr>
<td>fissured</td>
<td>$&lt; 0.5$</td>
<td>$&lt; 0.5$</td>
<td>$&lt; 0.5$</td>
<td>$&lt; 0.5$</td>
</tr>
<tr>
<td>fissured, altered</td>
<td>$&lt; 0.5$</td>
<td>$&lt; 0.5$</td>
<td>$&lt; 0.5$</td>
<td>$&lt; 0.5$</td>
</tr>
<tr>
<td>Murgia limestone</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
<tr>
<td>sound, thick layers</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
<tr>
<td>sound, jointed</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
<tr>
<td>heavy jointed</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
<tr>
<td>jointed and clay</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
<tr>
<td>After grouting</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
<tr>
<td>calcarenite ***</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
<tr>
<td>limestone ***</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
<td>$&lt; 0.2$</td>
</tr>
</tbody>
</table>

* Detailed values
** First tested absorption pressure: 2 - 4 MPa
*** First tested absorption pressure: 6 - 9 MPa
- 40 - 60 Hz due to the drill pipe by boring
- 300 - 320 Hz due to accelerometer mounting
- 1800 - 2000 Hz due to the piston of the hammer

The acceleration index ($I_a$) was defined as:

$$I_a = \frac{\sum S_i}{2 S_{50}} \frac{T^2}{T_{50}}$$

where:

- $a_{m}$ = 2 $S_{50}$ (SV/2)
- $i = 0, 1, ... N$ 2
- $S_i$ = power density spectrum
- $T = $ time of acquisition
- $N$ = number of acquisition
- $I_a$ is an index of rock compactness and heterogeneity:
  - if the rock is hard, $S_{50}$ is low and $I_a$ is high; the energy is partly transmitted to the head to throw the pipe.
  - if the rock is soft, all the energy is transmitted to the rock and low energy is transmitted to the head.
  - peaks indicate rock heterogeneity.

The values obtained using percussive drilling calibrated on the Murgia limestone are the following:

- calcarenite: $I_a < 1000$
- cavities and fissures: $I_a < 500$
- cavities and fissures filled by clay, detritus, $I_a < 800$
- very jointed limestone: $I_a < 1000$
- jointed limestone: $I_a = 1000 - 2000$
- sound limestone: $I_a > 2000$

5. ROCKS CHARACTERIZATION

The table shows the correlations between results obtained from instrumented 340 Pa.pe.ro. non coring drill and the results from 10 core drillings (rock description, $R_{QD}$) and acoustical soundings. In addition, the table shows the rocks characterization after consolidation grouting injections in 3257 boring (557654 m) grouting bores.

6. CONCLUSIONS

1. The Pa.pe.ro. system has permitted the geotechnical characterization of Gravina Calcarenite and Murge Limestone, for the foundations of the three viaducts.
2. The resulted rock characterization can be used for the entire territory of the Puglia region.
3. The instrumented non-core drillings executed before, in the natural rock and after, in grouted rock, allow an accurate control of consolidation.
4. The method fulfills economical geotechnical informations as an independent testing if it is calibrated with an even low number of standard core drillings.

REFERENCES

Characterization of the subsoil conditions at Swann's Bridge, N. Ireland

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ABSTRACT: This paper presents the results of an investigation into the properties and behaviour of a 37 m thick layer of stratified clay discovered during a site investigation at Swann's Bridge, Limavady, N. Ireland. From a study of the laboratory and in-situ test results on the clay a detailed picture of the properties of the material was achieved.

1 INTRODUCTION

The opportunity to study an unusual deposit of stratified clay arose as a result of the construction of a new bridge over the River Roe, north of Limavady in N. Ireland. Due to European legislation introduced in the late 80's all bridges on Regional Route Networks in the European Community had to be assessed for their load carrying capacity. The original Swann's Bridge was built in the early 1930's and had not been designed to carry today's larger loads. The bridge failed the structural assessment and a weight restriction of 7.5 tonnes was imposed. In 1991 the Department of the Environment (DoE) for Northern Ireland decided to replace the existing bridge.

A thorough site investigation was carried out which included: continuous sampling using the Wireline Core Drilling technique, piezocene penetration tests at four different positions close to the location of the new bridge piers. Maintained load pile tests were also carried out.

Continuous sampling revealed the existence of 37 m of stratified glacial lake material which is the deepest deposit of this type of material so far encountered in Northern Ireland (Doran 1992).

2 SUBSOIL CONDITIONS

The subsoil conditions found at the Swann's Bridge site were extremely interesting and a summary of the strata encountered in the area is presented in Fig. 1. Using the wireline Core Drilling Technique continuous samples to bedrock were obtained and were used for laboratory based studies. The top 13 m of material was extremely soft and was laid down after the last glaciation. The material was classified as Alluvium. Below this soft layer lay 37 metres of Stratified Clay known as "Limavady Clay". Below the Limavady Clay 15 m of Lodgeament Till was discovered and then finally bedrock was reached at 65 m below the ground surface. For the purpose of this research only the properties of the stratified clay were considered.

Four piezocene penetrations were carried out and provided a quick, accurate and economic method of obtaining a record of the ground conditions. Fig. 2 presents the results from the first penetration CPT 1.

All four profiles recorded at the site were very alike
identifying the uniformity of the subsoil conditions. Initially the cone resistance profile consisted of values less than 1 MPa punctuated by several large pulses. Considering the corresponding friction ratios it was clear that for the large pulses in the point resistance there was a drop in the friction ratio. The large pulses of point resistance 4-6 MPa with corresponding values of friction ratio 1 % identified a layer of "sand". These layers of sand were contained within a material which was classified as "sandy silts and silts". It is evident from Fig. 2, that at a depth of 13 m there was a distinct change in the all the profiles which indicated that the cone had entered a different strata of material. The change could be observed for all the piezocone tests by a marked increase in the values for the point resistance and friction ratio. In this region the point resistance had doubled from 1 MPa to 2 MPa and the friction ratio increased from 1 % to 2.5 %. These increased values identified the different strata as "clayey silts and silty clays". This strata begins at a depth of 13 m and continued to 50 m with a gradual increase in point resistance and a steady value of friction ratio. This form of profile continued with occasional large pulses in the point resistance with corresponding drops in the friction ratio and excess pore water pressure profiles which identified an existence of fine layers of "sand".

For each of the tests, the number of large pulses in the point resistance gradually increased until at a depth of approximately 30 m, there was a noticeable increase in the number and size of the pulses. The pulses, although large, were narrow, and established the existence of this layers of sand and silt. Hence the CPT results identified the two distinct zones within the layer of stratified clay.

A clear change in the strata was identified at a depth of 50 m by a large pulse in point resistance and friction ratio. This identified the start of the layer of Lodgement Till. The point resistance had increased from 3 MPa to 4 MPa and the friction ratio had increased from 2 % to 3.5 % which classified this strata as "silty clays and clays".

Figure 2 Variation of cone resistance, sleeve friction, pore pressure and friction ratio with depth from piezocone test 1
3 CLASSIFICATION

Numerous samples were taken from the cores for classification testing. The stratified nature of the material made it impossible to separate out individual layers on which to carry out classification tests therefore the tests were carried out on bulk samples. Figure 3 presents a graph, where moisture content is plotted against depth. From Fig. 3 the profile could be divided into two zones, represented by the two linear lines. The moisture content decreased from 38 % at 15 m to 31 % at 35 m. Below this there was a noticeable and sudden increase in the moisture content back up to 40 %. This increase was again followed by a gradual decrease to 32 % at 50 m. The increase in moisture content at depth was unusual as a gradual decrease with depth would be expected due to increasing overburden pressure.

The liquid limit over the full 37 m stratum was in the range 35-50 %. It is important to note that the liquid limit was greatest at the top of the layer and decreased with depth. The plastic limit was typically 20% and the plasticity index was in the range of 15-30%.

4 CONSOLIDATION CHARACTERISTICS

A large number of vertical one-dimensional consolidation tests were carried out on the Limavady clay so that the consolidation characteristics over the full depth could be studied. In Fig. 4 the preconsolidation pressures along with a linear line representing the effective overburden pressure are plotted.

In the upper zone (13-30 m) all the preconsolidation pressures determined were equal to or higher than the effective overburden pressure. Where as in the lower zone (30-50 m) all the preconsolidation pressures determined lie below the effective overburden pressure which was very unusual.

The estimated preconsolidation pressures in the upper zone (13-30 m) implied that the material was slightly overconsolidated. In the lower zone (30-50 m) all the values of preconsolidation pressure were lower than the effective overburden pressure i.e. the OCR=1.

Several possible explanations for the low preconsolidation pressures in the lower zone were considered.
(o) The possibility that the material was not fully consolidated was considered. Gibson (1958), proposed a method for determining the time for consolidation. Various times for the deposition of the 37 m layer of Limavady clay were considered, all of which produced a total time for deposition and consolidation (90%) of around 5000 years. As it was established that the material was deposited around 13000 B.P. it was clearly fully consolidated.
(o) Another hypothesis to explain the unusually low preconsolidation pressures (OCR<1) was given by the elucidation of a very similar problem by Casagrande (1936). Casagrande (1936) suggested the reason for the low values of preconsolidation pressure was due to water in the underlying rock.
being under an artesian pressure which was greater than the hydrostatic pressure. From the site investigation at Swann's Bridge the possibility that the bedrock contained water under artesian pressure was ruled out.

Therefore a different approach to this anomaly of low preconsolidation pressures at greater depths was required.

5 INTRINSIC CONSOLIDATION PROPERTIES

Burland (1990) proposed a basic frame of reference for interpreting compressibility characteristics using reconstituted properties on a wide range of clays. This basic frame of reference is termed the "Intrinsic Properties" for reconstituted clays which are inherent to the soil and independent of the natural state. This frame of reference provided an excellent opportunity to assess the unusual consolidation characteristics of Limavady clay.

By considering the normalised compression line for reconstituted Limavady clay along with the results presented in Burland (1990) all the curves fell on a reasonably unique line. This unique line has been labelled the Intrinsic Compression Line (ICL). The normalising parameter used was Void Index (IV).

Skempton (1970) considered the sedimentation compression curves for many different clays. Burland reproduced these curves but plotted them in terms of IV against Log ε'. Burland (1990) fitted a regression line to all the data and labelled this line the "Sedimentation Compression Line" (SCL).

If the sedimentation compression points for Limavady clay was analysed in two separate zones, the upper zone (13-30 m) and the lower zone (30-50 m). In Fig. 5 the points for the two zones have been plotted separately as well as the ICL and the SCL. There is a distinct difference in the results for the two zones.

The values for void index for the upper zone (13-30 m) are lower than those for the lower zone (30-50 m). The sedimentation compression curve in the upper zone is close to the SCL. From visual inspection of the clay in this upper zone the laminations of silt and fine sand are not as common as at the lower depths. It is the sedimentation compression curve in the lower zone (i.e. 30 m to 50 m) that is unusual. From visual inspection of the material at this depth very thin laminations of silt and fine sand are apparent and in greater quantity.

Fig. 5 shows that the sedimentation compression curve for the Limavady clay in the lower zone is well above the SCL, hence it can be concluded that the Limavady clay in the lower region has gained added resistance to compression from the structure of the material. This would have been as a result of a combination of factors namely the depositional environment and the material which was deposited. Since the Intrinsic Compression Line and Sedimentation Compression Curves for Limavady clay have been established it was then necessary to consider within the intrinsic framework the results from the consolidation tests on the undisturbed samples.

Fig. 6 shows the normalised compression curves for samples at 14.2 m and 33.6 m plotted with the ICL and the SCL. The line representing the undisturbed sample at 14.2 m comes across the ICL and reaches the SCL at which point the line gradually drops down. The "yield stress" or preconsolidation pressure predicted from the intrinsic frame work is 200 kN/m² which is in agreement with the value estimated for that depth. This result is representative of the consolidation behaviour in the upper zone of Limavady clay.

In Fig. 6 the normalised results for an "undisturbed" sample at a depth of 33.6 m is presented. The values lie below the ICL and gradually move towards the ICL eventually joining it at the greater pressures. The line never reaches the sedimentation compression curve for the lower zone. It is clear from studying the results within the intrinsic framework that they are closer to the ICL than the SCL, thus it can be concluded that the material in the lower zone has somehow been disturbed and so the results obtained for the "undisturbed" sample are more representative.
of a reconstituted sample than for an undisturbed sample.

6 SHEAR STRENGTH CHARACTERISTICS

A detailed knowledge of the shear strength of Limavady clay was required for the design of the most suitable and economic foundation for the new bridge. In Fig. 7 the results of the quick undrained triaxial tests (QU) carried out in the laboratory are presented. The line on the graph represented the values of shear strength which were expected for a normally consolidated clay which were determined from the relationship presented in Skempton (1957).

From Fig. 7 it is clear that in the upper zone 13-30 m the values of undrained shear strength determined lay on or above the line. Hence the results in this upper zone were higher than expected for normally consolidated clay indicating that the material was slightly overconsolidated. In the lower zone all the points lay below the line. From Fig. 7 the undrained shear strengths determined in the laboratory remained between 30-60 kN/m² over the full depth of the material. To try to confirm the integrity of the laboratory results it was imperative to obtain a measure of the in-situ shear strength of Limavady clay to determine if the anomaly was replicated.

The results from the piezocene tests provided a means of classifying the material, and they also provided a measure of the in-situ undrained shear strength of the Limavady clay. The undrained shear strength has been assessed using a cone factor $N_c$ in the equation (Powell et al 1988):

$$c_u = \frac{(q_d - \sigma_{uv})}{N_c} \quad \text{Eqn 1}$$

where $c_u$ = undrained shear strength, $q_d$ = corrected cone resistance, $\sigma_{uv}$ = overburden pressure, and $N_c$ = cone factor taken as 18.

Using equation 1 the values of undrained shear strength were determined over the full 37 m of Limavady clay and the results are presented in Fig. 7. The results over the full depth all lie above the line representing the values estimated from Skempton’s (1957) relationship. When Fig. 7 was considered it was clear that the in-situ undrained shear strength measured over the full depth was in the range 100-130 kN/m². This was substantially higher than the range of shear strengths determined in the laboratory from the quick undrained triaxial tests (30-60 kN/m²). Therefore the laboratory test results from the quick undrained triaxial tests were not representative of the in-situ behaviour.

As part of the site investigation at Swan's Bridge Dr. I.G. Doran & Partners (Geotechnical Advisers) recommended driving two test piles, one on each side of the River Roe close to the proposed position of the piers for the new bridge. The test pile specifications used were:
1. 2.85x2.85 mm precast concrete pile
2. 35 m long consisting of five 7 m segments
3. The design load was 70 tonnes

These tests provided the in-situ pile capacity and the factor of safety (it was hoped that an estimate of the
in-situ undrained shear strength could also be obtained.

Due to the importance of determining the pile settlement it was decided to use "maintained load" tests to discover the ultimate carrying capacity and the settlement of the piles during the loading. Both test piles were loaded to failure. In Test Pile 1 failure occurred at 225 tonnes and Test Pile 2 failed at 250 tonnes.

From the analysis of the pile test results (Fleming 1992) it was noted that there was very little base resistance developed. Knowing the skin friction provided by the pile and the length of the pile it was possible to estimate the undrained shear strength of the clay in-situ:

\[
\begin{align*}
\gamma & \text{ at } 13 \text{ m } = 119 \text{ kN/m}^2 \\
\gamma & \text{ at } 35 \text{ m } = 137 \text{ kN/m}^2
\end{align*}
\]

Therefore from the piles tests the undrained shear strength of Limavady clay was estimated to be in the range 119-137 kN/m² between 13-35 m. It was clear that the CPT tests produced results which were in agreement with the estimated values from the piles tests.

7 DISCUSSION & CONCLUSIONS

For the construction of the new bridge over the river Roe an initial site investigation identified a 37 m layer of stratified clay. Laboratory tests on stratified clay uncovered unusually low preconsolidation pressures and undrained shear strengths at the lower depths. A further investigation was specified which included continuous sampling by the wireline core drilling technique, piezocone tests, maintained load pile tests and more intensive laboratory testing of the samples. From this further investigation it was discovered that the 37 m layer of stratified clay could be divided into two zones an upper and a lower zone. The preconsolidation pressures and the undrained shear strengths determined in the laboratory from the lower zone were very unusual and needed further investigation.

By considering the consolidation characteristics in the Intrinsic framework it was identified that the material in the lower zone when removed from the ground behaved as a reconstituted clay.

To check the integrity of the quick undrained triaxial tests the in-situ shear strength was determined from the piezocone and pile tests. This confirmed that the quick undrained triaxial tests carried out in the laboratory did not provide a true measure of the in-situ behaviour. Hence even when using a high quality sample recovery technique such as wireline core drilling sample disturbance can occur which can produce misleading results.

The use of the Intrinsic framework provided a clear indication that the samples in the lower zone had become disturbed.

By using accurate in-situ testing a detailed and accurate characterization of the ground profile was achieved.

Using the results of the laboratory and in-situ tests a suitable piled foundation was completed and the bridge was successfully built. The final design was cost effective and provided a considerable saving for the client.

ACKNOWLEDGEMENT

QUIS, DEAN, Dr. L.G. Duran & Partners & Dal Roads Service (NI) for funding and for use of the data recorded from the site.

REFERENCES

The use of electronics in the management of site investigation and soil improvement works: Principles and applications

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ABSTRACT: This paper outlines the use of electronics in geotechnical investigations, with particular emphasis on instrumented drilling. The concept, interpretative principles, and guidelines are described. A case history of geotechnical investigation by instrumented drilling in soil improvement works completes the dissertation.

1. INTRODUCTION

The continuous growth in capabilities and reliability of computers, combined with the development of technical and managerial software packages, has fostered use of electronics in the construction industry. Correspondingly, many companies in the industry are optimizing their operations, developing a comprehensive information system able to support the site activity and ensure an effective quality control.

Main advantages of applying computers in the construction process are:

- facilitate the control of technical information and results produced during the construction process and their conformity with the project specifications;
- automate the data processing, work documentation and certification;
- compliance with the ISO9000 quality control regulations.

To this regard, a few construction companies have initiated the so-called "site data processing chain", or a computer system able to elaborate data recorded by an acquisition system through dedicated engineering software.

Thus, allowing a high level of quality control of the work performed. An example of the information chain for a typical grouting site is illustrated in Figure 1. The concept of the data processing chain can be applied to a variety of activities. This paper is focused on the use of instrumented drilling in site investigation and process control.

2. INSTRUMENTED DRILLING

In the geotechnical industry, the digital recording of drilling parameters represents a useful and economic tool in the ground investigation phase, as well as in the verification and certification of the end product.

Figure 1. A typical site data processing chain for grouting works.
interpretative methods are now slowly converging in a uniform interpretative theory, on the basis of the large number of applications and progress of computer tools.

One of the main advantages of instrumented drilling is represented by its relatively low cost and fast realization time. In addition, the results can be compared to in situ penetrometric tests, with the advantage that instrumented drilling can be used to collect representative data from a wider range of soils and rock conditions.

2.2 Drilling Parameters Recording Systems

A drilling parameter recording system pentes fundamentally on the following elements:

- transducers, or devices mounted on the drill rig, able to measure physical parameters and transmit data in form of electrical pulses to the data logger unit;
- data logger unit, made with reliable and resistant components to endure site use. It usually allows for user interface (keyboard, monitor or display, etc.), as well as print-out;
- dedicated software loaded on the data logger, allowing real-time visualization/printing of data and recording on magnetic support.

Since 1985, Rado have finalized their own system, the PAPERO (Pianostruzione Perforazione Rado, drilling parameters Rado), developing the hardware and software and interpretation criteria for soil characterization by drilling. This system can be employed in a very wide range of subsurface conditions, varying from fine soils to rock, resulting in a highly versatile tool.

The system is able to record the following parameters (see figure 2):

- penetration speed
- rotation speed
- effective thrust
- effective torque
- pressure and flow of drilling fluid
- acceleration of the drill string (vibration)

The instantaneous values of the above parameters are measured at intervals as low as 10 μsec and memorized every 500 μsec or every centimeter of penetration. The data logger, powered by battery or external supply, converts the electric signals into numeric data, presents the data in numeric and graphic format, records the data at the selected frequency. A designated printer can be used to print on site preliminary graphics of the main parameters. Eventually, the data can be further analyzed by a personal computer.

Characteristic applications of the system are:
- determination of fine and coarse alternances in alluvial deposits;
- determination of soil/bedrock interface;
- localization of tectonic discontinuities (faults, fissures) and karstic phenomena;
- delineation of boulders or obstacles;
- verification of soil improvement works (grouting, jet grouting, soil mixing, etc.).

3. DATA INTERPRETATION

Drilling parameters can be categorized as primary parameters and control parameters, the former being the most sensitive of the stratigraphic variations, while the latter, impacting the drilling procedures, can be used to enhance interpretation of the results obtained.

The main primary parameters are:
- penetration speed;
- torque on the drill string;
- vibration of the drill string.

The control parameters are:
- drilling method, type and dimension of drilling tools (drill bit, rods);
- pull-down and rotation force on the tools;
- pressure of drilling fluid;
- flow rate of drilling fluid.

It must be emphasized that, in order to use the control parameters for a combined evaluation of the soil characteristics, the drilling operation has to be as close as possible to ideal conditions, i.e.: drill cuttings promptly removed from the hole, avoiding plugs or “collars” around the rod, adequate and continuous circulation of drill fluid, and tools in good operating condition.

Undoubtedly, the most indicative and immediate primary parameter is the penetration speed, which, in the same operative conditions (pull down force, rotation, etc.), is a function of the soil characteristics. Nonetheless, similar penetration rates can be obtained when drilling through considerably different soils (for instance stiff cohesive materials interbedded with coarse granular layers); in these instances, secondary parameters like torque and vibration on rods, or the circulation conditions, are extremely meaningful.

The analysis of the data recorded by the PAPERO system is attained by a three-step evaluation process.
The first step is represented by a comparative breakdown of the parameters recorded and their graphical representation as a function of depth. This process aims at the determination of possible variations and establishes their geological or operational origin. There are various examples of possible interpretation available in the literature, mainly focused on the variation of drilling parameters (also defined as "analogic approach"). However, the writers believe that a high degree of subjectivity and indetermination is still inherent to this approach, relegating it to just an initial stage of the interpretation process.

The second step is constituted by the analysis of the data recorded on the basis of the energetic approach, i.e., correlating the energy spent to drill a defined hole length, to the strength characteristics of the soil. The basic concept is that a small diameter drilling operation reproduces, on a different scale, mechanism similar to large excavations. The relation is expressed by the specific energy, or the work spent to excavate a unit volume of soil. The drilling specific energy is expressed by an algorithm calculating the energy in function of parameters recorded at the selected frequency.

The definition of this measure was the scope of experimental research conducted by Tsoutsoulis (1969), Punno et al. (1963, 1966), Bregman (1974), Brown & Barr (1979, 1983) and is based on the following specific energy theoretical equation (Tats, 1965; Rowlands, 1971; Brown 1978, 1983):

$$ F = \frac{2 \pi N T}{A V} \quad [\text{KJ/m}]$$

where:

- $F$ = [KJ] = pull down force
- $A$ [m2] = hole section
- $N$ [rpm] = rotation speed
- $T$ [KJ/m] = applied torque
- $V$ [m/sec] = penetration speed

In the calculation of specific energy, the characteristics of the drill rig and the drilling procedures are considered, so that a meaningful or accurate value can be obtained, independently of the drill rig utilized.

The second step is then the energetic approach, employed to obtain geological and geotechnical information. The third step is described in the following section.

3.1 Export System for Energetic Characterization

A further level of interpretation of the data is constituted by their statistical analysis. Statistical studies of frequency distribution for a given homogeneous soil show that the specific energy values are concentrated on a gaussian unimodal type of distribution, classifiable according to the Pearson model. By drilling preliminary holes in the vicinity of calibration borings with continuous recovery, it is possible to determine the frequency distribution for each soil stratum encountered in the borehole, and assign to a specific type of soil a characteristic drilling energy.

A necessary condition for an accurate determination of a characteristic curve, is the analysis of homogeneous and congruent data. To satisfy this condition, it is necessary to appraise the factors influencing the drilling parameters, like:

- type of soil;
- drilling method;
- geological and hydrogeological situation;
- depth of the measurement;
- the inclination/deviation of the hole;
- the relative density (for granular deposits).

The large quantity of data recovered in past investigations, allowed the creation of comprehensive data-base, correlating diverse geologic conditions with specific energy values. The availability of continuously enhanced software, permitted the creation of a specific program (Export System for Energetic Characterization), optimizing the numeric analysis and enabling the "translation" of energy values in stratigraphic information, in consideration of the boundary conditions.

The program is structured in a sequence of Criteria, a list of Indexes, and a data Baseline.

The Criteria are the boundary conditions/filters/assessment of the project (see above).

The Indexes are the result of the statistical analysis and are the base for the interpretation:

- population;
- the mode;
- the average;
- the standard deviation;
- the variance;
- the quartile;
- the decile.

Specifically, the mode gives a reference energetic value for the soil type; the values corresponding to the 90% confidence interval conventionally allow designation of a specific lithotype; the square root of the deviation and the semi-difference between quartiles, giving indication on the dispersion of the data, are useful in the verification of soil improvement works, or in the differentiation of heterogeneous granular formations having similar energy characteristics.

The Baseline is constituted by the whole of the drilling energy values stored in the database and to which the Program refers in the determination of the soil type. Currently the Baseline contains over a million of reference values.

The Program prepares the "general" frequency distribution of the data analyzed and selects the characteristic curve in the Baseline corresponding to the Criteria identified by
the user. The comparison between the actual distribution and the Pearson distribution curves (differentiated by the parameter $K$) allows then the calculation of the indexes relative to the recorded values, and to translate the single energy values into stratigraphic informations (i.e.: rock-void, convoid soil-loose soil, etc.). The Pearson statistical classification is based on the k parameter function of the curve shape, where:

$$K = \frac{\beta_1 (\beta_2 + 3)^2}{4(2\beta_2 - 3\beta_1 + 6)(4\beta_2 - 3\beta_1)}$$

where:

$\beta_1 = \text{asymmetry coefficient}$

$\beta_2 = \text{flattening coefficient}$

In practice, the final result of the statistical analysis, is the determination of limit values (the interval of confidence) for every soil, from these, it is possible to assign to any energetic value a corresponding soil type, thus establishing the local stratigraphy sequence without continuous recovery boring.

4. CORRELATION BETWEEN SPECIFIC ENERGY AND GEOTECHNICAL PARAMETERS

The increasing demand for quality verification and certification of soil improvement works, triggered studies aimed at defining empirical relationships between specific energy and certain geotechnical parameters. In particular, the effort has been directed towards the correlation of the drilling energy with unconfined compressive strength (UCS) and blow counts in Standard Penetration Tests (SPT). The basic approach of the study is the energetic characterization of the soil pre- and post-treatment.

For the UCS part, correlations have been made between the strength recorded in laboratory test on specimen of grouted soil and the energy recorded in an envelope of $\pm 30$ cm from the sample location (see Figure 3). Correlations have also been made with the in situ pressometric modulus. The correlation shown should be considered as an indicative trend at this stage, as it is currently subject of further research.

Similarly, several tests have been made in different soils, from coarse alluvial to silty clays, for correlation with SPT data. Figures 4b and 4c portray the energy values relative to gravelly sands and silty sands respectively, the relative grain size distribution curves are shown in Figure 4a. The different distribution of the Npt for these two soil groups is remarkably evident and is well depicted by the specific energy values. It must be also noted that the regression curve for the coarse soils is almost linear, and with a gradient greater than the fine soils.

![Figure 3](image-url)  
Figure 3. Correlation between unconfined compressive strength and drilling energy in grouted sandy soil.

![Figure 4a](image-url)  
Figure 4a. Specific Energy - Npt in grained sandy soils.

![Figure 4b](image-url)  
Figure 4b. Specific Energy - Npt in gravelly sands.

![Figure 4c](image-url)  
Figure 4c. Specific Energy - Npt in silty sands.

![Figure 4d](image-url)  
Figure 4d. Specific Energy - Npt in clay soils.
Figure 4d draws the relation between Npt and specific energy for silty soils. Although the geotechnical interpretation of Npt data in silty soils is often questionable, this diagram is shown for comparison purposes.

In conclusion, the preliminary phase of the study confirms the direct correlation between drilling specific energy and UCS of treated soils and Npo of granular soils.

5. DATA PROCESSING CHAIN: A CASE STUDY

The Aurelia Tunnel, on the Rome-Macerates railway line, was subject to widening work. A 160 m long section in Rome’s Via Frosinone, between station 171+387 and 171+549, required a continuous approach due to prior damages and settlement occurred on the buildings above the existing tunnel alignment, prior to the commencement of the widening works; consequently, an extensive and detailed site investigation was undertaken, followed by a grouting test program.

The soils surrounding the tunnel comprise alternating layers of very stiff grey-bluish clays (Vatican Clays) and sandy silts or medium silty sands. The variable dipping of the pliocenic basement required tailoring grouting parameters in order to improve the granular strata to mechanical properties at least equal to the clays.

The geological investigation comprised 13 instrumented borings, aimed at defining the local subsurface profile and defining a baseline for the eventual verification of the grouting works.

The results of the grouting test constituted the basis for a detailed grouting program, comprising traditional continuous mixes integrated by silica based gurts, injected on a “controlled velocity-refrual pressure” basis.

As part of the grouting works, a systematic verification program was implemented to progressively evaluate the efficiency of the treatment and investigate deformations induced in the surrounding of the tunnel. The program was so composed of:

- topographic survey;
- automatic control of the injection process through electronic instrumentation;
- localized verification of the effectiveness of the treatment by instrumented drilling.

The surface topographic survey was performed daily to monitor possible settlements on buildings on deep foundations (bored piles) possibly impacted by the tunnel excavation. Convergence measurement were also taken daily on the tunnel lining.

The automatic management of the grouting process, performed by electronic control of the grout pumps via computer, allowed the continuous and real-time adjustment of the injection parameters according to the soil response and within pre-selected criteria. The injection parameters (pressure, flow, takes) were summarized and synthesized by dedicated software, allowing a tomographic analysis of the grouted mass. This analysis lead to the allocation and spatial distribution of the injection parameters for each pass.

The verification of the soil improvement effectiveness was done by instrumented drilling. A large number of holes were drilled after each grouting phase through the treated zone, thus allowing the collection of a large number of data at different locations in an efficient and economic manner.

The comparative analysis of the drilling energy frequency distribution (before-after soil treatment), showed a progressive tenfold increase, going from 16 MJ/m² in natural soil, to 140 MJ/m² at the end of the treatment. A drilling energy comparison between two adjacent holes, before and after the treatment, is shown in Figure 5. It is evident that the energy required to drill a hole having the same geometrical characteristics, before and after the soil improvement works, is drastically different.

In conclusion, the drilling energy analysis, detected a remarkable increase of punctual values as well as the corresponding indexes in the grouted granular soil, to values comparable to the very stiff clays. An indicator of the treatment efficacy is the ratio E/E₀, where E and E₀ are respectively the drilling energy typical of the treated and natural soil. The ratio is proportional to the treatment effectiveness, and, in the same boundary conditions (i.e., soil type, grout mix properties, etc.), allows an accurate and economic evaluation of a grouting work. In addition, the use of instrumented drilling for soil improvement verification, is practically preferable to traditional coring; in fact, coring and sampling of treated soil generates high
disturbance to the samples, thus providing a limited assessment of the treatment quality.

REFERENCES


Detection of cavities by monitored borehole drilling (TMD)

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ABSTRACT: A quick soil investigation method has been successfully adopted to detect cavities in Beddawi, north Lebanon. It consists of drilling with tricone with a monitoring equipment. Digitally recorded data can be easily handled for further processing and representation. This procedure allows a good fitting of the geotechnical data profile with additional information on the presence of cavities, weak rock layers, weathering degree.

1. INTRODUCTION

Within the works for the post war reconstruction of Lebanon, Ansaldo Energia is building, for Electricité du Liban, a new 220 MW combined cycle power plant in Beddawi, north Lebanon (figure 1).

Most of the plant lays on platforms excavated into the rock at different levels to follow the original morphology of the site in order to reduce the excavated volumes. As the excavation for the site preparation took place, a big cavity, of about 50 m³, has been discovered some few metres apart from the main power block area.

Such a worrying discovery induced Ansaldo Energia and Electricité du France, the Engineer, to undertake a massive investigation campaign in order to find out if other cavities could be present beneath the main foundations of the power plant.

2. GEOLOGY

Three main rocky formations are present in the area of the plant: the upper formation is a calcareous sandstone, the middle one is a calcareous conglomerate and the lower one is a yellow calcareous sandstone. All these formations are Miocene rocks. The upper one is a very hard and cemented pale beige calcareous sandstone, with a cementation degree generally ranging from medium to very high, but some weaker zones can also be found where the action of the water has partially dissolved the bounding cement. The second one is a calcareous conglomerate chemically very close to the previous one since its matrix is made of calcareous sandstone and also the cobbles are mainly calcareous and secondarily marly. The third formation is a partly cemented deposit, so that the structure of this

Figure 1. Layout of Beddawi Power Plant.
yellowish calcareous sandstone is not homogeneous
and part of the rock is a moderately cemented
calcareous sandstone passing elsewhere to a silty
sand. The three calcareous formations are overlain
superficially, by a layer of reddish residual soil which
is very frequent in these regions where the climate
changes seasonally from hot and dry to humid with
sudden and stormy rainfalls.

The site is on a hilltop, on the coast of the
Mediterranean Sea, dipping towards north-west and
the morphology of the site, with a wall at the centre,
is favourable to the collection of the water rushing
from the surrounding areas. A path of joints and
cracks assures either the percolation of the meteoric
water into the calcareous sandstone and the
circulation of the underground water which comes
from the nearby mountains.

The problem of the formation of cavities is very
complex and it is out of the aims of this paper. Only a
brief description of the most likely theory will be
shortly discussed within the following lines.

The superficial meteoric water, which enters into
the ground by the action of the gravity force,
occupies the pores and fractures of the rocks up to a
surface, called "water bearing surface" whose level,
in normal conditions, varies with the season and with
the intensity and frequency of the rainfalls. The water
bearing surface divides the soil into two main parts:
the lower one is the "water bearing zone" which is
completely saturated with water and no air is present;
the upper layer is the "aired zone", where the water is
only temporarily present during the way to the water
bearing zone.

The underground water has a main role in the
process of erosion and the production of subsurface
cavities acting onto the soluble components of the
rocks and forming a three-dimensional path of
chambers, tunnels, siphons that often follows an
original system of fractures. Cavities can be empty
or, more frequently, filled by loose sand or even
saltier material coming from the weathering process.

So far there is not still a clear and universally
accepted theory on the origin of cavities. It is widely
believed that the cavities follow two main distinct
stages during their evolution. During the first stage
the rocks of the water bearing zone are dissolved
along the cracks and the bedding surfaces: the
meteoric water contains some carbon dioxide which
transfers the calcium carbonate of the calcareous
rocks into calcium bicarbonate, that is soluble into
the water.

The second stage begins when the so formed
cavity emerges from the rock bearing zone to the
aired zone because of a tectonic uplift of the rocks or
the lowering of the water bearing surface, then the
cavities are transformed either by the mechanical
action of the flowing water and the chemical action
of the air-water system. The mechanical action of the
water contributes to a further enlargement of the
cavity, while the chemical action of the air-water
system contributes to the deposition of calcium
carbonate either in an amorphous status and/or in
crystalline form.

3. CAVITIES

Three main kinds of cavities have been found in
Bredasi. The first one is the chemical frank void
where part of the original rock has been completely
dissolved by the chemical action of the circulating
water, and the deriving cavity is kept clean from
sediments by the mechanical action of the circulating
water. In this kind of karst structure the water plays
the two complementary roles of solution and
transportation agent. The second kind of cavities are
those where most of the bounding cement, linking
the particles of the deposit, has been dissolved, but
the grains of the sandstone are still in place. In this
case a part of the sandstone undertakes a
decomposition process which takes it back to a
slightly cemented sand.

In many cases these two kinds of cavities coexist.
There is a inner core, completely void, and an outer
part made of slightly cemented sandstones as
indicated in figure 2.

In some cases a multilayered thin interbedding of
these two forms of cavities, as indicated in figure 3,
has been found.

Figure 2. Sketch of cavity type 1.

Figure 3. Sketch of cavity type 2.
The third kind of cavity has been found within the lower yellowish calcareous sandstone formation. It is a sort of path of tunnels within the rock. Sometimes these tunnels are completely full of water while, in some other cases, these channels are filled by sediments. The section of these cavities is usually circular to elliptical and the cross surface is often less than 0.3m² but their axial persistence is considerable (feet of metres).

All the three kinds of cavities, if massive enough and if present at shallow depth with respect to the foundation's level, could affect very seriously the stability of the structures of the power station. Thus it was absolutely imperative to find out, in proximity of the main foundations, if some cavities were present and if their characteristics made them dangerous for the structures.

The presence of a widespread continuous mesh of cavities, tubular shaped and interconnected, even if of relatively small dimensions, was a problem for the temporary dewatering needed for the construction of the pump house, 12 metres below m.s.l. These kind of cavities are pipelines for water coming into the excavation: either for salt water from the sea and fresh water from the water table headword.

It was thus of paramount importance to find the zones and the depths at which these meshes of cavities were located. Geophysical methods have often given adequate response to the problem of the detection of cavities. However geophysics have some limitations related to the boundary conditions of the problem, leading to a series of models which mathematically fit with the acquired parameters rather than giving an universal description of the subsurface situation. These models should be critically filtered by an accurate geological interpretation.

Among the geophysical methods, ground penetrating radar would be the fastest and most precise one to find out cavities at shallow depth. Unfortunately this technique couldn’t be used because of a widespread presence of water at very shallow depth which causes a drastic energy attenuation and a widespread scattering of the signal.

Micro gravimetry method could not be used too in Beddawt since it would have been necessary to stop, for the whole period of the campaign, all those site activities, producing noise to the gravimeter.

4. INVESTIGATION METHODS

The only geophysical investigation that could be executed under such circumstances was an electrical resistivity survey. In a first phase 65 electrical resistivity profiles, according to the Wenner configuration, have been performed. In the second phase, over the detected anomalies, other 36 vertical electric soundings, using the Schlumberger method, have been carried out.

The adopted geophysical method has been able to produce a panoramic view of the limited areas of the power block, showing the zones where it was most likely to find out some cavities, but few considerations could be made about their precise position and dimensions. For that reason the need to do something for verifying directly the actual situation below the foundations was established.

Since geotechnical boreholes are expensive, need time and skilled personnel and are often misleading as result in failing of core recovery, it was then decided to carry out continuous monitored tricone drilled holes (TMD).

The tricone monitored investigation technique consists on drilling with a traditional rotary drilling machine and recording some of the most important drilling parameters.

The drilling parameters to be recorded can be divided into two main categories: speed parameters and pressure parameters. The speed parameters, which are also called primary parameters, are mainly the advancement speed and the rotation speed. The pressure parameters are the drilling fluid pressure; vertical push; negative push; rotation torque; these are also called monitoring parameters. Generally they are not directly involved in the interpretation model, and should be kept as constant as possible.

A scrupulous calibration of the system was made by comparing the TMD results to well known situations previously checked by direct inspection and core recovering. Calibration TMD holes have been drilled near artificial slopes, where it was possible to relate the changes within parameters to the changes within the stratigraphy and the degree of cementation of the encountered deposits. Furthermore, to improve correlation in the zones where vertical slopes were not present, three high quality full coring boreholes have been drilled 1.5 m apart from three corresponding TMD holes.

Default values for the TMD procedure such as power, pressure on the rods and torque, to be adopted during the investigation campaign, have been obtained from pilot TMD drillings near a big cavity.

Once the optimisation of the pressure parameters has been reached, every effort was adopted to keep these parameters as constant as possible.

Vertical advancement speed and rotation speed are those parameters that can be correlated to the rock-bed properties and to the presence of weathered zones or cavities, while the recording of the drilling fluid flow can substantiate the transition from one layer to another with different permeability and the presence of a cavity: loss of fluid and sudden drop of the pressure. Even if those 3 parameters are particularly significant, since in most cases they can
be directly related to the soil/rock properties, all the parameters mentioned in figure 4 should be critically analyzed and cross checked in order to obtain reliable data. In fact only by an overall analysis of the data, including the so-called monitoring parameters, a reliable interpretation can be achieved.

Composite parameters have been proposed (Hanenin & Puvilland, 1987), in order to be able to interpret the data when the monitoring parameters are not taken constant during the work. For the purpose of the detection of cavities, where the contrast within the soil/rock parameters vary in a very wide range, the use of these composite parameters, such as hardness, index of alteration and Sonerton index, did not reveal to be very helpful. Direct parameters, such as the advancement speed and rotation speed as well as the flow of the circulating water, give a more direct view of the contrast between sound rocks and cavities; composite parameters on the contrary can be successfully used to distinguish another variations of the soil/rock characteristics in a much narrow range of variations.

In the case of Beddawi, the monitoring have been set constant while the data were scanned each centimetre of drilling advancement. This thin sampling frequency is stated considering that perforation is a process with quite a strong aleatory component.

5. MONITORING EQUIPMENT

The monitoring equipment for TMD procedure is composed by: a timer, transducers (displacement, velocity, angular velocity, torque, pressure and flow), a display on which one or more parameters can be represented in real time, a magnetic RAM card for data storage. Data can be directly downloaded on the hard disk of a portable computer for a further elaboration and representation (see fig. 4). The equipment can be easily fitted to all types of drilling machine.

6. RESULTS

The results obtained by TMD procedure fitted the scope of finding cavities. An adequate calibration of the recorded data has been put forward first, by performing TMD tests close to outcrops of the soil/rock type on slopes or close to geological boreholes executed with particular care. TMD tests were used either to find local situations and to picture a general view of the selected areas.

An extensive campaign was carried out starting with a borehole pattern, then adding more boreholes in these zones where cavities have been found or are suspected to be.

Figure 5 shows two examples of the results obtained. The advancement speed and rotation speed only have been plotted. The results have been obtained with reference to the following default values of the monitoring parameters maintained constant: vertical push = 125 bars; negative push = 75 bars; torque = 50 bars; fluid pressure = 5 bars.

In plot 5 a) the cavities are marked and can be clearly seen. A cavity is present from 5.6 metres to 6.4 metres with two main parts clearly visible. Down to 6.15 m, there is a slightly cemented sandstone, passing to a cavity filled by loose sand, below 6.15 m an air-filled cavity is present, down to the depth of 6.4 m. The advancement speed increases from 1 m/min to more than 4 m/min. Other two air-filled cavities are present at 6.35 m, and another at 6.85 m. The cavity at 9.6 m is already partly made of weakly cemented sandstone and partly air filled. The rotation speed plot, although less representative than the one of the advancement speed, is helpful in confirming the presence of these cavities, above all the air-filled ones where the rotation speed increases clearly.

Plot 5 b) is a dense interbedding of cavities, weak levels and hard rock that can be deeply recognized only by recording drilling parameters. Each of these data, could be almost meaningless by itself, but the global data are essential to portrait precisely the condition of such an heterogeneous deposit.

From the experience acquired in Beddawi site we can try to define some ranges for the advancement speed within the calcareous sandstone:

Hard calcareous sandstone: 0.1 to 0.3 m/min; moderately cemented calcareous sandstone: 0.4 to 0.6 m/min; slightly cemented calcareous sandstone: 0.6 to 1.2 m/min; cavity filled by sediments: > 3.0 m/min; air-filled cavity: > 3.0 m/min.

These values are referred to a 85 mm tricone drilled hole with monitoring parameters set at the values previously specified, using a drilling machine with a maximum torque of 1000 kgm.

When enough data are collected from TMD the three-dimensional relationships between the rock types, weathered zones, cavities, etc., can be defined.
Figure 5. Examples of interpretation of TMD logs.

Figure 6. 3D view of the spread of cavities in the pump house area.
TMD plots are superimposed to the soil profile obtained from the geotechnical boreholes highlighting those depths, those levels, and those locations where cavities or rock weathering are present. Figure 6 gives a 3D view of the pump house area where weak rocks are detected in which spider meshes of tubular cavities are present, confirmed by visual inspection at the excavation stage. Figure 7 shows similarly the presence of thin localized lens shaped cavities and weaker layers in the tank farm area, which is 11 m above the pump house platform.

7. CONCLUSIONS

The problem of cavities is, still now, a major problem since the soil investigation for geotechnical purposes is generally punctual and maybe that none of the boreholes meet any cavity or that the cavity may be not recognized. In calcareous rocks this problem could be hardly overemphasised. First of all a deep geological and geomechanical study is needed, possibly supported by a geophysical investigation in order to put forward if in the soil/rock under examination cavities are likely to be expected or not. If yes, TMD procedure can be a powerful tool to investigate the presence of cavities after having decided the drilling pattern and the investigated depth. Plots of the recorded parameters versus the soil profile can give really a picture of the subsoil showing sound rock, weak rock, cavities, changes in soil types etc.

Advancement speed and rotation speed can give a picture of the soil profile with respect to rock soundness and weathering degree, but only a complete cross-check of all the recorded parameters can allow a reliable interpretation.

To get the procedure successfully, calibration tests are absolutely needed. A standardisation of the procedure is foreseen; it would lead to an interpretative key of the TMD drilling log.

REFERENCES


Effect of sample disturbance on yield stress and compressibility of Ariake clay

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ABSTRACT: Ariake clays have been widely deposited around Ariake Bay in Japan. The basic characteristics of Ariake clays are very soft and sensitive. In this study, the undisturbed Ariake clays are deliberately disturbed to various levels of disturbance. Then, the samples with different levels of sample disturbance are used for standard consolidation tests to investigate the effect of sample disturbance on mechanical parameters. And a simple index for providing a quantitative measure of the degree of sample disturbance is proposed.

1. INTRODUCTION

Ariake clays have been widely deposited around Ariake Bay, located at the western part of Ryugu Island, Japan. The basic characteristics of Ariake clays are very soft and sensitive. The natural water content is often greater than the liquid limit. In addition, the thickness of the Ariake clay deposit varies generally 10-30m, with the maximum being 40m. Hence, the settlement and the stability in the area of such a marine deposit are serious problems in the engineering practice. Accurately assessing the consolidation yield stress and the compressibility are indispensable in the settlement and the stability analyses. Stress release and mechanical disturbance during sampling and handling, however, may lead to a significant difference in the mechanical parameters.

2. TESTING PROGRAM

In order to investigate the effect of sample disturbance on the mechanical behavior of Ariake clays, a vibration equipment was used to disturb deliberately the undisturbed Ariake clays to various levels of sample disturbance. The vibration equipment is schematically shown in Fig.1. The soil specimen was put on the vibration table, and vibrated for a certain time. It is considered that the level of sample disturbance varies with the vibration time.

The Ariake clays used herein were obtained from the same depth at the same site in Saga Prefecture of Japan with the polyvinyl chloride pipes. The basic physical properties are shown in Table 1. It can be seen that the natural water content is about 1.5 times the liquid limit. Then, a series of standard consolidation tests were performed on the samples with different levels of sample disturbance by vibrating to investigate the effect of sample disturbance on the consolidation properties of Ariake clays as follows.
Table 1. Basic properties of Ariake clay

<table>
<thead>
<tr>
<th>property</th>
<th>value</th>
</tr>
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<tbody>
<tr>
<td>density of soil particles (g/cm³)</td>
<td>166</td>
</tr>
<tr>
<td>natural water content (%)</td>
<td>164</td>
</tr>
<tr>
<td>liquid limit (%)</td>
<td>117</td>
</tr>
<tr>
<td>plastic limit (%)</td>
<td>52</td>
</tr>
<tr>
<td>sand content (%)</td>
<td>0.9</td>
</tr>
<tr>
<td>silt content (%)</td>
<td>38.6</td>
</tr>
<tr>
<td>clay content (%)</td>
<td>60.5</td>
</tr>
</tbody>
</table>

3. INTERPRETATION OF OXYMETER TEST DATA FOR ARIAKE CLAYS

Fig. 2 shows the typical compression curves of the samples with different levels of sample disturbance by vibrating, designated T-1 through T-12. It can be seen that the consolidation test data can be well interpreted by two linear lines in the ln(1+e)-log p plot. The interpretation technique using a bilinear plot was firstly proposed by Butterfield (1979). Ohtsuka et al. (1995) have verified the validity of the ln(1+e) versus log p method for Ariake clays. The stress at the intersection point of the bilinear lines in the ln(1+e)-log p plot is the consolidation yield stress, designated p<sub>y</sub>. The bilinear lines bounded at the yield stress are termed as the preyield and the postyield lines, respectively. Their slopes are defined as the compression indices in the ln(1+e)-log p plot, designated C<sub>CLa</sub> and C<sub>CLA</sub>, respectively. The consolidation test results are summarized in Table 2 for the samples T-1 through T-12. It can be seen that the initial water content keeps almost unchanged with the level of sample disturbance. That is, the basic physical properties are almost not affected by sample disturbance. The mechanical parameters shown in Table 2, however, depend on the level of sample disturbance. The effect of sample disturbance on the compression index in the preyield state C<sub>CLa</sub>, the consolidation yield stress p<sub>y</sub>, and the compression index in the postyield state C<sub>CLA</sub> will be discussed as follows.

Table 2. Consolidation test results for an Ariake clay

<table>
<thead>
<tr>
<th>sample</th>
<th>T-1</th>
<th>T-2</th>
<th>T-3</th>
<th>T-4</th>
<th>T-5</th>
<th>T-6</th>
<th>T-7</th>
<th>T-8</th>
<th>T-9</th>
<th>T-10</th>
<th>T-11</th>
<th>T-12</th>
</tr>
</thead>
<tbody>
<tr>
<td>vibration time (min)</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
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<tr>
<td>initial water content (%)</td>
<td>16.4</td>
<td>15.4</td>
<td>15.7</td>
<td>15.6</td>
<td>15.4</td>
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<tr>
<td>C&lt;sub&gt;CLa&lt;/sub&gt;</td>
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<td>0.048</td>
<td>0.038</td>
<td>0.028</td>
<td>0.020</td>
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<td>C&lt;sub&gt;CLA&lt;/sub&gt;</td>
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<td>0.245</td>
<td>0.203</td>
<td>0.190</td>
<td>0.177</td>
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<td></td>
<td></td>
<td>0.251</td>
<td>0.255</td>
<td>0.260</td>
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</table>

Note: 0) T-10 through T-12 are the artificially-consolidated samples under the various applied stresses 30, 50 and 100 kPa, respectively.
* 0) The value of C<sub>CLa</sub> is the average for T-10 through T-12.
4. EFFECT OF SAMPLE DISTURBANCE ON COMPRRESS INDEX IN PREYIELD STATE C_{Yr}

Fig. 3 shows the relationship between the compression index in the preyield state, C_{Yr}, and the vibration time, t. There is a strong tendency of the value of C_{Yr} increasing with the vibration time.

Schuerenberg (1981) has illustrated that all natural soils possess a soil structure developed during their formation. Nagaraj et al. (1980) have indicated that the initial recompression path for the sensitive clays is always much flatter than the rebound line from a stress level higher than the yield stress. A few in-situ load-settlement data (Fishkin and Crooks, 1988; Fellisier et al., 1979) have indicated a negligible settlement up to the yield stress. Hence, the value of C_{Yr} can be negligible for the field compression path. In fact, this is based on the logic that the mechanical behavior of natural soils is controlled by the soil structure developed during their deposition. Sample disturbance during sampling and handling, however, will cause damage on the soil structure, consequently reducing the resistance of the soil structure. Hence, during the recompression up to the yield stress, the value of C_{Yr} will increase with the decrease in the resistance of soil structure. It has been well documented that sample disturbance increases the compression index in the preyield state, but decreases the compression index in the postyield state (Jamrozowski et al., 1985). When the sample is completely disturbed, the value of C_{Yr} reaches the uppermost value. Its value is equal to that in the remolded state.

Since the magnitude of the compression index during the recompression up to the yield stress is directly affected by sample disturbance, the degree of sample disturbance can be quantitatively defined as Eq. 1.

\[ SD(\%) = \frac{C_{Yr} - C_{Yr}^o}{C_{Yr}^o} \times 100\% \]  

The quantity C_{Yr} represents the compression index in the remolded state. Its value is equal to the slope of the compression path in the ln(1+ε)-lnp plot. The value of SD ranges from 0% being responsible for the field behavior to 100% for the remolded state. Based on the above analysis, the effect of sample disturbance on the compression curves of Ariake clays can be conceptually shown in Fig. 4.

In Fig. 4, the compression path of a remolded soil is plotted as a straight line in the ln(1+ε)-lnp plot. Generally, an ε-lnp compression curve for a remolded soil has been assumed to be a straight line. But it is generally in a shape being slight concave upwards (Burland, 1990), as shown in Fig. 5 after Burland (1990). These curves are alternatively plotted herein in the ln(1+ε)-lnp plot, as shown in Fig. 6. It can be seen that they can be defined very well by a straight line. Hence,
the values of $C_{Ck}$ and $C_{Cm}$ can be measured, only if the oedometer tests are performed on the so-called undisturbed and the remolded samples. Consequently the quantitative measure of the degree of sample disturbance can be obtained using the index of SD.

5. EFFECT OF SAMPLE DISTURBANCE ON CONSOLIDATION YIELD STRESS

For the samples obtained from the same depth at the same site, the field consolidation yield stress should be the same. As shown in Table 2, however, the value of the yield stress varies with the level of sample disturbance. Herein, the oedometer test data of Ariake clay, shown in Table 2, on the samples disturbed to various levels of sample disturbance are used to investigate the effect of sample disturbance on the consolidation yield stress. The highest value of $p_y$ among the investigated samples is considered to be responsible for the lowest degree of sample disturbance, designated $(p_y)_{md}$. Herein, the value of $p_y$ of T-2 is the best value. Fig. 7 shows the relationship between the difference value of $[p_y]_{md} - (p_y)_{md}$ and the degree of sample disturbance SD for the samples T-1 through T-12. It can be seen that the value of $[p_y]_{md} - (p_y)_{md}$ increases linearly with the increase in the degree of sample disturbance SD. Regression analysis gives a linear correlation coefficient of $0.982$, indicating that the reduction of the yield stress measured in laboratory is almost caused by sample disturbance. The best fitted regression line is obtained as Eq. 2.

$$[p_y]_{md} - (p_y)_{md} = -0.881 + 0.340 \times \text{SD}\%$$

The negative intercept indicates that even the best estimate value of $p_y$ is still responsible for some extent of sample disturbance. The corrected yield stress for
sample disturbance, designated $p_d$, corresponds to that without disturbance (i.e., $SD=0\%$). That is, the negative intercept in Eq.2 is responsible for the difference of $[p_{c,obs} - p_{c,0}]$. Hence, the value of the corrected yield stress $p_{c,c}$ for the samples T-1 through T-12 can be obtained by shifting the linear regression line given in Eq.2 parallel to the intercept with the origin. That is, the following equation can be used to obtain the $p_{c,c}$:

$$p_{c,c} = p_{c,0} + B_y \times SD(\%)$$ \hspace{1cm} (6)

The quantity $B_y$ represents the correction coefficient for the yield stress. Its value is equal to the slope of the linear regression line of $[p_{c,obs} - p_{c,0}]$ versus SD. The corrected yield stress $p_{c,c}$ for the samples T-1 through T-12 are summarized in Table 3. It can be seen that the corrected yield stresses are little changed among the samples. This result indicates that the SD defined in this study can quantitatively reflect the sample quality.

6. EFFECT OF SAMPLE DISTURBANCE ON COMPRESSION INDEX IN POSTYIELD STATE

The oedometer test data of T-1 through T-12 for an

<table>
<thead>
<tr>
<th>Table 3. Corrected yield stress for T-1 through T-12</th>
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<tr>
<td>Sample</td>
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<tr>
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<td>T-1</td>
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<td>T-10</td>
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<td>T-11</td>
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<td>T-12</td>
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</table>

Ariake clay with various extent of sample disturbance shown in Table 2 are used to investigate the effect of sample disturbance on the compression index $C_{100}$. The relationship between $C_{100}$ and the degree of SD is shown in Fig.8. It can be seen that the value of $C_{100}$ decreases linearly with the increase in the degree of sample disturbance, similar to the relationship of $p_{c,0}$ versus SD.

7. CONCLUSIONS

The main conclusions obtained from this study are summarized as follows.

1) The specimens obtained from the same depth at the same site are deliberately disturbed by vibration to different level. The oedometer test results indicate that the compressibility during the recompression up to the respective yield stress increases with the increase in the level of sample disturbance. For the field compression path, the compression can be negligible during the recompression. The compression index reaches the uppermost when the sample is completely remolded.

2) A new index for sample disturbance is proposed as $SD(\%) \times C_{100}/C_{100} \times 100\%$. This index can reflect quantitatively the extent of sample disturbance.
3) The consolidation yield stress pcy. and the compression index CCA measured in laboratory decrease linearly with the increase in the degree of sample disturbance.

REFERENCES


New method of coring and cohesion determination on jet grouting improved soft clay

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ABSTRACT: In Shanghai foundation improvement engineering, jet grouting is widely used for saturated clay and silt to reinforce the soil of deep foundation pit and improve the existing buildings' foundation. Since the strength of reinforcement formed cement soil is rather lower, traditional coring methods would easily cause samples’ crushing, besides, test on the special property samples is different with general soil samples. The paper introduces a new method of coring and cohesion determination, which has been successfully used in the engineering.

1. INTRODUCTION OF IMPROVEMENT METHOD

Jet grouting is a new foundation improvement technique, and has been widely applied. In the early 70’s, Japanese firstly put the high pressure jet technique into foundation improvement and cut off wall, which was formed a special foundation improvement technique, as so called C.C.P (Chemical Churning Piling) shown as Fig. 1, a. Later, the jumbo special pile which simultaneously jet high pressure paste and compressive air (Fig. 1, b) and column jet pile (Fig. 1, c) was also developed. After developing, the methods have been widely used in the retaining wall, improvement, water isolation on deep foundation pit, soil improvement, shield structure terminals soil improvement, foundation improvement of existing buildings, bridges and protection to adjacent underground pipes etc..

Jet grouting is generally divided into two stages: first is hole-making stage, which the jet head reached the planned depth; the second is jet and mixing stage, by high pressure jet method, the cement paste and soil is mixed. When the method is used to improve the saturated and soft clay, it forms the high water content mixture of cement and clay.

2. MECHANICAL PROPERTIES OF IMPROVED SOIL

The major material used by jet grouting is cement. When soil is saturated cohesive clay, the strength of mixture of cement paste and clay is rather lower and is uneven, and it’s stress and strain properties is quite different with soil or concrete.

Experiences show that for saturated clay, the improved soil cubic strength is 0.5 – 2.5 MP, which changing range is quite large. The cement soil stress and strain properties is related to soil quality and cement content. Stress and strain properties of cement soil formed by cement and cohesive soil is different with different cement content. Fig. 2 is the stress and strain curves of cement soil with different cement content. Under the lower cement content (5%), sample strength is destroyed when strain reaches high enough, and the curve is rather smooth. On the contrary, under the higher cement content (25%), the sample strength is destroyed when strain is still very low, the curve is go down quickly, as "brittleness".

Fig. 3 is the compressive strength at various lime on the samples directly taken at pile body. The compressive strength has obvious dispersion.

It is shown that the cement soil is different with natural sedimentary soil, rock, mortar and concrete.

1). Strength: Its strength is higher than natural soil, but much lower than rock and concrete;
2). Stress and strain relationship: It is at the position between natural and concrete, has the properties of both "plasticity" and "brittleness";
3). Dispersion: Its strength has larger dispersion.

From above discrition, we know that general coving
3. NEW METHOD OF ON-SITE CORING AND COHESION TEST

1). New method for on-site coring. Since the compressive strength of jet grouting improved soil is only about 1 MPa during coring, by the influence of jet grouting and vibration, the sample is easily smashed, which influence the correct evaluation to improved soil quality.

The new method is to conduct coring before the improved soil hardening, that is, before the hardening of cement soil, putting coring tube [Fig. 4 (2)] in certain depth and close valve (3), and by (1) to draw out the coring tube. By this method, complete samples can be obtained in coring tube.

2). Test method of cohesion. During jet grouting engineering, the cohesion between cement soil and adjacent medium is often needed according to improvement
The new method has been used to test the cement soil compressive strength and the cohesive strength of cement soil and adjacent cement wall for "Shanghai Mansion" in Shanghai, which foundation was improved by jet grouting method. The cement soil strength is 1.5 MPa. The cohesive strength of cement soil and cement wall is 0.4-0.8 MPa.

CONCLUSIONS

By jet grouting method to improve saturated cohesive soil, its mechanical properties of cement soil obtained is difference with natural soil, rocks and concrete. So, the coring and evaluation method for this kind of cement soil should be studied based on its specialties. The paper is only a new attempt on coring and adhesive test, which need to be more experienced later.
REFERENCE

Particularities of geotechnical investigation on loessial soils

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Geoconstruct Design, Bucharest, Romania

ABSTRACT: The geotechnical characteristics of the Romanian loessial soils, especially their particularities of collapsivity, are presented. For an area of 2 km² covered by homogeneous loessial deposits there are compared the laboratory test results obtained on samples collected from boreholes (in sampling tubes) and on block samples made by hand cutting in open pits. It is shown that the use of drive samplers determines unacceptable errors in the evaluation of initial porosity and of the additional settlement by wetting. In the case of loess compaction without humidification the correlations between the variation of dry density ρ₀ and the variation of cone resistance qₖ (CPT) are established.

1 GENERAL

Loessial soils cover about 17% from Romanian territory and are located mostly in the eastern and southern regions as it results from the map shown in Figure 1. These deposits represent the western part of a very large area spanning from East Asia to South Europe. In the above mentioned map the loess has been divided in different classes of collapsivity in terms of calculated or measured settlement after flooding without external load, hₑ. It is generally admitted that loessial deposits come from redeposition in rather dry conditions of wind-carried particles from the North-East during the Quaternary Age. Their thickness varies from 5 to more than 40 m; but usually is comprised between

![Figure 1 Loess distribution over the Romanian territory](image-url)
and electronic microscopy showed that the fine fraction illite is prevailing in the Romanian loess, sometimes associated with more active minerals and the bonding agent is of limonitic or hematitic nature as well as sodium soluble salts, especially for more collapsible soils (Botea et al. 1969). It is also to note the appreciable contains (5 to 10%) of calcium carbonate, both diffused and in concretional form. The natural moisture content of loessial strata is generally low near the surface as a rule between 8 and 12% with a grow trend towards the base, where it reaches 10...18%, also due to ground waterlevel vicinity. The porosity is rather high - usually 47 to 52% at small depths and it generally decreases downwards by several percents. Plasticity usually varies between 12 and 20%. As regarding permeability, the values of $k$ are between $10^{-6}$ and $10^{-2}$ cm/s and an anisotropy is observed, the vertical permeability being by several times greater than the horizontal one.

Loess collapsivity is evaluated in laboratory by double oedometer tests, allowing settlements induced by wetting to be determine at various vertical effective stresses $\sigma'$ (Figure 4). The difference of strain $\Delta_s$ for a given $\sigma'$ (usually $\sigma' = 200...300$ kPa) is a reference value of the collapsivity of a studied loess.

According to Romanian regulations, a soil is considered as collapsible if the value of $\Delta_s$ for $\sigma' = 300$ kPa exceeds 2%; it is to mention that for

### 2. LOESS GEOTECHNICAL CHARACTERISTICS

Mineralogical studies carried out by spectroscopy...
3 EXPERIMENTS CONCERNING THE DISTURBANCE OF LOESS SAMPLES

According to current practice in Romania, the samples considered to be undisturbed are collected from the cohesive soils in sampling tubes with inside diameter of the cutting shoe \( D_h = 100 \ldots 120 \text{ mm} \), length \( L = 300 \ldots 450 \text{ mm} \), inside clearance ratio \( C_h = 1\% \), area ratio \( C_a = 13\% \) and edge angle \( \alpha = 8^\circ \). These parameters are according to the limits considered to be “acceptable” in the international prescriptions (IMSSCS 1981).

In order to evaluate the disturbance rate that appears in the samples of loess collected in such sampling tubes, a comparative study has been performed on an industrial constructions site 2 km² area, located in the southern part of Romania. The layer of loess, homogeneous on the whole site, is 10\(\pm\)12 m thick (Figure 2, Profile A). The results obtained on sampling tubes collected from 117 bordeules have been compared to the ones obtained from block samples (cube shape 20 cm side) hundly collected from 17 open pits.

The statistical processing of the two series of results revealed, as expected, irrelevant differences concerning the values of the natural moisture and plasticity index.

On the other hand, the porosity appears substantially decreased on the samples collected by sampling tubes, as it is shown by the variation of the average values with depth (Figure 5). The dispersion of the experimental values is extremely high for the samples collected by sampling tubes (Figure 6a), while for the block samples taken from the same depths, the variation around the average value is not over \(\pm\)2% (Figure 6b).

The compaction effect of the loessial samples collected by sampling tubes is followed by the substantial decrease of the additional strain on soaks, \( \varepsilon_{so} \), the variation of the average values of additional strain on soaking \( \varepsilon_{so} \) for a vertical effective stress \( \sigma' = 200 \text{ kPa} \) is shown in Figure 7.

These large differences comparatively with the block samples as well as the large dispersion of results (Figure 6a) make impossible the use of results obtained from sampling tubes, even in the situation of the application of some empirical corrections of the porosity values.

4 USE OF CPT IN COMPACTED LOESS

In order to render possible shallow foundation upon
loessial strata the compaction procedures are utilized. It was found out that loess whose porosity decreased by compaction below $n = 40...42\%$ (corresponding to the dry density $\rho_d > 1.65 \text{ g/cm}^3$) no longer exhibits supplementary strains after flooding.

The control of the compacted loess layers by using ground stiffening columns or heavy hammers can be accomplished by comparing the CPT diagrams obtained before and after compaction (Marcus & Culpić 1995). In Figure 8 is shown the correlation established between the increase $\Delta q_c$ of the initial (before compaction) cone resistance $q_c$ and the increase $\Delta \rho_d$ of the initial dry density $\rho_d$ (determined on block samples). A conservative correlation for the Romanian loess with low content (less than 8%) of CaCO_3 can be (Figure 8):

$$\Delta \rho_d = \rho_d \left( 0.25 \frac{\Delta q_c}{q_c} - 0.05 \right)$$

5 CONCLUDING REMARKS

The detailed comparative study performed on a homogeneous loessial deposit, situated on a limited area, indicated a strong effect of disturbance in case
of samples collected by mean of sampling tubes. Due to the large spreading of the results, the introduction of some local empirical corrections (for certain types of loess) is not reliable. In such soils to obtain block samples is necessary for reference. The number of open pits as well as the cost of site investigations can be reduced if these are associated with CPT which allow to evaluate the effect of compaction proceedings in loessial soils.

REFERENCES


Sampling disturbances of soft sensitive clays

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ABSTRACT: Sampling disturbance is unavoidable and hence the laboratory testing must often be on partially disturbed samples. This paper deals with the development of a simple method to assess degree of sample disturbance from the prediction of yield stress due to cementation and comparison of yield stress in compression of partially disturbed sample with reference to a predicted compression path of the clay devoid of any mechanical disturbance. The method uses simple parameters which are normally determined in routine investigations.

1 INTRODUCTION

Although soils, basically are particulate media, the stress to which they are subjected to, the environment in which the deposits are formed and the time that has elapsed, in the geological time scale, have all been recognized as potential factors whose effects are subsumed in the in-situ state of the soft deposits encountered. It is very well known that the equilibrium state of the in-situ deposits are influenced by stress, time and environment. They are neither mutually exclusive processes nor the simple superposition of their influences is tenable in the analysis and assessment of their engineering behaviour.

Although great strides have been made in the in-situ testing and evaluation of engineering parameters for practice, laboratory testing cannot be dispensed with owing to their specific advantages in exercising better control over boundary conditions and simulation of subsequent loading and other environmental conditions. Hence undisturbed sampling of soils is inevitable. If the natural water content of in-situ deposits are such that the liquidity index is greater than unity and the yield stress is only in the range of 25 to 50 kPa, such soft and sensitive deposits are prone to sample disturbance than stiff cemented soils. The specific requirements of assessment of sample quality and evaluation are:

(i) Quantification of the sample disturbance from the laboratory test on undisturbed samples suffered unavoidable mechanical disturbance.

(ii) Ease with which corrections can be effected to other test results obtained by tests on such partially disturbed samples.

2 STATE OF THE ART

Extensive investigations have so far been reported in the literature on different methods of sample quality evaluation based on tests on partially disturbed samples (Hvorslev, 1949, Ladd and Lahte, 1963, Nelson et al., 1971, Okumara, 1971, Anderson and Kolstad, 1979, Nagaraj et al., 1990, Onitsuka, et al., 1995, 1995a, Sibgah and Kinniko 1994, Shogaki 1996 and others). Although sample disturbance, in the absence of macro fabric features, implies destrucrual effects, only very few attempts can be traced to analyses of disturbance to a reference state of the soil not prone to any disturbance.

3 SCOPE OF THE PRESENT ANALYSIS

In this paper the following questions are addressed to develop a simple procedure to assess the sample disturbance from the test on partially disturbed sample.

(i) Is there a need to have a reference state of the clay free from any mechanical disturbance?
relation to which possibility of sample disturbance is critical?

(ii) What is the mechanical disturbance of the sample possible and how to identify the same?

(iii) How quantification of sample disturbance can be attempted?

(iv) How to correct the laboratory test data for sample disturbance in order to modify other engineering properties?

4 BASIC CONSIDERATIONS

The simplest state of a clay - water system devoid of any stress history, time and cementation effects is to have a clay paste with water content at its liquid limit and compress from that state. At liquid limit state the clays with variation of the order of 36 to 159% clays are in equilibrium under the same order of matric suction (5 to 6 kPa), exhibit same order of shear strength (1.7 to 2.5 kPa), and exhibit hydraulic conductivity of the same order 10^{-7} cm/sec (Nagaraj et al., 1993) and hence has the attributes to be a reference state provided the fabric at that state is not prone to any mechanical disturbance. Since the liquid limit state is due to an internal stress field, which is only a function of water holding capacity of the clay, the microfabric would remain the same as long as the water content is unaltered and hence is not prone to any mechanical disturbance. Conventionally laboratory compression paths of clays from water content corresponding to its liquid limit state is devoid of any stress history, secondary time effects and natural cementation.

5 INTRINSIC STATE - EFFECTIVE STRESS RELATION

By undisturbed sampling at various depths, apart from undisturbed samples for testing the information available would be that of the overburden pressure. The information from the sample would be the natural water content and the bulk density. This does not directly provide any information about the structured state of the clay as a result of aging and cementation. Hence the practical way appears to be to examine the dominance of any of the three effects viz., stress, time and cementation responsible

![Diagram](image)

Figure 1. Intrinsic compression and recompression lines and relative disposition of insitu state of undisturbed clays
for the present state of the clay deposit. How far the relation between state of clay in its reconstituted condition and the consolidation stress would provide any inkling to determine the dominance of any one of the three factors merits examination.

A detailed examination of the basic frame work, already developed and discussed (Nagaraj and Srinivas Murthy et al., 1986; Nagaraj et al., 1993; Nagaraj et al., 1994) would help as a reference state. The schematic $\sigma - \log a$ compression paths of clays at different consolidation stress levels are shown in figure 1. The combined functional relation is of the form:

$$\left\{\frac{e}{e_k}\right\} = a - b \log \sigma'_s + c \log \left(\frac{\sigma'_s}{\sigma'_s^*}\right)$$

(5.1)

For the analyses of published and experimental data the value of the constants are $a = 1.122 \cdot 1.123$, $b = 0.234$ and $0.276$ and $c = 0.046$ and $0.042$ for the clays with $w_r$ range of 36 to 159%. For $\sigma'_s = \sigma'_s^*$, the relation reduces to that of the intrinsic compression path. What is depicted by these compression paths is the reference template. What has been considered is only the physico-chemical parameters of the clay and non-clay solid constituents and their synergetics with the pore medium. Nagaraj and Srinivas Murthy (1986) have logically shown that there is gradual grouping of particles into clusters with the increase in stress level. Such a grouping of particles results in a reduction in the operating specific surface and hence the phisicochemical potential of the soil. This reduction is proportional to the original specific surface and during unloading from a particular stress level it is assumed that the clusters will not be dismembered. At engineering level, the original specific surface is indirectly reflected in the water content determined in the liquid limit test. As can be seen in the relation developed (equation 1) this generalized relation involving the void ratio corresponding to water content at liquid limit state is possible encompassing both normally consolidated and overconsolidated mechanically compressed states of the same clay. In a way the average rebound - recompression path of a clay can be regarded as normally consolidated path of a clay of low colloidal activity i.e., low liquid limit water content. Examination of intrinsic compression equation reveals that the ratio of compression to recompression indices, due to mechanical stress histories are of the same order for different reconstituted clays. The specific advantage of this generalization is that the slopes of compression paths prior to and after transitional stress level would aid in the analysis of soft clay behaviour as influenced by any other dominant factors, such as time and cementation subduing the effects of mechanical stress history. Soft clay deposits data from one hundred locations all over the world, collected and collated for reexamination of their classification (Nagaraj et al., 1997a), are plotted on $w/w_r$ plot of figure 1 revealing that practically all clays plot above the intrinsic compression line AB. Particularly clays whose $w/w_r$ greater than unity the microstructure is in metastable state since the stable compatible state would be on the intrinsic compression line. Hence the soft clays of such initial state are prone to mechanical disturbance during sampling and handling.

6. STRUCTURED CLAYS

The reexamination of extensive data of compression paths of structured (fabric and bonding) intact clays (Nagaraj et al., 1991, 1994, 1997, Vatsala et al., 1995) suggest the following hypotheses. At any given void ratio, the applied stress, $\sigma$, is in excess of the stress, $\sigma'_s$, corresponding to the reconstituted compression path by an amount $\sigma = \sigma'_s - \sigma'_s^*$, which is being considered to be from the cementation bonds. Once the capacity of the bonds is reached further increments of effective stress have been premised to be carried by the fabric due to compatible reductions in void ratio. The analysis of the data of the intact clays indicated that the bond resistance remains of the same order (or even increases) with the increase in stress level. It is this bond resistance that contributes to the structured in-situ state (fabric and bonding) of the clay which might suffer mechanical disturbance upon sampling. Sampling disturbance, even in the absence of macro-structural features, implies dominantly degradation of one of the components of the structured state i.e., bonding, the other undisturbed component of structure being fabric which is unlikely to suffer any recognizable disturbance. The slopes of the compression paths during pre and post yield stress in compression test is very different from the preconsolidation pressure while stress history effects are dominant.
7 ANALYSIS OF SAMPLING DISTURBANCE - A QUANTITATIVE APPROACH

Shogaki and Kaneko (1994) and Shogaki (1996) for their detailed study on sampling disturbance assessment, developed a simple laboratory device to impart various degrees of mechanical disturbance. Undisturbed samples after imparting different levels of disturbance were subsequently tested for their strength and compressibility characteristics. Figures 2 and 3 show the compression paths obtained. It can be seen that the degree of rigidity (non-particulate response reflected in yield stress level) decreases as the degree of disturbance decreases. In these plots the predicted compression paths of the clay from its liquid limit water content of the clays (ICL paths) are shown. These paths do not exhibit any yield stress and hence forms a reference compression path. It can be seen that degree of rigidity reflected by the compression path up to yield stress and the value of yield stress itself reduces as the sample disturbance increases and moves inwards towards the completely remoulded path. This suggests the possibility of building up of a scale to assess the degree of sample disturbance. This is similar to the approach suggested for field samples by the first author in 1990 (Nagaraj et al., 1990). But presently the reexamination has been on a more detailed scale and on samples intentionally progressively mechanically disturbed.

The most probable yield stress (at point "a") and sample disturbance (SD) has been determined in relation to the predicted remoulded path free from any disturbance. The paths of reduction in yield stress are shown by arrows along the path in the figures. The degree of disturbance as per the relation indicated (Nagaraj et al., 1990) for each of the paths are shown in the figures and as well the modified values of the compression index and unconfined compression strength values for the test data on partially disturbed samples are also indicated. It can be seen that except for the values underlined for most of the cases there is convergence towards the most probable values corresponding to undisturbed (least disturbance) case.

8 SUGGESTED APPROACH

It is believed that the examination of compressibility and strength data of many undisturbed samples of soft and sensitive clays in the above lines would provide additional credibility to the above approach. Hence based on the basic considerations discussed in this paper and the analysis provided in this paper the following tentative step by step procedure has been suggested for analysis and assessment of sample disturbance and accounting its effects on other engineering properties.

![Figure 2. Analysis of data of Shogaki and Kaneko (1994) for sample disturbance](image1)

![Figure 3. Analysis of data of Shogaki (1996) for sample disturbance](image2)
Data required: in-situ water content and overburden pressure, index properties of the clay, compression path of the undisturbed clay (partially disturbed sample), vane strength data.

Step 1: Assess the location of the $w/w_{opt}$ versus overburden pressure point on the ICL plot in Figure 1. If the point plots above the ICL line, sampling disturbance effects can be dominant.

Step 2: Plot the $e$ vs. $\log \sigma$ path of the sample tested and on this superimpose the predicted compression path of the clay as calculated from equation 1.

Step 3: In fact due to the particulate nature of the clay, bonding in compression bears a relation to the vane shear strain resistance (Figure 4) which in turn is independent of the intrinsic particulate characteristics of the clay reflected in terms of index properties. Calculate the yield stress from the relation $\phi_{y} = 3.2\phi_{s} + 7$ (Nagaraj et al., 1990). This relation has been obtained after applying correction for sample disturbance in the cases where all the information is available. Draw a horizontal line from the in-situ void ratio and mark the yield stress level. Draw a line perpendicular to the predicted compression path up to yield stress level. This defines the direction and magnitude of path of reduction of yield stress due to sample disturbance.

From the relative location where this line cuts the compression path of partially disturbed sample the degree of sample disturbance can be assessed as stated in the figures 3 and 4.

Step 4: Using the value of sample disturbance the compression index and undrained strength value corresponding to undisturbed conditions can be assessed as indicated in the inset table in the figures 2 and 3.

9 CONCLUDING REMARKS

Based on the above basic considerations and the analysis of compressibility of partially disturbed data the following concluding remarks can be made.

Intrinsic Compression Line obtained from the water holding capacity of clays from a reference path for the clay which is not prone to any mechanical disturbance. The in-situ vane strength of undisturbed clay, which is a non-particulate component of structured clay, bears a unique relation with the yield stress of the clay. This relation provides a means of assessing the yield stress corresponding to the undisturbed situation. As sample suffers varied degrees of mechanical disturbance the path of reduction of yield stress is along the path perpendicular to the ICL and passes through the predicted yield stress. Based on the analysis provided in this investigation a simple approach has been suggested for analysis of sample disturbance of the clay from the data usually generated in routine investigations.

REFERENCES


Estimation of soil resistance using rotary percussion drill

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Y. Suzuki & H. Sasa
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ABSTRACT: A new sounding technique, measurement-while-drilling (MWD) logging, using a rotary percussion drill has been devised for quick in-situ soil profiling work. The logging produces a continuous soil resistance index, called \( N_p \)-value, calculated from the drilling energy and a conversion factor which makes the \( N_p \)-value close to the SPT N-value. The conversion factor is found to be affected by difference between conditions with and without percussion. The \( N_p \)-values are compared with the N-values at six sites. The results show good agreement and demonstrate the validity of MWD logging for profiling soil resistance.

1 INTRODUCTION

Several in-situ testing techniques are generally used for investigating soil profiles and soil resistances. Among these techniques, the standard penetration test (SPT) has been widely used for many years in Japan. The SPT has advantages: numerous relations between the SPT N-value and various soil constants have been proposed, and soil samples can be visually inspected. However, the SPT is expensive and time-consuming, and the results can vary among test equipment operators. Moreover, delicate changes and thin layers in the soil may be overlooked because the SPT is performed over rather large intervals and yields discrete data points rather than a continuous data stream.

The authors have developed a soil survey system vehicle (Suzuki et al. 1993), in which a seismic cone penetration test is used in combination with a rotary percussion drill. The system consists of two types of soil survey equipment. One is MWD (measurement while drilling) logging equipment which can quickly provide information indicating the hardness of the ground by measuring its resistance to a high-speed rotary percussion drill. The other is the seismic cone penetration test. A value called the \( N_p \)-value is derived from drilling data and indicates the hardness of the ground. The MWD \( N_p \)-value corresponds to the SPT N-value, and may be useful for determining the distribution of soil layers or the depth and thickness of the bearing stratum, for instance.

The present paper describes the method of MWD logging and subsequent MWD \( N_p \)-value calculation.
Fig. 1. Diagram of MWD logging equipment.

2.2 Survey method

MWD logging is conducted as follows.
1. Move the soil survey system vehicle to the site.
2. Jack up the body of the vehicle, using its outriggers to maintain the rotary percussion drill perpendicular.
3. Attach a rod equipped with a drilling bit to the rotary percussion drill. Insert the rod to the depth at which the soil survey is to begin.
4. Following inputting drilling and data-recording instructions, start automatic drilling while feeding water to the drilling bit.
5. Continue drilling, connect additional rods (45 mm diameter and 1 m long) one by one. The recording device displays in real time not only drilling data (such as drilling rate, thrust force, torque, rotation frequency, percussion energy, number of percussions, water feed rate and water feed pressure) recorded at regular soil sampling intervals (usually 2.5 cm), but also $N_e$ values calculated from equation (1) described in this paper.
6. Remove the rods after the drilling is finished.

2.3 Data acquisition

Fig. 3 shows an example of data acquired by MWD logging, as well as data acquired by a boring survey and $N_e$ values determined from the SPT. In MWD logging, soft ground is drilled by applying thrust force and torque, and hard ground is drilled by adding percussion energy. Owing to restrictions on the oil feed rate, thrust force tends to decrease as drilling rate increases. Also, thrust force tends to increase with depth as the weight of rods is reflected in load meter measurements. Because rotation frequency is held constant, torque increases as the ground becomes harder. If the ground has at least some hardness, percussion energy and percussion frequency can be held almost constant.

Table 1. Capacities of rotary percussion drill

<table>
<thead>
<tr>
<th>Items</th>
<th>Maximum Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drilling rate</td>
<td>5m/min</td>
</tr>
<tr>
<td>Thrust force</td>
<td>220N</td>
</tr>
<tr>
<td>Torque</td>
<td>11N·m</td>
</tr>
<tr>
<td>Rotation frequency</td>
<td>75 r.p.m.</td>
</tr>
<tr>
<td>Percussion energy</td>
<td>2.85 mm²/min</td>
</tr>
<tr>
<td>Number of percussion</td>
<td>2200/h.p.m.</td>
</tr>
<tr>
<td>Water feed rate</td>
<td>40 ℓ/min</td>
</tr>
<tr>
<td>Water feed pressure</td>
<td>4 MPa</td>
</tr>
</tbody>
</table>
3 METHOD OF EVALUATING SOIL RESISTANCE

3.1 Np-value by MWD logging

The Np-value obtained from the SPT is proportional to the amount of percussion energy required to penetrate a unit depth. The Np-value, an index of soil hardness derived from MWD logging data, is calculated from equation (1). This equation is based on the assumption that the Np-value is proportional to the total energy required to drill a unit depth.

\[ N_p = C_p \times \left( \frac{E_{pp} + \beta E_{rr} + \gamma E_{rv}}{V_d} \right) \]  \hspace{1cm} (1)

\[ E_{rv} = T_v \cdot V_d \] \hspace{1cm} (2)

\[ E_{pp} = \tau \cdot T_r \cdot R_p \] \hspace{1cm} (3)

\[ E_{pp} = \frac{P_e}{P_s} \] \hspace{1cm} (4)

where \( E_{rv} \) = thrust energy per unit time (N•m/min), \( E_{pp} \) = rotation energy per unit time (N•m/min), \( E_{pp} \) = percussion energy per unit time (N•min), \( T_v \) = thrust force (N), \( V_d \) = drilling rate (m/min), \( T_r \) = torque (N•m), \( R_p \) = rotation frequency (r.p.m.), \( P_e \) = energy per percussion event (N•min), \( P_s \) = number of percussion events (b.p.m.), \( C_p \) = N-value conversion factor (1/m), and \( \alpha \), \( \beta \) and \( \gamma \) = efficiency coefficients of each energy for drilling.

Equations (2), (3) and (4) represent thrust energy, rotation energy and percussion energy per unit time, respectively. The Np-value is determined by multiplying the total energy (called drilling energy: N•min/m) required to drill a unit depth by the N-value conversion factor \( C_p \). Fig. 4 shows the amounts of energy determined on the basis of measurement data in Fig. 3. Fig. 4 indicates that the thrust energy is so small that it has almost no influence on the Np-value, but the percussion energy, if used, accounts for an overwhelming proportion of drilling energy. To calculate an Np-value, the N-value conversion factor \( C_p \) and efficiency coefficients for drilling \( \alpha \), \( \beta \) and \( \gamma \) need to be estimated.

The efficiency of each energy, i.e., thrust energy, rotation energy and percussion energy, for drilling may not be equal to 1.0, but may differ from one another. However, it is assumed that each efficiency co-
efficient for drilling $a$, $b$ and $c$, is equal to 1.0 in this paper, because it is difficult to estimate these values separately at the present stage of this study.

3.2 N-value conversion factor, $C_n$

The authors believe that values of $C_n$ should differ depending on whether percussion is used, because drilling mechanisms are different.

Fig. 5 presents relations between drilling energy measured by MWD logging and $N$-values from the SPT for three types of soil: clayey soil, intermediate soil such as silt or sandy silt, and sandy soil including gravelly soil. Fig. 5(a) and Fig. 5(b) describe cases in which percussion was not used and was used, respectively. $N$-values from the SPT in Japan were measured with an energy efficiency of 78% (Seed et al. 1985). $N$-values which were originally above 50 or 60 were corrected by converting the original values to 30-cm penetration equivalents.

Values of the $N$-value conversion factor $C_n$ for cases without percussion, shown in Fig. 5(a), tend to vary with soil type. The values are around 0.038 for clayey soil, and around 0.092 for intermediate soil and sandy soil. Since it is difficult to differentiate types of soil by MWD logging, $C_n$-value is chosen to be an intermediate value of 0.06. MWD logging is applied primarily to ground with at least some hardness. It presently generates inaccurate results when applied to soft ground.

The $C_n$-value for cases with percussion, shown in Fig. 5(b), tend to be somewhat smaller for clayey soil as in the case when percussion is not used. The scarcity of data for determining the $C_n$-value for clayey soil made it necessary to assign the value of 0.021, which was derived from data on sandy soil.

4 COMPARISON BETWEEN $N_p$-VALUES AND $N$-VALUES

Fig. 6 compares $N_p$-values to $N$-values determined from the above values for $C_n$, i.e., 0.06 for cases without percussion and 0.021 for cases with percussion. The good correlation between these two sets of values indicates that $N$-values from the SPT can be estimated using $N_p$-values from MWD logging.

Fig. 7 and Fig. 8 compare $N_p$-values calculated from equation (3) and $N$-values obtained from the SPT, for six sites. The values used for $N$-value conversion factor, $C_n$, were 0.06 for cases without percussion and 0.021 for cases with percussion. Fig. 7 shows three sets of data for relatively soft ground, and Fig. 8
Fig. 7. Data of $N_p$-value fitting for soft ground.

Fig. 8. Data of $N_p$-value fitting for hard ground.

shows three sets of data for relatively hard ground.

Fig. 7(a) shows data where silty clay dominated to a depth of about 29 m. Fig. 7(b) shows data where silt layers alternated with sand layers to a depth of about 42 m. Fig. 7(c) shows data with loose sand layers to a depth of about 16 m. All three are data for soft ground in which almost no percussion was used during drilling. The data generally show good agreement with each other. As is clear from Fig. 8(a), however, there is some difference between $N_p$-values and $N$-values for each set of data because $N$-values of soft soil layers at the above-mentioned depths were small. Even so, the three sets of data shown in Fig. 7 demonstrate that MWD logging is satisfactory for practical use, considering that the primary purpose is to determine a general distribution of soil hardness.

Fig. 8(a) shows data where the ground is made up of sand and granite. Fig. 8(b) and Fig. 8(c) show data where the ground is made up of sand and gravel. $N_p$-values and $N$-values agree well in each set of data. For gravel, where $N$-values obtained from the same soil layer vary widely, $N_p$-values obtained at small sam-
pling intervals vary even more widely. In the data shown in Fig. 8(a), MWD logging detected the existence of a soft layer about 1 m thick within a granite layer at a depth of around 15 m. The existence of the soft layer was not detected by the SPT. Thus MWD logging, which measures soil resistance at small sampling intervals, can detect even thin soil layers. In addition, as is evident from Fig. 8(c), MWD can be used in ground where N-values are over 50.

The foregoing demonstrates that Np-values determined from MWD logging data are comparable and similar to N-values determined from the SPT, and may also detect the distribution of soil hardness.

5 APPLICATION OF MWD LOGGING

Fig. 9 shows an example of a survey of a pile-bearing soil stratum by MWD logging. The SPT conducted at the most western of three preliminary survey sites revealed that an intermediate sand layer at around GL-40m, which was expected for a pile-bearing soil stratum, was absent. Therefore MWD logging was performed at this site to determine the exact lengths of piles by obtaining more detailed knowledge of the distribution of soil layers. MWD logging detected that the sand layer in question thinned gradually to the west. This survey led to the conclusion that piles to the east should be born by the sand layer, and piles to the west should be born by a gravel layer lying below the sand layer. Incidentally, MWD logging at ten sites over a total depth of 550 m was accomplished in three days.

MWD logging makes it possible to quickly determine the hardness of soil. The method is particularly effective when more information is required after a boring survey has been done, as in the case shown in Fig. 9.

6 CONCLUSION

This paper proposes a method for determining the Np-value, an index which corresponds to the N-value obtained from the SPT, on the basis of MWD logging data. Differences in measurement method and penetration mechanism between the SPT and MWD logging mean that results obtained from the two methods will seldom concur. Nevertheless, the studies described here prove that appropriate values for an N-value conversion factor, depending whether percussion is used, make it possible to use MWD logging to estimate approximate N-values. The studies also prove that MWD logging is suitable for determining the depth and thickness of a hard soil layer such a pile-bearing soil stratum. Future studies will be directed at explicating efficiency of thrust, rotation and percussion energy for drilling and investigating the use of MWD logging for soil classification.

REFERENCE


Development of a high-quality undisturbed sand and gravel sampling method

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ABSTRACT: Over 25 years, more than sixty sites in Japan, we have been performed sandy and gravelly soils sampling using in-situ freezing sampling method. The purpose of the experiments are under the necessity of precision discussion of the liquefaction resistances and dynamic behavior of the soils. Many tests results have shown that the method can yield high quality undisturbed sand and gravel samples in-situ. The paper describes the history of the development of this sampling method in Japan. The most contemporary way of this sampling method is finally introduced.

1. SPECIAL FEATURE AND OUTLINE
A typical sampling operation is conducted as following process:
Process 1. A borehole is drilled to a depth which covered target soil layer. Then insert a freezer pipe, then supply a freezing medium to freeze the surrounding soil.
Process 2. After freeze the surrounding soil of a freezer pipe, obtain the sample by lift up or coring.
Process 3. Sample is cut, covered and insulated. Through these procedure, the sample is keep in the freeze condition.
Process 4. The sample is carried to a laboratory by using freezer cargo and stored in a freezer.
A special feature of the method is that the stress condition in-situ must be kept through the procedure, during sampling to testing. So that the disturbance of the sample must be quite small, able to discuss the test result precisely.

2. APPLICABILITY
To obtain a undisturbed sample, the following condition is essential.
(a) the soil must be in high saturation condition
(b) the soil include small fine contents
(c) the soil must be under some confining pressure condition
(d) the pore water must be discharged during the freezing
(e) the specimen for the test must be obtained a distance from freezing pipe

It is impossible to freeze the soil with no pore-water. And the strength of the freezing soil is increase by saturation condition. Generally, more than 70% of saturation factor in sandy soil, more than 80% in gravelly soil is necessary for this sampling method in our experience. Condition b. and c. suggest that this method is not able to apply for soils which consist of finer materials. If the large volume change (expansion) is occur, soil structure must be changed. Which means that the great disturbance occurring.
When water freeze, about 10% of volume change (expansion) must be occur. During the freezing, It is necessary to discharge a part of pore water. Which means that the freezing speed must be keep slowly.
(Condition d.)
Several test results suggest that the zone of surrounding the freezer pipe, same as freezer pipe diameter disturbed. The specimens for the test must be obtained from out of the disturbance zone. Fig. 1 shows a result of density measurement of sampled soil. The density near to the freezing pipe, as same as the diameter of the freezer pipe, has low density. Which suggest the zone of near to the freezer pipe is disturbed.

3. DETAILS OF THE DEVELOPMENT
Twenty five years pasted after our first execution. During these days, we performed 36 executions of sand sampling, 16 executions of gravel sampling and 5 executions of a kind of volcanic ash sampling. An improved sand and a wester lent lock sampling are also
conducted.
The year we tried first execution, there were no tube sampler existed, which is commonly used in these contemporary days. The purpose of the first try is to obtain the true value of the density of sand, which was ordered from Dr. Yoshimi and Dr. Hatano. As follow, the detail of the development is described.

3.1 Freezing the ground

The first stage, we used plural freezing pipes. Distributed some freezing pipes to construct some triangle, place the pipe at the edge of each triangles. The sampling was conducted at the center of each triangles. But on this method, pore-water discharge could not occur, to the disturbance was occurred. On this method, cost performance of the freezing was no good either. Today we use single freezing pipe, temperature during the freezing is monitored, control the coolant suction volume.

Most used freezing medium (cooler) is liquid Nitrogen. Because of cost reduction, the mixture of alcohol with dry ice is used to try. This freezing media is in the temperature of -40° C to -60° C.

The point of the use of this freezing medium is how to circulate the coolant. So, the circulation system development is the neck to use this coolant. Today, the circulation system is mostly completed after many try and error. The cost of this freezing medium is much lower than liquid Nitrogen, so the near future, it may use commonly.

3.2 Sampling

The first stage, we used a single tube core barrel with metal tip for the coring. The over coring was conducted surrounding freezing pipe. Thus, we could not lift-up the sampled core with the core barrel. After coring, the sample was lift-up with freezing pipe, using a crane. This way it must take a long time and cost.

Today we use double tube core barrel with core catching system. Obtain a 1 m to 2 m long core sample from freezing zone. And we freeze the only target soil layer. We called this method as "partial freezing method". The first stage we use the drilling mud which temperature of about 1 to 3° C. Use this drilling mud, outer side of the frozen sample was getting thaw. After this, we try to use ethylene glycol or water with calcium chloride for the drilling mud. These drilling mud are cool down by coolant which discharged from inner freezer pipe. Today we keep the temperature of the drilling mud lower than -15° C to -20° C.

The core catching system in the core barrel and coring bit have been developed after several try and error. And we almost complete them today.

Fig. 3 shows the procedure of the method.

4. CONTEMPORARY METHOD USING PARTIAL FREEZING

According to the several development on the hardware of sampling technique, today we often use the method, partial freezing and coring by double tube core barrel. This method just freeze the target soil layer. So it is able to reduce the sampling cost. The following, the outline of the method is described.

Here, we consider the case that there are some soil layer over clay or gravelly soil layer which is the target soil layer.
Fig 2. The procedure of the method (lift-up method)

process 1. An excavation is made for the soil over target soil layer. At this, excavation must be stopped to stay overburden soil, to keep some overburden pressure of objective soil. A guide pipe is installed. Then a borehole is drilled to a depth covered target soil.

process 2. A freezer pipe is installed.

process 3. Freezing medium is supplied to freeze the surrounding soil of freezer pipe.

process 4. Coring is conducted, using double tube core barrel with tip core catcher.

process 5. A sample core is cut using diamond blade cutter.

process 6. A sample core is covered and insulated, then carry to the laboratory under freezing condition. The sample is able to supply for some dynamic laboratory test without any trimming.

Photo 1 to 3 show the process of this sampling method. Photo 4 and 5 show a laboratory test of a sample. Fig 3 shows the procedure of the method.
5. CONCLUSIONS

A method to obtain the high-quality undisturbed sand and gravel sample is developed. We have conducted this method on over 60 sites, obtained over total length of 140 m gravel samples which diameter of 30 cm, and obtained over total length of 800 m sand samples.

We had the attack of South Hyogo Earthquake on 1995. After the earthquake, road and bridge code, which the basis of all civil structures code, is going to change.

We performed this sampling method on 9 sites in wide distributed area in Japan. The results are reflected for the change of the code, especially on the evaluation of liquefaction potential part.

REFERENCE

Y. Yoshimi and M. Hatsumuki, In-situ density measurement of a loose sand, 10th symposium on General Disaster Science, pp.241-242, 1973 (Japanese)
Interpretation of in-situ and laboratory tests on soft clays

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A. Iizuka
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ABSTRACT: Mechanical properties of soft clays, such as deformability and strength, are estimated using data obtained from in-situ and laboratory tests. These test methods were developed through experience gained in the development of geotechnical engineering. The undrained strength, for instance, of a soft clay estimated by using any one of these in-situ or laboratory tests is not usually the same as that estimated by another one of these tests. There are many possible reasons for the apparently different response of the clay to each of these testing methods. The differences caused by factors such as disturbance, stress history, detailed testing procedures, are sometimes surprisingly influential. This paper shows examples of the above mentioned complications encountered in engineering practice. The paper also describes the interpretation of such complications by means of an elasto-plastic constitutive model which was proposed by Sekiguchi and Ohda in 1977.

1 INTRODUCTION

The authors believe that the prediction of the mechanical behaviour of a soft clay deposit requires, in general, four categories of information (Fig. 1):

1) Mechanical properties of the clay (intrinsic properties such as permeability, compressibility, shear resistance, time-dependency etc.): CHARACTERISTIC OF SOIL

2) Stress history experienced by the clay (such as historical fluctuation of the ground water table, geological change in overburden pressure, i.e., the true maximum past consolidation pressure geologically estimated and the apparent pre-consolidation pressure estimated from oedometer tests, aging effect etc.): HISTORY

3) Present stress state (such as depthwise distribution of total and effective overburden pressures, total and effective horizontal stresses, and pore water pressure and shear stresses, if any): PRESENT STATE

4) Human action to be taken on the clay in the future (such as "Are we going to construct a building there, or do we want to place an embankment there, or shall we excavate a tunnel there?", or in case of testing, "What kind of test are we going to do?" etc.): FUTURE

Fig. 1. Four categories of information needed to predict the behaviour of soft clay deposits
2. CONSTITUTIVE MODEL EMPLOYED

The constitutive model employed by the authors originated in experimental work done by Shibata (1963) on the dilatancy of a normally consolidated clay. He carried out constant-$p'$ drained tests on an isotropically consolidated remolded clay and found that the volumetric strain was proportional to the applied $q'$, in which $q'$ denotes the principal stress difference and $p'$ the mean principal effective stress under an axisymmetric stress state. Shibata called it, the proportionality constant, the dilatancy coefficient and used the symbol $D$.

Shibata (1963) further obtained a volumetric strain equation by adding the volumetric strain due to dilatancy to the volumetric strain due to consolidation which is proportional to the logarithm of $p'$. It should be noted that Shibata's equation happened to be similar, in its physical meanings, to the state boundary surface introduced by Roscoe, Schofield and Thurairajah (1963), although they did not explicitly show the equation of the state boundary surface in their paper of 1963. The work by Shibata and that of Roscoe et al. were totally independent.

Shibata's experimental finding was further examined by Shibata and Karube (1965), Karube and Koriha (1966) and Karube and Harada (1967) for three dimensional states of stress. Their major concern was to investigate which is better as the rule governing phenomena such as dilatancy and failure, Mohr-Coulomb criterion or the von Mises criterion. One of the conclusions drawn by them was that, under three dimensional principal stress systems, dilatancy is governed by the von Mises criterion.

Hata and Ohma (1968, 1969) developed the experimental findings of Shibata, Karube et al. into a mathematical form of constitutive equation which is identical, when it is applied to isotropically consolidated clays, with the Cam Clay model proposed by Roscoe, Schofield and Thurairajah (1963). Ohma (1971) proposed a modification between the Cam Clay model, the modified Cam Clay model proposed by Roscoe and Burland (1968), and Hata and Ohma's model from the viewpoints of mathematical structure and physical meaning.

Sekiguchi suggested (personal communication) that Hata and Ohma should include the effect of anisotropic consolidation in a rational manner. Sekiguchi's suggestion resulted in a new model proposed by Sekiguchi and Ohma (1977). The new model took a stress parameter suggested by Sekiguchi in describing the ratio of shear stress to mean principal effective stress. Fig. 2 summarizes the logic used in deriving the new model.
3 CONSTITUTIVE SOIL PARAMETERS

The soil parameters needed to describe the mechanical behaviour of a particular clay are classified into 3 categories: mechanical properties of the clay; stress history experienced by the clay and the present stress state. The parameters required by the inviscid version of the model proposed by Sekiguchi and Ohba (1977) are listed in Table 1. These parameters, except Shibata’s dilatancy coefficient $D$ and the coefficient of earth pressure at rest $K_0$ (and $K_1$), are the same parameters as those of the Cam Clay model. Some of the stress parameters used in the model proposed by Sekiguchi and Ohba are also listed in Table 1.

The experimental specification of those parameters listed in Table 1 requires various kinds of in-situ and laboratory tests to be carried out. It would be practical if the required tests are not too complicated. It would also be practical if the empirical correlation between those parameters and simple index tests is effectively utilized. Fig. 3 summarizes the parameter determination chart originally suggested by Lizuka and Ohba (1987).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formula</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D$</td>
<td>$D = 5.435(G_1 + 0.103G_0)$</td>
<td>from Shibata (1963)</td>
</tr>
<tr>
<td>$D$</td>
<td>$D = 5.435(G_1 + 0.103G_0)$</td>
<td>from Shibata (1963)</td>
</tr>
<tr>
<td>$M$</td>
<td>$M = \frac{\lambda}{\kappa} \frac{1 + \kappa}{3 + \kappa}$</td>
<td>critical state parameter</td>
</tr>
<tr>
<td>$p$</td>
<td>$p = p' + K_0 \lambda$</td>
<td>mean principal effective stress</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>$\kappa = \frac{p}{p'}$</td>
<td>Koerner’s fabric, deviatoric stress</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>$\kappa = \frac{p}{p'}$</td>
<td>principal stress difference</td>
</tr>
<tr>
<td>$\sigma_1$</td>
<td>$\sigma_1 = p' + K_0 \lambda$</td>
<td>principal shear stress</td>
</tr>
</tbody>
</table>

Note: Subscript 0 specifies the value at the time of completion of stress consolidation. Subscript 1 specifies the value at the initial state prior to consolidation activities.
4 UNDRAINED STRENGTHS

The constitutive equation gives the plastic strain increments as functions of current effective stresses and of effective stress increments as shown in Fig. 4. The effective stress conditions for undrained shear are obtained by substituting zero volumetric strain into the constitutive equation. The effective stress condition for the state of failure (critical state) is found from the assumption of zero plastic volumetric strain, while the plane-strain condition is approximately derived from zero plastic strain increment in the direction of intermediate principal stress.

The stress conditions satisfying these three requirements must be the plane-strain/undrained failure condition in which a half of principal stress difference (undrained strengths) is given as a function of the principal stress direction \( \theta \).

Substitution of \( \theta = 0, \pi / 2 \) into the equation of undrained shear strengths under plane-strain conditions specifies the compression strength, the direct simple shear strength and the extension strength of anisotropically consolidated clays.

Substitution of the plane-strain/undrained failure condition into the equations of total stress equilibrium gives the stress characteristic line which converts the plane-strain/undrained strength as a function of principal stress direction \( \theta \) into the plane-strain/undrained strength as a function of the inclination \( \omega \) of the stress characteristic lines. The undrained strength obtained from a constant-volume shear box test is obtained by substituting \( \omega = 0 \) which means that the stress characteristic line is horizontal.

Theoretical reasoning introduced above results in the theoretical estimates of the undrained strengths measured by means of various kinds of testing methods. Table 2 demonstrates thus obtained theoretical estimates in comparison with the measured test results reported by Ladd (1973).

The differences between the undrained strengths of an anisotropically consolidated clay subjected to different types of tests arises not only from mere scatter of test results but also from the intrinsic role of the anisotropic effective stress state at the time of Ko-consolidation prior to undrained shear. The undrained strength anisotropy resulting from Ko-consolidation has been demonstrated often, for instance, by Bjerrum (1972).

The equations in Table 2 need three parameters M, A and Ko. This inversely means that the undrained strength obtained from a particular test method listed in Table 2 can be converted into the undrained strength from any other test method in Table 2, provided two of the parameters are known.

5 SAMPLE DISTURBANCE

The degree of sample disturbance cannot be estimated by purely theoretical reasoning, because sample disturbance is essentially experimental, i.e. it depends on the nature of soil tested, drilling...
methods and skill, sampling techniques, sample transportation, sample extraction methods, sample storage methods, trimming techniques and test preparation procedures. Locality and workmanship are the essential factors influencing the quality of samples prior to testing.

The effect of sample disturbance on the measured undrained strength is probably most serious for the unconfined compression test which has no reconsolidation during the test procedure. The unconfined compression test may be considered as a kind of simplified version of Ko-consolidated triaxial compression test starting from unknown initial effective stress state. The major difference between these two test methods is the magnitude of the pore water pressure; usually measured as positive in triaxial compression tests while usually unknown and negative in unconfined compression tests.

Fig. 5 shows the residual effective stress measured in the so-called “undisturbed” samples taken from 25 sites in Japan. The estimate of Fig. 5 is the residual effective stress ratio \( \frac{\sigma_{dr}}{\sigma_{cr}} \), where \( \sigma_{dr} \) is the effective stress remaining in the “undisturbed samples” actually disturbed to a certain degree. Residual effective stress in the perfect sample (\( \sigma_{cr} \)) is theoretically given as \( \sigma_{cr}^{*} = \frac{\sigma_{cr}}{\frac{1}{3} K_{o} \exp \left(-\frac{P_{w}}{K_{o} \sigma_{cr}}\right)} \) (After Ohta, Nishihara, Iizuka, Morita, Fukagawa and Arai, 1989) is the residual effective stress measured in the samples divided by the theoretical values of residual effective stress in the perfect samples. This ratio indicates the soundness of samples and is called “residual effective stress ratio \( \frac{\sigma_{dr}}{\sigma_{cr}} \)”, see Ohta, Nishihara, Iizuka, Morita, Fukagawa and Arai (1989). Parameters (Table 3) needed to calculate the theoretical values of residual effective stress in the perfect samples are estimated from the plasticity index \( I_p \) in the manner suggested in Fig. 3.

The residual effective stress ratio \( \frac{\sigma_{dr}}{\sigma_{cr}} \) shown by a shaded band in Fig. 5 is theoretically converted into a correction factor for disturbance as demonstrated by Ohta et al. (1989). Experimentally obtained unconfined compressive strengths being multiplied by the correction factor for disturbance as well as other correction factors for the effects of strain rate, confining pressure and stress release must be equivalent to the Ko-consolidated undrained triaxial compressive (KoUC) strengths. Ohta, Nishihara and Morita (1985) derived theoretically the correction factor which converts the KoUC strength into the average undrained strength mobilized along the circular slip surface under an embankment. Multiplication of all of these correction factors results in an overall correction

![Table 2](image)

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Reduced equation for specified test</th>
<th>Brunswick metric chart (( e = 0.20 ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ko-consolidated triaxial</td>
<td>( \sigma_{cr}^{*} = \frac{1}{3} (K_{o} + 2 \sigma_{cr}) \exp \left(-\frac{P_{w}}{K_{o} \sigma_{cr}}\right) )</td>
<td>0.66</td>
</tr>
<tr>
<td>Ko-consolidated plane strain</td>
<td>( \sigma_{cr}^{*} = \frac{1}{3} (K_{o} + 2 \sigma_{cr}) \exp \left(-\frac{P_{w}}{K_{o} \sigma_{cr}}\right) )</td>
<td>0.66</td>
</tr>
<tr>
<td>Direct simple shear</td>
<td>( \sigma_{cr}^{*} = \frac{1}{3} (K_{o} + 2 \sigma_{cr}) \exp \left(-\frac{P_{w}}{K_{o} \sigma_{cr}}\right) )</td>
<td>0.66</td>
</tr>
</tbody>
</table>

(Data reported by Ladd, 1973)

![Table 3](image)

<table>
<thead>
<tr>
<th>( L = 20 )</th>
<th>( L = 40 )</th>
<th>( L = 60 )</th>
<th>( L = 80 )</th>
<th>( L = 100 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M )</td>
<td>1.22</td>
<td>1.02</td>
<td>0.91</td>
<td>0.84</td>
</tr>
<tr>
<td>( A )</td>
<td>0.70</td>
<td>0.58</td>
<td>0.52</td>
<td>0.48</td>
</tr>
<tr>
<td>( K_{o} )</td>
<td>0.52</td>
<td>0.61</td>
<td>0.69</td>
<td>0.78</td>
</tr>
</tbody>
</table>

Fig. 5. Residual effective stress ratio \( \frac{\sigma_{dr}}{\sigma_{cr}} \), where \( \sigma_{cr}^{*} \) is the effective stress remaining in the “undisturbed samples” actually disturbed to a certain degree. Residual effective stress in the perfect sample (\( \sigma_{cr} \)) is theoretically given as \( \sigma_{cr}^{*} = \frac{1}{3} K_{o} \exp \left(-\frac{P_{w}}{K_{o} \sigma_{cr}}\right) \) (After Ohta, Nishihara, Iizuka, Morita, Fukagawa and Arai, 1989)
factor to be applied to unconfined compressive strengths to yield the reasonable factor of safety for embankment stability.

The overall correction factor thus obtained is shown in Fig. 6 as a shaded band while the data points shown by open circles are theoretically evaluated average of plane-strain undrained strength (Su) of mobilized along the circular slip surface at failure divided by a hal of experimentally obtained unconfined compressive strength. If the theoretical reasoning mentioned in this paper is completely correct, these open circles should lie in the shaded band. The open circles in Fig. 6 are much scattered but still seem to be in a reasonable accordance with the shaded band. The data points shown as solid circles in Fig. 6 are the correction factors for unconfined compressive strengths back-calculated from 24 case records of actual failure of embankments. They are also in accordance with the shaded band.

6 CONCLUDING REMARKS

The reasonable agreement of (a) theoretically based shaded band, (b) experimentally based laboratory data (open circles) and (c) failure records in the field (solid circles) plotted in Fig. 7 seems to support the discussions made in this paper. Similar discussion can also be extended to in-situ tests such as field vane tests, see Ohta, Nishihara, Ezuka and Morita (1992). This paper illustrates how constitutive models can assist with the interpretation of complicated soil behavior.

REFERENCES


Sample disturbance in soils – Results from investigations in an overconsolidated marine clay

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Morten Sjursen
Bardal Strømme Consulting Engineers, Oslo, Norway

ABSTRACT: Soil disturbance and distortions during sampling is one of the outstanding topics of discussion amongst both researchers and practitioners in geotechnical engineering. This paper presents results from a study of Glava clay, a moderately overconsolidated, medium stiff marine clay from the Trondheim region in central Norway. In the paper, laboratory test results on samples obtained by various sampling techniques and equipment have been evaluated in order to discover possible variations in sample quality. In the test program, a conventional Geonor $\phi$54 piston sampler, a $\phi$95 mm piston sampler and the $\phi$250 mm Sherwood block sampler were used. The use of block sampling has become more relevant in Norwegian practice in recent years, particularly in soft and sensitive clays.

Various laboratory reference tests have been carried out in order to develop a background for comparison of the various sampling techniques. The most important laboratory tests carried out for the purpose were various index tests, triaxial tests and incremental and continuous loading oedometer tests. The set of strength and deformation parameters indicates differences in sample quality obtained by the different sampling methods.

1 INTRODUCTION

Soil disturbance and distortions during sampling is one of the outstanding topics of discussion amongst both researchers and practitioners in geotechnical engineering. The problem has been well-known for many years, but is nowadays as relevant to discuss as ever. In Norway, a discussion is going on whether the high tempo in field test operations and the organization of the field investigation programs have led to reduced and insufficient sample quality in many projects, particularly in the soft and sensitive marine silts and clays frequently encountered in our country. A part of this discussion is whether the conventional $\phi$54 mm piston sampler produces soil samples of sufficient quality for design purposes, particularly in soft and sensitive marine clays.

To improve the situation, national guidelines for soil sampling have recently been developed in Norway, including specifications for equipment design and maintenance, recommended sampling techniques in various soil types and procedures for handling and storage of obtained samples. The guidelines introduce three different quality classes; undisturbed (Class 1), disturbed (Class 2) and remoulded (Class 3) samples. According to the guidelines, both piston sampling and block sampling may yield Class 1 samples in most Norwegian clays.

The guidelines are meant to serve as a reference manual for field and laboratory personnel, and hopefully the guidelines will contribute to a better sampling process through improved procedures and quality control.

This paper presents results from a study of a moderately overconsolidated, medium stiff marine clay from the Trondheim region in central Norway (Glava clay). This clay has for many years served as one of the research sites for the Department of Geotechnical Engineering, NTNU (formerly NTH), and is by now very well investigated. In this paper, laboratory test results on samples obtained by various sampling techniques and equipment have been evaluated, in order to discover possible variations in sample quality. A set of strength and deformation parameters establish a background for comparison of the various sampling techniques.

The results and test data reported herein may hopefully add to findings in similar research programs on the quality of soil sampling.
2 IN SITU SAMPLING

The aim of soil sampling is to obtain samples for soil identification and various types of laboratory testing. These tests shall provide knowledge of the soil conditions and determine geotechnical properties of the tested deposits. The intention of high quality sampling is to obtain samples where only slight or no disturbance of the soil structure has taken place during sampling, transport and storage. In these samples, the water content and the soil density correspond to that in situ, and no change in constituents or in chemical composition has occurred.

Table 1 summarises the in situ sampling carried out in the Glava clay deposit as a part of this study.

<table>
<thead>
<tr>
<th>Id.</th>
<th>Depth (m)</th>
<th>Visual classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>S4-1</td>
<td>4.00 - 4.80</td>
<td>Homogeneous; no visible disturbance</td>
</tr>
<tr>
<td>S4-2</td>
<td>5.00 - 5.80</td>
<td>Homogeneous; slightly softened</td>
</tr>
<tr>
<td>S4-3</td>
<td>6.00 - 6.80</td>
<td>Homogeneous; slightly softened</td>
</tr>
<tr>
<td>S5-1</td>
<td>5.50 - 6.50</td>
<td>Homogeneous; no visible disturbance</td>
</tr>
<tr>
<td>S5-2</td>
<td>6.50 - 7.50</td>
<td>Homogeneous; slightly softened</td>
</tr>
<tr>
<td>S6-3</td>
<td>7.00 - 7.80</td>
<td>Homogeneous; no visible disturbance</td>
</tr>
<tr>
<td>S6-4</td>
<td>5.80 - 6.50</td>
<td>Homogeneous; no visible disturbance</td>
</tr>
</tbody>
</table>

2.1 Piston sampling

Piston sampling is regarded as a simple, yet qualitatively good method for obtaining undisturbed samples in most clays and also in some silts. This type of sampling is by far the most popular for undisturbed sampling in Norway. However, shearing and retrieval of the sample may cause disturbance to the soil sample, either due to shortcomings in the sampling equipment or the procedures applied or both. Factors related to the equipment may be:

1. Badly prepared cutting edge. The edge should be even with no damages or irregularities.
2. Sampler geometry not according to recommendations (cutter angle, area ratio, inside clearance ratio, outside clearance ratio).
3. Quality and smoothness of inner sample cylinder surface.

Shortcomings in sampling operations in the field may also seriously influence the sample quality.

1. Irregularities in the shearing process (speed, discontinuous shearing, shearing exceeding the predetermined depth).

2. Techniques applied for releasing the sheared sample from the surrounding soil (e.g. tension, torsion).
3. Insufficient waiting time after shearing, before withdrawal from the ground (cause retrieval problems).
4. Vibrations and shocks during transport and sample handling.

In the test program, a majority of the undisturbed soil samples were obtained by the conventional Geonor 404 piston sampler, using both steel and fibre glass cylinders. The larger Geonor 405 mm piston sampler was also used in the field program. This sampler is more cumbersome in use, and has not gained much popularity in Norway. However, this sampling equipment usually provides samples of very high quality.

2.2 Block sampling

In recent years, the use of block samplers has also become more relevant in Norwegian practice, particularly when soft and/or sensitive soil conditions are encountered. In cooperation with the Norwegian Geotechnical Institute (NGI), high-quality block sampling were thus carried out at the Glava site, using NGI’s Ø250 mm Sherbrooke block sampler (Lefthov & Poulin 1979).

The Sherbrooke block sampler is developed for sampling of soil blocks of diameter 250 mm and height approximately 350 mm, see Figure 1. The equipment can be used in most clays, but requires a mud-stabilized or a cased borehole. In this study, a Geotech Prospector hydraulic drilling was used for sampling operations, whereas a 42 mm diameter flat auger was used to prepare the borehole.

Before sampling at a pre-selected level, a borehole was established and stabilized by water/rad. The block sampler was then lowered and positioned at the bottom of the borehole.

It is very important that the borehole is completely clean before the actual block sampling is carried out. Debris left from the augering may disturb the sampling process and influence the shape of the samples. The cutting of the block sample was carried out by slow rotation of the sampler with the three cutting knives active, and water flushing through the nozzles in the bottom end of the sampler. Due to this equipment configuration, a slightly modified sampling procedure was adopted.
The rate of rotation during sampling was 20 - 30 rpm, with only 1-2 minutes of stop rotation at end of the cutting. Before withdrawal of the sampler, the cutting knives are activated so that they become a supporting diaphragm beneath the sample, so that the sample can be hoisted safely to the surface. At the surface, the sample is cleaned and waxed before transport to the laboratory.

Various other field investigations have been carried out at the Glava site, including:

1. Conventional piezocone tests (CPTU)
2. Triple piezocone tests (CPTU)
3. Dilatometer tests (DMT)
4. Field vane tests (FVT)
5. Various other sounding methods

Figure 2 shows typical CPTU recordings at Glava obtained by conventional piezocone tests.

3 LABORATORY TESTS

Various laboratory reference tests were carried out in order to develop a basis for comparison between the various sampling techniques and the corresponding sample quality.

3.1 Index tests

A complete index test program was carried out on all obtained samples. Index test results such as water content, Atterberg limits and soil density indicated the amount of local variations in sample composition, and was used to evaluate the obtained test results. Results from unconfined compression tests gave also relevant information of the sample quality through...
the strain level at failure (\(\varepsilon_u\)) and the initial stiffness (E, G) of the sample.

3.2 Oedometer tests

Incremental (IL) and continuous loading (CL) oedometer tests were carried out to provide information of stress history, deformation and consolidation parameters of the soil. All oedometer tests were carried out on 20 cm³ samples with known density and water content. The following procedure was adopted for the incremental load tests:

1. Porous filters on both ends surfaces provided two-way drainage conditions.
2. Two samples were run with one-way drainage and pore pressure measurement at the base of the sample.
3. 20 minutes load duration on each load step for two-way drainage, 1 hour for one-way drainage.
4. Classical loading procedure with doubling of the vertical load in the subsequent load step.
5. Intermediate load step at 200 kPa, between the 200 and 400 kPa steps to improve determination of the preconsolidation stress.

The continuous consolidation tests (CL) were carried out in the following way:

1. Constant rate of strain (CRS) procedure with measurement of pore pressure at the sample base.
2. Filters and tubes were saturated after initial loading to the effective overburden stress (\(\sigma_{er}\)).
3. Continuous loading up to 1200 kPa with subsequent unloading to \(\sigma_{er}\). 3.3 Triaxial tests

Laboratory CIU and CAU triaxial tests provided values of attraction (\(a\)) and friction (\(\tan\phi\)), as well as information of the stress - strain mobilization of the samples. The amount of water expulsion from the samples during consolidation also indicated the sample quality.

The triaxial tests were mainly run according to the following procedures:

1. Test samples were trimmed to Ø54 mm diameter (block and Ø55 mm samples) and a height of 100 – 110 mm in a humid environment to reduce evaporation from the sample.
2. The samples were equipped with filter paper and filters at both end surfaces.

3. Consolidation of samples overnight for a stress level corresponding to in situ horizontal stresses.
4. Stepwise application of 400 kPa back-pressure before application of vertical stresses.
5. Control of pore pressure response by doing a B-test at the end of the consolidation phase.
6. Shearing of CIU tests the same day, shearing of CAU tests after additional consolidation overnight.
7. Rate of shearing 80 min/mm.

Some samples were run with an alternative procedure with shorter test duration. The main differences between the two procedures were shorter consolidation time, no back-pressure applied and a higher rate of deformation during shearing (20 min/mm).

Table 2 summarizes reference values of index, strength and deformation parameters for the Ghava clay from previous investigations.

<table>
<thead>
<tr>
<th>Soil parameter</th>
<th>Range</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil density, p</td>
<td>1.7-2.0</td>
<td>g/cm³</td>
</tr>
<tr>
<td>Grain density, (\rho_s)</td>
<td>2.71</td>
<td>g/cm³</td>
</tr>
<tr>
<td>Laboratory water content, w</td>
<td>30-40</td>
<td>%</td>
</tr>
<tr>
<td>Liquid limit, (w_l)</td>
<td>30-42</td>
<td>%</td>
</tr>
<tr>
<td>Plasticity limit, (w_p)</td>
<td>20-23</td>
<td>%</td>
</tr>
<tr>
<td>Undrained shear strength, (s_u)</td>
<td>25-50</td>
<td>kPa</td>
</tr>
<tr>
<td>Sensitive, (S_n)</td>
<td>5-10</td>
<td>( )</td>
</tr>
<tr>
<td>Friction, (\tan\phi)</td>
<td>0.5-0.6</td>
<td>( )</td>
</tr>
<tr>
<td>Attraction, a</td>
<td>10-20</td>
<td>( )</td>
</tr>
<tr>
<td>Modulus number, m</td>
<td>14.5-20</td>
<td>( )</td>
</tr>
<tr>
<td>Coefficient of consolidation, (c_v)</td>
<td>3.5-22</td>
<td>m²/yr</td>
</tr>
</tbody>
</table>

* from UCT and FC index strength tests

4 OBTAINED RESULTS

4.1 Results from index tests

Table 3 to 6 summarizes the general index properties for the obtained samples.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (mm)</th>
<th>w_0 (%)</th>
<th>w_1 (%)</th>
<th>w_2 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>4.00 – 4.80</td>
<td>39.8</td>
<td>31.0</td>
<td>32.1</td>
</tr>
<tr>
<td>S2</td>
<td>5.00 – 5.90</td>
<td>43.4</td>
<td>43.4</td>
<td>45.0</td>
</tr>
<tr>
<td>S3</td>
<td>6.00 – 6.80</td>
<td>41.4</td>
<td>37.4</td>
<td>35.6</td>
</tr>
<tr>
<td>S4</td>
<td>5.50 – 6.50</td>
<td>33.4</td>
<td>33.5</td>
<td>45.4</td>
</tr>
<tr>
<td>S5</td>
<td>6.50 – 7.50</td>
<td>40.9</td>
<td>37.7</td>
<td>31.7</td>
</tr>
<tr>
<td>S6</td>
<td>5.40 – 5.55</td>
<td>32.0</td>
<td>31.3</td>
<td>n/a</td>
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<tr>
<td>S7</td>
<td>5.55 – 5.75</td>
<td>31.8</td>
<td>32.9</td>
<td>n/a</td>
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<tr>
<td>S8</td>
<td>5.90 – 5.95</td>
<td>32.0</td>
<td>33.5</td>
<td>n/a</td>
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<tr>
<td>S9</td>
<td>6.05 – 6.15</td>
<td>30.5</td>
<td>31.3</td>
<td>31.7</td>
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</table>

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### Table 4. Atterberg limits

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>w_i (%)</th>
<th>w_p (%)</th>
<th>I (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>54-1</td>
<td>4.00 - 4.80</td>
<td>40.2</td>
<td>20.7</td>
<td>19.5</td>
</tr>
<tr>
<td>54-2</td>
<td>5.00 - 5.80</td>
<td>54.2</td>
<td>24.6</td>
<td>29.6</td>
</tr>
<tr>
<td>54-3</td>
<td>6.00 - 6.80</td>
<td>44.0</td>
<td>22.2</td>
<td>21.9</td>
</tr>
<tr>
<td>55-1</td>
<td>5.50 - 6.50</td>
<td>48.4</td>
<td>21.8</td>
<td>26.5</td>
</tr>
<tr>
<td>55-2</td>
<td>6.50 - 7.50</td>
<td>48.1</td>
<td>22.9</td>
<td>25.2</td>
</tr>
<tr>
<td>B1-3</td>
<td>5.40 - 5.75</td>
<td>36.8</td>
<td>19.7</td>
<td>17.1</td>
</tr>
<tr>
<td>B1-4</td>
<td>5.80 - 6.15</td>
<td>38.8</td>
<td>20.6</td>
<td>18.2</td>
</tr>
</tbody>
</table>

### Table 5. Undrained shear strength, index values.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>( \sigma_{u} ) (kPa)</th>
<th>( \sigma_{u0} ) (kPa)</th>
<th>( \sigma_{m} ) (kPa)</th>
<th>( \gamma )</th>
<th>( % )</th>
</tr>
</thead>
<tbody>
<tr>
<td>54-1</td>
<td>4.00 - 4.80</td>
<td>57.0</td>
<td>40.0</td>
<td>33.8</td>
<td>6.0</td>
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<td>54-2</td>
<td>5.00 - 5.80</td>
<td>42.0</td>
<td>38.0</td>
<td>35.4</td>
<td>8.0</td>
<td></td>
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<tr>
<td>54-3</td>
<td>6.00 - 6.80</td>
<td>39.0</td>
<td>37.0</td>
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<td>3.0</td>
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<td>55-1</td>
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<td>37.0</td>
<td>39.0</td>
<td>38.6</td>
<td>3.5</td>
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<tr>
<td>55-2</td>
<td>6.50 - 7.50</td>
<td>30.0</td>
<td>46.0</td>
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<td>8.0</td>
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<tr>
<td>B1-3</td>
<td>5.40 - 5.75</td>
<td>36.8</td>
<td>46.0</td>
<td>44.3</td>
<td>3.4</td>
<td></td>
</tr>
<tr>
<td>B1-4</td>
<td>5.80 - 6.15</td>
<td>38.8</td>
<td>48.9</td>
<td>48.9</td>
<td>2.9</td>
<td></td>
</tr>
</tbody>
</table>

### Table 6. Soil density and clay content.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>( \rho ) (g/cm³)</th>
<th>( % )</th>
</tr>
</thead>
<tbody>
<tr>
<td>54-1</td>
<td>4.00 - 4.80</td>
<td>1.96</td>
<td>29.0</td>
</tr>
<tr>
<td>54-2</td>
<td>5.00 - 5.80</td>
<td>1.82</td>
<td>60.7</td>
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<tr>
<td>54-3</td>
<td>6.00 - 6.80</td>
<td>1.85</td>
<td>44.1</td>
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<td>55-1</td>
<td>5.50 - 6.50</td>
<td>1.93</td>
<td>42.6</td>
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<td>55-2</td>
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<td>1.84</td>
<td>43.0</td>
</tr>
<tr>
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<td>5.40 - 5.75</td>
<td>1.94</td>
<td>29.3</td>
</tr>
<tr>
<td>B1-4</td>
<td>5.80 - 6.15</td>
<td>1.95</td>
<td>33.8</td>
</tr>
</tbody>
</table>

### 4.2 Results from oedometer tests

The detailed results from the oedometer test program are summarized in Tables 7 and 8. Table 7 identifies the performed tests with index properties of the tested specimens. Table 8 summarizes deformation parameters from the tests, according to the interpretation terminology shown in Figure 3.

### Table 7. Identification and index properties.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>Test</th>
<th>( w_i ) (%)</th>
<th>( \rho ) (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>54-1</td>
<td>4.40</td>
<td>IL</td>
<td>32.2</td>
<td>1.96</td>
</tr>
<tr>
<td>54-2</td>
<td>5.40</td>
<td>IL</td>
<td>44.0</td>
<td>1.82</td>
</tr>
<tr>
<td>54-3</td>
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<td>IL</td>
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<tr>
<td>55-1</td>
<td>5.90</td>
<td>CR5</td>
<td>36.6</td>
<td>1.90</td>
</tr>
<tr>
<td>55-2</td>
<td>6.95</td>
<td>CR5</td>
<td>42.4</td>
<td>1.84</td>
</tr>
<tr>
<td>B1-3</td>
<td>5.50</td>
<td>IL</td>
<td>33.5</td>
<td>1.93</td>
</tr>
<tr>
<td>B1-4</td>
<td>5.75</td>
<td>CR5</td>
<td>32.5</td>
<td>1.95</td>
</tr>
<tr>
<td>B1-4</td>
<td>5.85</td>
<td>CR5</td>
<td>31.5</td>
<td>1.96</td>
</tr>
</tbody>
</table>

### Table 8. Interpreted deformation parameters.

<table>
<thead>
<tr>
<th>Sample</th>
<th>( \sigma_{u} ) (kPa)</th>
<th>( \sigma_{u0} ) (kPa)</th>
<th>( \sigma_{m} ) (kPa)</th>
<th>( \gamma )</th>
<th>( % )</th>
</tr>
</thead>
<tbody>
<tr>
<td>54-1</td>
<td>43</td>
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<td>50</td>
<td>71</td>
<td>18.3</td>
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<tr>
<td>54-2</td>
<td>33</td>
<td>35</td>
<td>50</td>
<td>6.0</td>
<td>60.0</td>
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<tr>
<td>54-3</td>
<td>32</td>
<td>35</td>
<td>35</td>
<td>6.0</td>
<td>60.0</td>
</tr>
<tr>
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<td>63</td>
<td>330</td>
<td>350</td>
<td>7.9</td>
<td>17.3</td>
</tr>
<tr>
<td>55-2</td>
<td>66</td>
<td>460</td>
<td>500</td>
<td>8.8</td>
<td>14.5</td>
</tr>
<tr>
<td>B1-3</td>
<td>59</td>
<td>360</td>
<td>400</td>
<td>9.8</td>
<td>18.2</td>
</tr>
<tr>
<td>B1-4</td>
<td>62</td>
<td>350</td>
<td>390</td>
<td>9.3</td>
<td>18.1</td>
</tr>
<tr>
<td>B1-4</td>
<td>62</td>
<td>370</td>
<td>410</td>
<td>11.4</td>
<td>22.9</td>
</tr>
<tr>
<td>B1-4</td>
<td>65</td>
<td>300</td>
<td>340</td>
<td>7.9</td>
<td>21.6</td>
</tr>
</tbody>
</table>

### Table 9. Identification and index properties for triaxial test samples.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>Test</th>
<th>( w_i ) (%)</th>
<th>( \rho ) (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>54-1</td>
<td>4.33</td>
<td>CAUc</td>
<td>32.1</td>
<td>1.95</td>
</tr>
<tr>
<td>54-2</td>
<td>4.50</td>
<td>CIIc</td>
<td>32.6</td>
<td>1.95</td>
</tr>
<tr>
<td>54-3</td>
<td>5.35</td>
<td>CAUc</td>
<td>35.3</td>
<td>1.91</td>
</tr>
<tr>
<td>55-1</td>
<td>5.90</td>
<td>CIIc</td>
<td>44.0</td>
<td>1.81</td>
</tr>
<tr>
<td>55-2</td>
<td>6.35</td>
<td>CAUc</td>
<td>38.4</td>
<td>1.85</td>
</tr>
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<td>55-3</td>
<td>6.50</td>
<td>CIIc</td>
<td>38.7</td>
<td>1.86</td>
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<td>55-4</td>
<td>5.80</td>
<td>CAUc</td>
<td>34.0</td>
<td>1.93</td>
</tr>
<tr>
<td>55-5</td>
<td>5.95</td>
<td>CIIc</td>
<td>34.6</td>
<td>1.91</td>
</tr>
<tr>
<td>55-6</td>
<td>6.15</td>
<td>CIIc*</td>
<td>43.1</td>
<td>1.81</td>
</tr>
<tr>
<td>55-7</td>
<td>7.05</td>
<td>CIIc</td>
<td>39.1</td>
<td>1.81</td>
</tr>
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<td>6.85</td>
<td>CAUc</td>
<td>40.2</td>
<td>1.81</td>
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<tr>
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<td>CIIc*</td>
<td>37.0</td>
<td>1.80</td>
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<td>CIIc</td>
<td>32.0</td>
<td>1.93</td>
</tr>
<tr>
<td>55-11</td>
<td>5.65</td>
<td>CIIc</td>
<td>33.1</td>
<td>1.93</td>
</tr>
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<td>55-12</td>
<td>5.65</td>
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<td>55-13</td>
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<td>CIIc</td>
<td>32.7</td>
<td>1.93</td>
</tr>
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<td>55-14</td>
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<td>CIIc</td>
<td>32.5</td>
<td>1.93</td>
</tr>
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<td>6.05</td>
<td>CIIc</td>
<td>33.1</td>
<td>1.92</td>
</tr>
</tbody>
</table>

* Simplified NTH procedure
4.3 Results from triaxial tests

Results from the triaxial test program are summarized in Tables 9, 10 and 11. Table 9 identifies the performed tests with index properties of the tested specimens. Table 10 summarizes sample behaviour during consolidation, including the applied consolidation stresses, vertical and volumetric strains during consolidation and finally B-values from the pore pressure response tests. Table 11 summarizes strength and pore pressure parameters from the test, according to the interpretation terminology shown in Figure 4.

Figure 4. Interpretation of strength and pore pressure parameters from triaxial test results.

5 DISCUSSION OF OBTAINED RESULTS

The Glaiva clay deposit is a natural clay deposit with relatively small variations in composition and geotechnical properties. However, the results from index tests clearly reveal some variations in water content, density and clay fraction, probably due to the presence of thin silt lenses in the dominating clay matrix. This is difficult to avoid when performing studies in a natural deposit, but should be kept in mind when evaluating and comparing test results.

Some of the index test results obtained in this study fall outside the typical range from previous investigations (see Table 2), and may be explained by extreme local variations or possible errors made when performing the tests. Results from samples associated with these extreme variations are less emphasized in the evaluation.

In the following, the obtained sample quality is discussed and related to some of the factors revealing the sample quality of clays.

Table 10. Sample behaviour during consolidation.

<table>
<thead>
<tr>
<th>Sample</th>
<th>( \sigma' ) (( \text{kPa} ))</th>
<th>( \Delta \varepsilon' ) (%( \Delta ))</th>
<th>( \varepsilon' ) (%( \Delta ))</th>
<th>( s' ) (%)</th>
<th>( e' ) (%)</th>
<th>B (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>54-1</td>
<td>50</td>
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<td>0.35</td>
<td>0.55</td>
<td>99.6</td>
<td></td>
</tr>
<tr>
<td>54-2</td>
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<td>76</td>
<td>0.32</td>
<td>2.11</td>
<td>98.7</td>
<td></td>
</tr>
<tr>
<td>54-2</td>
<td>56</td>
<td>34</td>
<td>0.70</td>
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<td>99.4</td>
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<td>57</td>
<td>84</td>
<td>0.74</td>
<td>2.45</td>
<td>98.7</td>
<td></td>
</tr>
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<td>0.67</td>
<td>1.37</td>
<td>99.3</td>
<td></td>
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<tr>
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<td>98.6</td>
<td></td>
</tr>
<tr>
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<td>37</td>
<td>0.46</td>
<td>1.30</td>
<td>99.1</td>
<td></td>
</tr>
<tr>
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<td>64</td>
<td>97</td>
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<td>2.34</td>
<td>98.8</td>
<td></td>
</tr>
<tr>
<td>54-5</td>
<td>66</td>
<td>100</td>
<td>0.74</td>
<td>2.33</td>
<td>99.8</td>
<td></td>
</tr>
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<td>95.1</td>
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</tr>
<tr>
<td>54-5</td>
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<td>42</td>
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<td>1.82</td>
<td>99.5</td>
<td></td>
</tr>
<tr>
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<td>75</td>
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<td>0.84</td>
<td>2.04</td>
<td>99.3</td>
<td></td>
</tr>
<tr>
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<td>92</td>
<td>0.52</td>
<td>96.8</td>
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<td></td>
</tr>
<tr>
<td>HH-3</td>
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<td>60</td>
<td>0.41</td>
<td>98.8</td>
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<td></td>
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<td>HH-3</td>
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<td>0.56</td>
<td>99.3</td>
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<td></td>
</tr>
<tr>
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<td>91</td>
<td>0.48</td>
<td>1.50</td>
<td>97.8</td>
<td></td>
</tr>
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<td>100</td>
<td>0.42</td>
<td>98.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HH-4</td>
<td>66</td>
<td>60</td>
<td>0.58</td>
<td>99.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HH-4</td>
<td>66</td>
<td>99</td>
<td>0.48</td>
<td>1.48</td>
<td>98.3</td>
<td></td>
</tr>
<tr>
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<td>99</td>
<td>0.58</td>
<td>99.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Simplified NTH procedure

Table 11. Interpreted strength parameters.

<table>
<thead>
<tr>
<th>Sample</th>
<th>( a' ) (( \text{kPa} ))</th>
<th>( c' ) (( \text{kPa} ))</th>
<th>( \phi' ) (%)</th>
<th>( \beta' ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>54-1</td>
<td>20</td>
<td>46</td>
<td>8.0</td>
<td>0.05</td>
</tr>
<tr>
<td>54-2</td>
<td>20</td>
<td>49</td>
<td>4.5</td>
<td>-0.10</td>
</tr>
<tr>
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<td>18</td>
<td>45</td>
<td>3.7</td>
<td>0.00</td>
</tr>
<tr>
<td>54-2</td>
<td>18</td>
<td>45</td>
<td>2.5</td>
<td>-0.20</td>
</tr>
<tr>
<td>54-3</td>
<td>8</td>
<td>55</td>
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<td>-0.02</td>
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<tr>
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<td>25</td>
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<td>-0.10</td>
</tr>
<tr>
<td>54-4</td>
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<td>50</td>
<td>2.0</td>
<td>0.00</td>
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</tr>
<tr>
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<td>1.2</td>
<td>0.10</td>
</tr>
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<td>54</td>
<td>1.2</td>
<td>0.10</td>
</tr>
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<td>58</td>
<td>1.1</td>
<td>0.10</td>
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<td>0.10</td>
</tr>
<tr>
<td>BH-4</td>
<td>22</td>
<td>50</td>
<td>1.0</td>
<td>0.10</td>
</tr>
</tbody>
</table>

* dilatancy parameter for strain levels \( \varepsilon < 0.1 \% \)

5.1 Evaluation of oedometer test results

The quality of oedometer test samples can best be judged from the sample behaviour in the preconsolidation stress range \( \sigma < \sigma_c \). Holtz et al
have previously shown that the vertical strain level (εv) corresponding to the present effective overburden stress can be used to evaluate the sample quality. This means that the behaviour of the deformation modulus in the preconsolidated stress range, Mce = (Δεv/Δσce)ce, also can serve as an indicator of sample quality. A larger value of εv at εv, will lead to a lower value of Mce, thus indicating sample disturbance. A completely disturbed sample will not show any preconsolidation effects since the soil structure has been totally destroyed.

In this study, the peak value of Mce in the overconsolidated stress range has been used as a quality indicator, values shown in Table 9. Additionally, representative results from samples obtained with block and piston samplers are summarized in Figure 5. Selected samples in the graph refer to approximately the same depth beneath the terrain surface.

All test results from IL oedometer tests are within or close to the parameter range obtained by previous reference tests on φ54 mm samples. Moreover, all modulus curves give a clearly defined preconsolidation stress range, which should indicate acceptable quality for all sample categories. However, results from block samples consistently give higher values of Mce compared to results from φ95 and φ54 mm piston samples, with the latter sample category giving the poorest results. Consequently, there may be an as much as 100 % difference between the highest and lowest Mce - values in the reported tests. Results from CL oedometer tests show a similar trend.

5.2 Evaluation of triaxial test results

In this study, the results from CAU and CIU triaxial tests have been discussed separately. Both groups of results have been evaluated according to the selected criteria of sample disturbance:

1. Volumetric strain εvol during consolidation.
2. Vertical strain εv at failure.
3. Corresponding peak shear stress τmax (undrained shear strength ud).

Previous studies of sample disturbance have shown that the volumetric strain εvol is a good indicator of sample disturbance. A tentative evaluation system based on the level of volumetric strain during consolidation has been developed, see Table 12.

The peak shear stress τmax and the corresponding value of the vertical failure strain εf (i.e. the shape of the stress path) should clearly reveal a disturbed sample structure. A sample with an intact, well-preserved structure and stiffness will reach its peak strength at a significantly lower strain than a disturbed one, and the strength itself may also be influenced.

Table 12. Quality of CAU triaxial test samples based on volumetric strain εvol (Borre 1981).

<table>
<thead>
<tr>
<th>αv</th>
<th>εvol (%)</th>
<th>Depth (m)</th>
<th>Perfect if</th>
<th>Acceptable if</th>
<th>Disturbed if</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>3.0-5.0</td>
<td>0-10</td>
<td>εvol &lt; 3.0</td>
<td>εvol &lt; 5.0</td>
<td>εvol &gt; 5.0</td>
</tr>
<tr>
<td>2.3</td>
<td>1.0-3.0</td>
<td>0-10</td>
<td>εvol &lt; 1.0</td>
<td>εvol &lt; 3.0</td>
<td>εvol &gt; 3.0</td>
</tr>
<tr>
<td>3.2</td>
<td>0.5-1.0</td>
<td>0-10</td>
<td>εvol &lt; 0.5</td>
<td>εvol &lt; 1.0</td>
<td>εvol &gt; 1.0</td>
</tr>
</tbody>
</table>

Selected deviator stress paths from CAU triaxial tests are shown in Figure 6, whereas τ - εv curves are shown in Figure 7. The results clearly reveal the higher quality of the block samples, whereas both φ54 and φ95 mm samples with one exception give lower τmax values and higher values of the failure strain. Based on results from CAU triaxial tests, φ95 mm samples are of slightly better quality than the φ54 mm samples judged from the τmax values, whereas values of failure and volumetric strains are quite

![Figure 5. Vertical effective stress versus oedometer modulus.](image-url)
Figure 6. Summary of stress paths from CAU triaxial tests.

Figure 7. Summary of $\gamma - \varepsilon$ curves from CAU triaxial tests.

Figure 8. Summary of normalized values of the maximum shear stress $\tau_{max}$.

Similar for both groups of piston samples. Results from CIU triaxial tests show similar trends, with block samples showing the smallest volumetric strains as well as the smallest failure strains. Figure 8 shows a summary of normalized values of $\tau_{max}/\sigma_0$ where $\sigma_0 = (\sigma_1 + 2\sigma_3)/3$. The figure shows that CAU and CIU triaxial tests yield approximately the same tendencies in obtained results, both quantitatively and qualitatively. However, the CAU tests give a more distinct and pronounced difference in sample behaviour at failure.

It is important to stress that 54 mm piston sampling, if adopting recommended field and laboratory procedures and using carefully maintained sampling equipment, still should be regarded as a satisfactory method for most practical design work and in most soil deposits. The method is rational and relatively simple, and is believed to defend its position as a state-of-the-art sampling method in Norway also in the future.

Block sampling, on the other hand, requires specially trained personnel, special field equipment and installations, and a more complex and time consuming in situ technique. This sampling method is hence more expensive than ordinary piston sampling. However, block sampling is an interesting method which should be considered used in special projects where the safety may be marginal, for example projects with complex soil conditions and strict requirements involved. More reliable determination of the soil parameters may in turn lead to a more precise and better design, which in the long run may prove more cost-effective.
6 CONCLUSIONS

The Givura clay utilized in this study is taken from a natural clay deposit with some variations in material composition and properties. This should be kept in mind when evaluating the test results and arriving at conclusions. The test results show that samples obtained by the 4250 mm Sherbrooke block sampler perform better in laboratory oedometer and triaxial tests than samples from the smaller 454 and 495 mm piston samplers. The block samples show generally higher stiffness in the preconsolidation stress range, smaller volumetric strains during consolidation, and lower failure strain in triaxial tests than do samples obtained by the piston samplers. Similar studies on other Norwegian clays show similar tendencies, e.g. Karlstrud (1995).

It is important to stress that piston sampling, adopting recommended field and laboratory procedures, as well as using carefully maintained sampling equipment, still is regarded a satisfactory method for most practical design work in most Norwegian fine-grained deposits. Block sampling is however an interesting sampling method to consider in special projects with complex soil conditions, such as sensitive and soft clays. More reliable determination of the soil parameters may in turn lead to a more precise and better design, which in the long run may prove more cost-effective.

7 ACKNOWLEDGMENTS

Special thanks are offered to the Norwegian Geotechnical Institute (NGI) in Oslo for loan of the Sherbrooke block sampler. Particular thanks to Mr. Tom Lune, Mr. Reidar Otter and the laboratory staff at NGI for assistance during the M.Sc. thesis of Morten Sjursen, leading to this paper. Particular thanks are also directed to the entire laboratory staff at the Department of Geotechnical Engineering at NTNU, Trondheim, especially Mr. Jan Jørlund for the field work and Mr. Jonas Finseth for laboratory assistance.

LITERATURE AND REFERENCES


Sandven, R. 1990. Strength and deformation parameters obtained from piezocone tests. Dr. eng. dissertation, Geotechnical department, Norwegian Institute of Technology (NTNUI, presently NTNU). Trondheim.


Estimation of in-situ undrained shear strength using disturbed samples within thin-walled samplers

T. Shogaki & Y. Manuyama
National Defense Academy, Yokosuka, Japan

ABSTRACT: A method for estimating in-situ undrained strength of clays by unconfined compression test with suction measurements was developed. The effect of sample disturbance can be evaluated quantitatively from the results of the unconfined compression test performed on small size specimens, 15 mm in diameter and 35 mm in height, at a strain rate of 1 %/min after the suction of the specimen is measured.

The validity of the proposed method is examined using the method for estimating the in-situ preconsolidation pressure (Shogaki, 1996) and the in-situ rate of strength increase measured by the precision triaxial apparatus using small size specimens (Shogaki, 1997).

1 INTRODUCTION

The undrained shear strength is the most important design parameter for the short term stability problem of clay foundation. The unconfined compressive strength (\(q_u\)) is widely used in Japan for the stability analysis of clay foundation under undrained condition. This is mainly because the average value of \(q_u/2\) well describes the undrained shear strength on failure surface in a ground in total (Matsuo and Asahina, 1976, Shogaki et al. 1997), and in addition to this, the simplicity of the testing procedure always meets the demand in engineering practice.

The variation of the \(q_u\) value is caused by the intrinsic inhomogeneity of a ground and the degree of technician’s working skill through the in-situ sampling to testing in a laboratory. For the approximately 60 factors influencing \(q_u\) values, the effects of sample disturbance were analysed from the results of questionnaires given to senior engineers and researchers with a wealth of practical experience, and the laboratory tests and field investigations were carried out, as much as possible, under the simplified conditions of each factor (Matsuo and Shogaki, 1988). Even so the quantitative interpretation of the effective stress behavior of disturbed samples was not enough in this study.

Shogaki (1995) proposed the method for estimating the in-situ undrained shear strength from the results of the unconfined compression test on a S (Small size) specimen, 15 mm in diameter and 35 mm in height, at a strain rate of 1 %/min after the suction of the specimen was measured. The applicability for practical use, however, is difficult because the sample used in this method was taken by the disturbing equipment (Shogaki and Kaseko, 1994).

In this paper, the estimation of in-situ undrained shear strength using disturbed samples within thin-walled samplers from in-situ sampling is examined using the method proposed by Shogaki (1995). The validity of this method is also examined using the method for estimating the in-situ preconsolidation pressure (Shogaki, 1996) and the in-situ rate of strength increase measured by the precision triaxial apparatus (Shogaki, 1997).

2 SOIL SAMPLES AND TEST PROCEDURES

The undisturbed soil and its remolded samples used in this study were obtained from Holocene marine clays located offshore at Aomori in Japan. Field sampling was performed with a stationary piston sampler with an inner diameter of 75 mm to enhance the quality of the samples. The procedures for preparing the remolded soil samples were as follows:

1) The soil was put into a plastic pouch after the undisturbed specimen was removed. The amount of soil used for remolding almost equals the total amount of five S specimens.

2) The mouth of the pouch was closed after the air was removed by suction.

3) The soil was well kneaded by hand from outside the pouch. The \(q_u\) value of the Aomori clay did not change after a remolding time longer than 5 mins.
The portable unconfined compression apparatus for measuring the suction ($S$) and the unconfined compressive strength ($q_u$) is shown in Fig. 1. This equipment has a height of about 20 cm and a weight of about 8 kg. The load is applied by the linear head and is transmitted through a A.C. or D.C. powered motor. Therefore, since the equipment is portable, it is practical for field use. The unconfined compression test was performed on $S$ specimens at a strain rate of 1 %/min, after the suction of the specimen was measured by using a ceramic disc plate.

The value of $q_u$ was determined to be the maximum stress corresponding to axial strain of less than 15 %. There was no difference in suction and shear strength characteristics between the $S$ and the $O$ specimens, which were tested for soils of $L$ and $G_i$ in the range of 20 % to 100 % and 10 to 1000 kPa, respectively (Shogaki, 1991; Shogaki et al., 1993).

The in-situ rate of strength increase was measured by the precision triaxial apparatus using $S$ specimens (Shogaki, 1997). The specimens were sheared at a strain rate of 1 %/min after the $K_0$ consolidation at a strain rate of 0.005 %/min.

The oedometer tests were performed using a load increment ratio of unity and the duration of loading for each load increment was one day. The values of the compression index ($C$) and the preconsolidation pressure (o') were determined from the e-log o' curve. Mikasa's method was used to identify preconsolidation pressure on the 24-hour e-log o' curve. This method is employed in Japan as the Japanese industrial standard for determining one-dimensional consolidation properties of soils (JIS A 1217-1990). It was shown by Okumura (1974) that there is no difference in the o' values between the Mikasa method and the Casagrande method (1936).

3 A METHOD FOR ESTIMATING IN-SITU UNDRAINED SHEAR STRENGTH

In a method for estimating in-situ undrained shear strength, two parameters were considered. They are the ratio of effective pressure to the maximum value of suction ($S_0$) as Eq.(1) and the ratio ($R_{(q_u)}$) of $q_u$ of other sample to that of $q_u$ of the high quality sample within the tube ($q_u$ in-tube). (1)

$$\sigma'_{op}/S_0$$

in which $\sigma'_{op}$ is the effective stress of the perfect sample subjected to the complete release of total stress and is determined by Eq.(2)

$$\sigma'_{op} = p_o (K_o + A (1 + K_o))$$

in which $p_o$ is the effective overburden pressure, $K_o$ the coefficient of earth pressure at rest, and $A$ the pore pressure coefficient at $q_u$. The ratio of the effective pressure of a perfect sample to suction is equal to one. The $R_{(q_u)}$ of the sheared samples are plotted versus the ratio of effective pressure of Eq.(1). The $R_{(q_u)}$ of the perfect sample for a ratio of effective pressure of one can be extrapolated using the data points in such a plot. For practical use, Shogaki et al. (1993) mentioned, the mean consolidation pressure ($p_{cons}$) instead of $\sigma'_{op}$ such as Eq.(3)

$$p_{cons}/S_0$$

in which

$$p_{cons} = \frac{p_o + 2p_oK_o}{3} = \frac{2}{3} p_o$$

4 ESTIMATING IN-SITU UNDRAINED SHEAR STRENGTH

The results of the unconfined compression

Figure 1. Portable unconfined compression apparatus.
test for various points from the cutting edge of the sampling tube ($D_s$) to the end are shown in Fig. 2. To examine the effect of the site of the specimen for the plane of the sampling tube, one to ten specimens were taken from samples 75 mm in diameter and 45 mm in height as shown in Fig. 3. The symbols in Fig. 3 for each specimen are the same as that of Fig. 2. In Fig. 2, the variation of natural water content ($\varphi$) or the tube sampler is about 10% for the total tube length and about 5% for the same depth. The strain at failure ($i$) has a small value of about 3% in the range of $D_s \approx 200 \sim 600$ mm. The $i$ values of $D_s \approx 70$ mm and $D_s \approx 700 \sim 750$ mm are larger than those of $D_s \approx 200 \sim 600$ mm. This may be caused by sample disturbance from the penetration of the sampling tube and the extraction of soil samples. Therefore, the small value of $S_i$, $\varphi$, and $E_{soil}$ of $D_s \approx 70$ mm and $D_s \approx 700 \sim 750$ mm are caused by the sample disturbance.

To examine the effect of sample disturbance on the suction, Fig. 4 shows the results of measuring suction for three different $D_s$ and reconstituted soil. The procedure for measuring the suction is the same as that reported in other literature (Shigaki et al. 1993). In Fig. 4, zero time is indicated when the water on the top surface of the ceramic disc plate is wiped with paper. The specimen was put on the ceramic disc plate when the piezometer point indicated the expected suction for the specimen. The suction in the ceramic disc plate reached the expected suction of the specimen, a few seconds after the water on the top surface of the ceramic disc plate was wiped with paper. The time in which the suction became constant was a few minutes for each specimen, and this time was independent of sample disturbance. The
suction of the specimen, \( D_i = 221 \text{ mm} \) and remolded soil were 0 and 74 kPa. The suction of the specimen, in which the \( D_i \) value was smaller, was also a smaller value. Figure 5 shows the relationship between the pore water pressure \( (u) \) measured at the base of the specimen, the stress, and the axial strain \( (\varepsilon) \) for the same specimen as in Fig.4. The suction under the shear is represented as the pore water pressure in Fig.5 since the suction under the shear became 0 with small \( S_o \) value. Therefore, the \( u \) values at the \( \varepsilon = 0 \% \) are \( S_o \) values. The \( S_o \), \( q_o \), and \( E_{so} \) values of each specimen are also given in the inset of Fig.5. These values decrease when the soil is disturbed.

Figures 6 and 7 show the relationship between the \( R(q_o) \) and the ratio of effective pressure as determined by Eqs.(2) and (3). The \( K_c \) value was assumed to be 0.5 in Figs.6 and 7. The solid line represents the regression curves for the data by Eqs.(2) and (3).

According to this extrapolation in Figs. 6 and 7, the \( R(q_o) \) of the perfect sample \( (R(q_o)) \) are 1.10 for Eq.(2) and 1.12 for Eq.(3). The \( R(q_o) \) values obtained from Eq.(2) and (3) are almost the same. In-situ \( q_o \) value, \( q_{tot} \text{=103} \) kPa can be obtained from the \( q_{peak} \text{=172} \) kPa at \( D_i = 221 \text{ mm} \) times \( R(q_o) \text{=1.1} \) from Fig.7.

5 ESTIMATING IN-SITU PRECONSOLIDATION PRESSURE

Figure 8 shows the relationship between the \( \sigma' \rho \) values and volumetric strain \( (\varepsilon_v) \) (Shogaki, 1996). The \( \varepsilon_v \) value is defined as

\[
\varepsilon_v = \frac{e_0 - e}{1 + e_0} \times 100 \, (\%)
\]

where the \( e_0 \) is the initial void ratio of the sample. In the in-situ soil under the in-situ effective overburden pressure \( (\sigma'_0) \), there was no sample disturbance, therefore, the \( \varepsilon_v \) value is 0 since the \( e_0 \) value is equal to the \( e_0 \) value. The \( \sigma'_0 \) value decreases with sample disturbance under the same \( e_0 \) value (Schmidt, 1955; Shogaki and Keenon, 1994), and the \( \varepsilon_v \) value increases. Therefore, the \( \varepsilon_v \) value can be employed as an index in order to express the effect of sample disturbance (Shogaki, 1996). The \( \sigma'_0 \) value decreases because the degree of rigidity against compression decreases with sample disturbance. According to this extrapolation in Fig.8, the \( \sigma'_0 \) value of in-situ preconsolidation pressure \( (\sigma'_{p,0}) \) is 239 kPa since \( \varepsilon_v \) value of the in-situ soil is 0 as described in
the explanation of Eq.(5).

The void ratio is plotted against the logarithm of consolidation pressure in Fig.9, and the $\sigma_{w}^{*}$, $C_{u}$ and $t_{w}$ values of each specimen are also given in Table 1. The $\sigma_{w}^{*}$ and $C_{u}$ values decrease when the soil is disturbed. Fig.9 also shows the $e$ vs $\log \sigma_{w}^{*}$ curve of the in-situ condition using the $\sigma_{w}^{*}$ shown in Fig.8 and the $C_{u}$ value obtained in the same manner as the $\sigma_{w}^{*}$ value shown in Fig.8.

The $e_{w}$ value of the in-situ condition was assumed to be that of the specimen of $D_{s}=385$ mm. This assumption is generally accepted for saturated clays. The gradient between $\sigma_{w}^{*}$ and $\sigma_{w}^{*}$ was derived from the swelling index ($C_{u}$) for the specimen of $D_{s}=385$ mm. It was confirmed that the $C_{u}$ values are independent of sample disturbance. It can be judged, by comparing the four different measured $e$ vs $\log \sigma_{w}^{*}$ curves, that the estimated $e$ vs $\log \sigma_{w}^{*}$ curve of the in-situ soil is a reasonable curve.

6 VALIDITY OF PROCEDURE FOR OBTAINING $q_{o,s}$

The coefficient of earth pressure at rest ($K_{o}$) measured by the precision triaxial apparatus using small size specimens (Shogaki, 1997) are plotted against the $\sigma_{w}^{*}$/$\sigma_{w}^{*}$ value in Fig.10, where the $\sigma_{w}^{*}$ value is the effective axial stress in a triaxial test. As shown in Fig.10, the $\sigma_{w}^{*}$/$\sigma_{w}^{*}$ value changes to values of 0.8, 1.8, 2.7, 3.2 and 3.4. The $K_{o}$ value for all specimens decreases from 1.0 with consolidation and becomes about 0.4 at the $\sigma_{w}^{*}$/$\sigma_{w}^{*}$ value greater than 1.4. In the region of $\sigma_{w}^{*}$/$\sigma_{w}^{*}$ < 1.4, the $K_{o}$ values for all specimens are less than 0.4 since the specimens were affected by the effects of aging and the overconsolidated state caused by the stress release and sample disturbance.

Figure 11 shows the relationship between the rate of strength increase ($e$) and the $\sigma_{w}^{*}$/$\sigma_{w}^{*}$ value, where the $e_{w}$ value is the minimum value of the principal stress difference under 1 %/min of the axial strain after the $K_{o}$ consolidation for the same specimens as shown in Fig.10 and the $p$ value is a $\sigma_{w}^{*}$ value. The $e_{w}$/$p$ value tends to decrease with an increase in the $\sigma_{w}^{*}$/$\sigma_{w}^{*}$ value and have come to the constant value of 0.4 at $\sigma_{w}^{*}$/$\sigma_{w}^{*}$ = 1.8. The $\sigma_{w}^{*}$/$\sigma_{w}^{*}$ value obtained from Fig.8 divided by the $\sigma_{w}^{*}$

---

**Figure 9.** $e$ vs $\log \sigma_{w}^{*}$ curves.

**Figure 10.** Relationship between coefficient of earth pressure at rest and $\sigma_{w}^{*}$/$\sigma_{w}^{*}$

**Figure 11.** Relationship between $e_{w}$/$p$ and $\sigma_{w}^{*}$/$\sigma_{w}^{*}$

---

<table>
<thead>
<tr>
<th>$D_{s}$ (mm)</th>
<th>$e_{w}$ (kPa)</th>
<th>$C_{u}$ (%)</th>
<th>$t_{w}$ (%)</th>
</tr>
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<tr>
<td>385</td>
<td>218</td>
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<td>3.6</td>
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<tr>
<td>186</td>
<td>217</td>
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<tr>
<td>53</td>
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<tr>
<td>Refined</td>
<td>37</td>
<td>0.83</td>
<td>17.9</td>
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Table 2. Estimated stress values of $q$ and $\sigma_\nu$.

<table>
<thead>
<tr>
<th>$\sigma_\nu$ (kPa)</th>
<th>$p_r$</th>
<th>$q_{vo}(kPa)$</th>
<th>$R(q_r)$</th>
<th>$q_{vo}$ (kPa)</th>
<th>$\sigma_\nu$ (kPa)</th>
<th>$2\sigma_\nu$</th>
<th>$\sigma_{p/2}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>166</td>
<td>52</td>
<td>172</td>
<td>1.10</td>
<td>189</td>
<td>1.00</td>
<td>1.12</td>
<td>193</td>
</tr>
</tbody>
</table>

1) $\sigma_\nu / \sigma_{pl}$ (K=0.5), 2) $\sigma_\nu / \sigma_{pl}$ (K=0.4), 3) $p_r / \sigma_{ol}$
4) in-situ $c_{ol}$ value (from CHGCE test)

is 1.4. From the regression curve of plots in Fig.11, the $c_{ol}=0.41$ at the value of the $\sigma_\nu / \sigma_{pl}=1.4$ can be seen. This value is a in-situ rate of strength increase ($c_{ol}$).

The estimated $q_{vo}$, $\sigma_{p/2}$, and $c_{ol}$ values are summarized in Table 2. The $q_{vo}$, $\sigma_{p/2}$, and $c_{ol}$ values are 0.40 for $\sigma_\nu / \sigma_{pl}$ (K=0.5), 0.39 for $\sigma_\nu / \sigma_{pl}$ (K=0.4) methods, and 0.40 for the $p_r / \sigma_{ol}$ method. The $q_{vo}$, $\sigma_{p/2}$, and $c_{ol}$ values obtained from three different methods are the same as $c_{ol}(1/4)=0.41$ measured by the precision triaxial apparatus. This has the support of the validity of the proposed $q_{vo}$ and $c_{ol}$ method. The $p_r / \sigma_{ol}$ method is a simple and easy one for practical engineering usage.

7 CONCLUSIONS

The conclusions obtained in this study are summarized as follows:

1) The validity of the proposed method was certified using the method for estimating the in-situ preconsolidation pressure by Shogaki (1996) and the in-situ rate of strength increase measured by the precision triaxial apparatus using small size specimen by Shogaki (1997).

2) The proposed method is a simple and easy one for practical engineering usage.

3) The portable unconfined compression apparatus for measuring the suction and the unconfined compressive strength is practical for field use.

REFERENCES


Applicability of the 45-mm sampler with two chamber hydraulic pistons on clay deposits

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ABSTRACT: A new diameter sampler with two chamber hydraulic pistons (45-mm sampler) was developed by Shogaki (1997). An undisturbed sample of 45 mm in diameter and 500 mm in length can be taken from it. The dimensions of the 45-mm sampler are 60 mm in outer diameter, 1350 mm in total length and 11 kgf in weight. The outer diameter of the 45-mm sampler takes into consideration the diameter (66 mm) of a borehole for the standard penetration test (SPT).

In this paper, an outline of the 45-mm sampler is shown, and its applicability is investigated through the unconfined compression test for samples obtained from two types of samplers. The quality of samples obtained from the 45-mm sampler is higher than those of the sampler normally employed in Japan for Holocene clay deposits in Japan.

1 INTRODUCTION

A new small diameter sampler with two chamber hydraulic pistons (45-mm sampler) was developed by Shogaki (1997), and its applicability is examined through the unconfined compression tests and the standard oedometer tests for samples obtained from Holocene and Pleistocene clay deposits in Mito, Japan (Shogaki and Sudoh, 1997).

In an engineering sense, there is no difference in shear strength and deformation characteristics between the S (Small size) specimen (15 mm in diameter (d) and 15 mm in height (h)) and the O (Ordinary size) specimen (35 mm in diameter and 80 mm in height (h)), which are examined for soils of the plasticity index (I_p) from 17% to 98%, unconfined compressive strength (q_u) from 20 kPa to 1000 kPa, overconsolidation ratio (OCR) from 0.7 to 78 and from different sites (Shogaki, 1991, 1993 and 1995). Ten S specimens can be taken from a sample, 75 mm in diameter and 50 mm in height, sampled from the sampler (75-mm sampler) of 75 mm in inner diameter normally employed in Japan as the Japanese geotechnical standard (JGS) for the method for obtaining undisturbed soil samples using a thin-walled tube sampler with fixed piston. For the ten specimens, the stress-strain curves of specimens very closely located to the sampling tube wall are similar to those of specimens located in the central site of the plane of the sampling tube (Shogaki et al., 1995). This fact suggests the possibility of the development of a small diameter sampler and its applicability in practice. The new 45-mm sampler was developed from this information.

In this paper, an outline of the 45-mm sampler is shown, and its applicability is investigated through the unconfined compression tests for samples obtained from both 45-mm and 75-mm samplers. The quality of samples obtained from the 45-mm sampler is higher than those of the 75-mm sampler for Holocene clay deposits in Sakurada, Japan.

2 OUTLINE AND CHARACTERISTICS OF THE 45-MM SAMPLER

The longitudinal section and the specifications of the 45-mm sampler are shown in Fig. 1 and Table 1, respectively. The specifications of the 75-mm sampler are also shown in Table 1. The outer diameter of the 45-mm sampler takes into consideration the diameter (66 mm) of a borehole for the standard penetration test (SPT). The 45-mm sampler, which solves many of the problems of the 75-mm sampler used by the JGS, has the following advantages:

1. It will contribute to the job of sampling by reducing the workload and increasing efficiency because the 45-mm sampler is 62% lighter in weight and 46% shorter in length than the 75-mm sampler.
2. The 45-mm sampler can take high quality samples for the sample disturbance, since the penetration of the sampling tube can be advanced smoothly because of the lower outer tube.
3. The penetration force of the 45-mm sampler is about twice as powerful as the 75-mm sampler, since the 45-mm sampler has two chamber hydraulic pistons, as shown in Fig. 1. Therefore, the 45-mm sampler is better for hard soils than the 75-mm sampler.

4. When there is a change from sand layer to clay layer during the SPT, an undisturbed clay sample can be taken from the same borehole with the 45-mm sampler.

![Diagram of 45-mm and 75-mm samplers]

Figure 1. Longitudinal section of the 45-mm piston sampler.

<table>
<thead>
<tr>
<th>Sampler</th>
<th>45</th>
<th>75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer diameter (mm)</td>
<td>60</td>
<td>45</td>
</tr>
<tr>
<td>Inner diameter (mm)</td>
<td>45</td>
<td>75</td>
</tr>
<tr>
<td>Total length (mm)</td>
<td>1200</td>
<td>1000</td>
</tr>
<tr>
<td>Mass (kg)</td>
<td>24</td>
<td>24</td>
</tr>
</tbody>
</table>

Table 1. Specifications of the 45-mm and 75-mm samplers.

3 SOIL SAMPLES AND TEST PROCEDURES

The undisturbed soil samples used in this study were obtained from the Holocene clay deposit located on the Holocene plain in Sakai, Japan. The sampling depths (z) of both samplers were minus 12 m and 18 m below the ground surface for the Holocene clay deposit. The sampling was performed at two sites about 3 m apart in horizontal distance at the same elevation.

The lengths of soil sampled from the 75-mm and the 45-mm samplers were 800 mm and 500 mm, respectively. After 10 mm of the soil at the cutting edge of the sampler were cut off, samples with a length of 45 mm were extruded from the sampling tube. To examine the effect of the site of the specimens for the plane of the sampling tube, ten S specimens from the 75-mm diameter sample and four S specimens from the 45-mm diameter sample were taken, as shown in Fig. 2. The sites of each specimen, as shown in Fig. 2, are identical with each sample.

In the unconfined compression tests, the specimens were sheared at a strain rate of 1%/min using a portable unconfined compression apparatus (Shengki, 1991). The value of $q_c$ was determined to be the maximum stress corresponding to axial strain ($\varepsilon_a$) of less than 15%. The secant modulus ($E_s$) is given by $E_s = E / (1 + \varepsilon_s)$, in which $\varepsilon_s$ is the strain at the value of $q_c / 2$. The $E_s$, $q_c$, $F_s$, and $E_s$ are the mean values of water content ($\phi$), $q_c$, $F_s$, and $E_s$.

4 EFFECT OF SAMPLER TYPES ON STRENGTH PROPERTIES

4.1 Effect of the site of the specimen for a longitudinal section of sampling tube on strength properties

The results of unconfined compression tests for all of specimens tested from the 45-mm and 75-mm samplers are shown in Fig. 3 (a) and (b), respectively. The sampling depths ($z$) are minus 12 m for Fig. 3 (a) and (b), and minus 18 m for Fig. 3 (c) and (d). In Fig. 3 (b) and (c), the plots for the specimens of the No.5 and No.6 as shown in Fig. 2 (a) are connected by the solid and dashed lines, respectively.

The $F_s$, $q_c$, and $E_s$ values for the specimens of the $z = -12$ m and $z = -18$ m obtained from both samplers in Fig. 4 (a) and (b), respectively, are plotted against the $z$ values. The $F_s$, $q_c$, and $E_s$ values obtained from both samplers in the region of the $z = -(12.2 ~ 12.3)$ m are similar. In this region, it can be seen that the $F_s$ and $E_s$ values obtained from the 75-mm sampler are about 20% and 7% smaller, respectively, than those of the 45-mm sampler. In Fig. 4 (b) and other regions, except for $z = -(12.2 ~ 12.3)$ m in Fig. 4 (a), there are some differences in $F_s$. However, the $T_s$ values obtained from the 45-mm sampler are smaller than those of the 75-mm sampler.
The \( w_i, \epsilon, E_i \), and \( E_{so} \) values for the specimens of the \( z = -12 \) m and \( z = -18 \) m obtained from both samplers in Fig. 5 (a) and (b), respectively, are plotted against the relative position (\( P_i \)) from the cutting edge of the sampler. In Fig. 5 (a), the \( w_i \) values obtained from both samplers are similar in the region of \( P_i = 0 \sim 50\% \). Therefore, in this region of the \( P_i = 0 \sim 50\% \), the \( w_i \) and \( E_i \) values obtained from the 75-mm sampler are about 17% and 43% smaller, respectively, than those of the 45-mm sampler.

On the other hand, in the case of the \( z = -18 \) m as shown in Fig. 5(b), there are some differences in \( w_i \) under the same \( P_i \) value. However, the \( \epsilon, E_i \) values obtained from the 45-mm sampler are smaller than those of the 75-mm sampler. It can be judged from Fig. 4 to Fig. 5 that the quality of samples obtained from the 45-mm sampler is higher than those of the 75-mm sampler presently employed in Japan for the holocene clay deposit.

The ratios of the \( q_i \) values of each sample to the largest value of those of \( q_i \) (unconfined) are plotted against the \( P_i \) values for the \( z = -12 \) m and \( z = -18 \) m in Fig. 6 (a) and (b), respectively. In the case of \( z = -12 \) m as shown in Fig. 6 (a), the \( q_i / q_{(unconfined)} \) values near the cutting edge and its opposite side decrease for both samplers. The small values of \( q_i / q_{(unconfined)} \) values are caused by sample disturbance. The \( q_i / q_{(unconfined)} \) values for the \( P_i \) values are independent of sampler for types for the both depths.

![Diagram](image)

Figure 3. Results of the unconfined compression tests.
4.2 Effect of the size of the specimen for phase of sampling tube on strength properties values

The $\bar{q}_J / \bar{q}_x$ values are the ratios of the $\bar{q}_J$ value, except for No.5 and No.6 in each sample, to the mean value of those of specimens of No.5 and No.6 in the 75-mm sampler, as shown in Fig. 2 (a). The $\bar{q}_J / \bar{q}_{x,0}$ values are plotted against the distance from sampling depths ($z_J$) in Fig. 7. The $\bar{q}_{x,0}$ values for the 45-mm sampler were employed to that of the 75-mm sampler at the same depths. The $\bar{q}_J / \bar{q}_{x,0}$ values are smaller than 1 for all $z_J$ values in the case of the 75-mm sampler of the $z = 12$ m. It can be inferred that the specimen too closely located to the wall of the tube are disturbed. In the other plots, however the $\bar{q}_J / \bar{q}_{x,0}$ values, are about 1. Therefore, it can be judged from Fig.7 that the variation of the $\bar{q}_J / \bar{q}_{x,0}$ values in these plots effect the variation of inherent $q_y$ values without the sample disturbance caused by the penetration of the sampling tube and the extrusion of soil samplers.

Using the SEM (Scanning Electron Microscope), Shogaki and Matuo (1985) examined the effect of microstructural disturbance caused by the penetration of the sampling tube and the extrusion of soil samples. For undisturbed Yokohama clay with $I_x = 44$ % obtained from...
the 75-mm sampler there was complete remolding at the tube wall, but 2 mm from the wall, the microstructure was not greatly altered from that at the center of tube.

Shogaki and Matsuo (1985).

For other plots except the 75-mm sampler of the z = -12 m, it can be recognized that the results of the unconfined compression test in Fig. 7 coincide well with the results of microstructural disturbance using the SEM obtained by Shogaki and Matsuo (1985).

\( \frac{E_{\text{obs}}}{E_{\text{calc}}} \) are the ratios of the \( E_{\text{obs}} \) value, except for No. 5 and No. 6 in each sample, to the mean value of those of specimens of No. 5 and No. 6 in the 75-mm sampler, as shown in Fig. 8. The \( \frac{E_{\text{obs}}}{E_{\text{calc}}} \) values of the 45-mm sampler of the z = -12 m are larger than those of the 75-mm sampler for the reasons described in Fig. 4(a).

![Graph](image)

**Figure 6. Relationship between \( q_{3}/q_{\text{calc}} \) and \( P_{x} \).**

![Graph](image)

**Figure 7. Relationship between \( q_{3}/q_{\text{calc}} \) and \( D_{x} \).**

![Graph](image)

**Figure 8. Relationship between \( E_{\text{obs}}/E_{\text{calc}} \) and \( D_{x} \).**

![Graph](image)

**Figure 9. Stress-strain curves (z = -12.25 m).**

<table>
<thead>
<tr>
<th>Sampler</th>
<th>( q_{x} ) (kPa)</th>
<th>( E_{\text{obs}} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>45</td>
<td>54~71</td>
<td>1~4</td>
</tr>
<tr>
<td>75</td>
<td>26~61</td>
<td>0.6~1.4</td>
</tr>
</tbody>
</table>

**Table 2.** \( q_{x} \) and \( E_{\text{obs}} \) values of specimens obtained from z = -12.25 m.
All of the fourteen stress-strain curves at the $e = 12.25$ m are shown in Fig. 9, and the range of the $q$ and $E_o$ values of each sampler are also given in the Table 2.

The stress-strain curves and the range of the $q$ and $E_o$ values of specimens obtained from the 45-mm sampler are larger than those of specimens from the 75-mm sampler.

In the case of the Japanese soft clays, the 75-mm sampler gives samples with qualities similar to those of the Laval type sampler [Tanaka & Tanaka, 1995]. Therefore, the 45-mm sampler is also comparable to the Laval type sampler.

5 CONCLUSIONS

A new small diameter sampler with two chamber hydraulic pistons (45-mm sampler) was developed by Shogaki (1997). By using the 45-mm sampler, an undisturbed clay sample can be taken from the same borehole for the standard penetration test. The 45-mm sampler is better for hard soils than the sampler (75-mm sampler) normally employed in Japan. The quality of samples obtained from the 45-mm sampler is higher than that of the 75-mm sampler presently employed in Japan for Holocene clay deposits in Sakura.

REFERENCES


Importance of instrumented drilling

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ABSTRACT: The scope of this paper is directed to address the importance of instrumented drilling for development of sophisticated drilling technology in the future. The purpose of instrumented drilling is clearly defined as represented by the three key words: i.e., Ground Characterization, Numerical Control, and Quality Control. Conceptual designs of the measuring system are discussed for the relevant purpose respectively. In the end, some experimental results of instrumented drilling for rotary core sampling are presented for the cases of the sedimentary soft rock ground.

1 INTRODUCTION

It is a matter of course that drilling and/or boring, particularly rotary core sampling, is undoubtedly of utmost importance in all geotechnical investigation. However, this technology has been left in the hands of practical engineers, and has drawn little attention among academic researchers. Consequently, the current state-of-the-arts on drilling are not ready to be either theoretically outlined or properly presented in an organized manner like textbooks.

The main reasons for this is considered as three-folds:
1. Drilling processes of ground are, in themselves, extremely complicated phenomena which are governed by numerous factors.
2. Observation of essentially deep phenomena and data acquisition at rotating parts of the machine, namely a drilling bit where we are most interested, are very difficult tasks.
3. Drilling is compound technology. Various techniques are designed to be combined depending on the purposes of projects.

Our ignorance on drilling leads to such situation where overwhelming empiricism as well as excessively practice-oriented expertise are prevailing among practical engineers. This, from a different viewpoint, implies the fact that long experience and high-level engineering judgement are required for foremen to be qualified. Furthermore, the technological knowledge has been transferred by old-fashioned apprenticeship rather than by modern education and training programs. Consequently, with the unorganized job environment of severe working condition, the industry now faces serious problems regarding not only aging of experienced foremen but also lack of the successors because of failure to attract young generation.

In order to overcome this problem, it is realized that the drilling technology has to be modernized and organized in a systematic manner, and the state-of-the-arts technology should be summarized. Upon this, labor-saving and standardization have to be promoted so that the drilling job would be considered as a kind of attractive vocations (DDC, 1985). As concrete countermeasures, the following development should be attempted for the future sophisticated drilling technology:

1. To elucidate the drilling phenomenon:
2. To develop the instrumented technology to measure the drilling process in depth;
3. To establish an algorithm to optimize the drilling operation for the relevant purpose.

2 DATA MANAGEMENT IN DRILLING

To develop sophisticated drilling technology, it may be interesting to try to simulate the expertise of well-experienced foremen. The appropriate engineering judgement can only be made with their abilities to collect accurate and effective information and to interpret them properly. This process can be briefly summarized as intelligent data management which is the key issue for the future drilling technology. Hence, the intelligent data management in drilling can be defined as accurate and quantitative measurement of all the information related to the drilling and appropriate engineering judgement based on the collected data.

The fundamental technology for data management is comprised of instrumentation for
accurate measurement, analysis & processing of the measured data, and interpretation & control based on the appropriate analysis. The instrumented drilling is the technology to measure, to transmit, and to record the information related to the drilling; namely drilling parameters.

Herein denoted as drilling parameters are defined as various physical measures on the drilling equipment, thereby related to the drilling operation. The information with respect to drilling work is varied and vast. Some of the representative drilling parameters include, for example, the thrust force and the rotational torque on the drilling rod or at the drilling bit, the rotational rate of the drilling rod, the drilling rate or the rate of penetration, the pressure and the flow rate of the drilling liquid, the vibrations of the core barrel and so forth.

Since many years ago, various attempts have been made of instrumented drilling for some specific purposes. In civil engineering, sounding techniques have been reported to characterize the ground by measuring some drilling parameters. As a whole, these previous techniques, however, have not been welcomed among the practical engineers, and may not be successful. It is partly because the drilling operation became troublesome, the accurate measurement and data processing were not satisfactory, and failure to demonstrate the additional benefit of instrumented drilling to the clients. Nevertheless, recent development in electronics and instrumentation techniques has gradually made this “instrumented drilling” possible so that various drilling parameters can be measured confidently. MWD (Measuring While Drilling) and LWD (Logging While Drilling) have become increasingly popular for deep drilling in the field of exploration for thermal energy and/or natural resources.

The scope of the paper is directed to address the importance of instrumented drilling, particularly for the cases of rotary core sampling as most popular drilling work for site characterization in the field of civil engineering. In the following chapters, conceptual designs of the measuring system are discussed for the relevant purpose of instrumented drilling. Later, a simple instrumentation system developed at CRIEI is presented with some experimental results.

3 PURPOSES OF INSTRUMENTED DRILLING

Main purposes of instrumented drilling are considered as three-fold. These can be adequately expressed by the following key words;

(1) Ground characterization by sounding;
(2) Numerically-controlled drilling;
(3) Quality control of drilling work.

As to make these three terms eye-catching, they can be symbolized as three “C”s which appear in the first letter of the second word respectively, i.e. “Characterization” and “Control”. The appropriate system suitable for each purpose is different, and explained in the following.

Firstly, “Ground Characterization”, sometimes referred as rotary sounding, is the technique to evaluate the physical, mechanical, or seepage properties of the ground from the relevant drilling parameters. This idea to utilize the information collected through the drilling for the purpose of ground characterization is not a new idea at all. Many attempts have already been made to date (Misawa et al., 1978; Chida, 1989; Moxaux, 1978; Pfister, 1983; Zacas et al., 1995). It should be noted that although there are various sounding techniques for soil ground, such as standard penetration test and cone penetration test, there is no appropriate one proposed for rock ground which is too hard to allow penetration of any solid probes.

Secondly, “Numerical Control” stands for the numerically-controlled drilling technique. The drilling machine is computerized so that the most suitable, thus the optimum drilling operation for the ground at work can be achieved automatically regardless of the qualification level of the operators. In many fields of machine operations, a numerical control system is the key technology to achieve automation, e.g., a numerically-controlled lathe. If proper algorithm for optimal drilling operation is established, it will be possible to develop a numerically-controlled drilling machine, which benefits the industry to a great extent by reducing the cost for drilling.

Thirdly, “Quality Control” is the technique to evaluate the quality of drilling works from the...
records of drilling operation. This technique is considered as in alignment with the recent movement to promote standardization of drilling and sampling. This purpose for instrumented drilling is the effective use of measured records of various drilling parameters to evaluate the quality of drilling procedures, drilled wellbores and retrieved cores. Up to now, there has not been any know-how of quality control in the field of drilling. It is probably because drilling works are generally regarded as requiring highly specialized skills and also because there has not been any good methods as to assess the performance of drilling. However, it is widely acknowledged that retrieved cores and field test data are strongly dependent on expertise of the operators, i.e. workmanship. Thus, for modern drilling works, it is of great concern that the works should become more accountable for the clients. One of the plans to achieve this is to present the drilling records to the clients so that they can understand how the drilling works were conducted.

4 SYSTEMS OF INSTRUMENTED DRILLING

4.1 System for “Ground Characterization”

Figure 1 illustrates the fundamental system for “Ground Characterization”.

(1) It is essential to obtain the downhole data to avoid the unwanted depth effect. It is because the data collected at the point of drilling, i.e. the bit, is believed to be most representative of the ground of interest. Thus, instrumentation is recommended to be installed as close to the bit as possible.

(2) Since it is seldom in most civil engineering works that the ground characterization is requested to be made at the time of drilling. Taking account of the difficulty to transmit the data to the ground surface from the downhole instrumentation, the need to monitor the measuring performance is considered as of secondary importance. Therefore, the data can be stored downhole in an appropriate recording device while drilling and collected later together with the core barrel.

4.2 System for “Numerical Control”

Figure 2 illustrates the fundamental system for “Numerical Control”.

(1) Instrumentation has to be made on both the downhole equipment and the surface machine. The downhole instrumentation is to monitor the drilling process, while the instrumentation on the machine is to control its operation.

(2) In order to achieve real-time controlling rather than to operate the machine under a certain preset condition, all the system should respond in real-time to cope with any change of the geological formation as well as the machine condition. The collected downhole data should be transmitted immediately and continuously to the PC placed on the drilling machine to serve judgement if the drilling process is controlled under the optimum condition or out of it. On the other hand, the collected data on the machine should also be transmitted to the PC all the time to monitor the machine operation.

(3) Appropriate control can only be achieved if the suitable algorithm is set for the intelligent judge which is equivalent to the high-level engineering judgement by well-experienced foremen.
### Table 1: Material properties of the alternating strata

<table>
<thead>
<tr>
<th>Layer</th>
<th>Unit weight (kN/m³)</th>
<th>Water content (v%)</th>
<th>Elastic wave velocity (m/s)</th>
<th>Undrained shear strength (kPa)</th>
<th>Current Young's modulus (kPa)</th>
<th>Polyurethane (kPa)</th>
<th>Tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>19.4</td>
<td>25.3</td>
<td>550 (500-600)</td>
<td>180</td>
<td>100</td>
<td>0.31</td>
<td>0.65-1.05</td>
</tr>
<tr>
<td>Layer 2</td>
<td>20.1</td>
<td>20.8</td>
<td>200 (200-250)</td>
<td>120</td>
<td>150</td>
<td>0.21</td>
<td>0.31</td>
</tr>
</tbody>
</table>

1. Ultimate wave velocity measurement under a confining pressure.
2. Triaxial compression (CU) test under the confining pressure of effective confining pressure, 
3. Young's modulus (CU) test under the confining pressure of effective confining pressure.

### 4.3 System for “Quality Control”

Figure 3 illustrates the fundamental system for “Quality Control”. Appropriate documentation must be presented to the clients as the certificate for the requested drilling works.

1. As the first stage, the records of machine operation may be presented. For this, the drilling machine should be instrumented. The data can be transmitted to the PC and later analyzed for interpretation.
2. As the second stage, the records of drilling process may be presented. The data can be transmitted either to the downhole recording device or to the PC on the ground. They can be subsequently analyzed for interpretation.
3. As the third stage, if the numerical controlling is attempted, the records of the control command can also be presented.
4. As the final stage, the clients can be invited on the spot so that they can monitor the time sequence of all or some of the measured data at the time of operation.

### 5 CONDITIONS FOR DESIGN

Depending upon the above-mentioned purposes, the following points should be considered to design the whole instrumentation and data logging system:

1. What are measured?
2. How to measure:
3. How to transmit and to record the data:
4. How to process the data.

Main drilling parameters include the thrust, the torque, the drilling rate, the rate of drill rod revolution, the pressure and the flow rate of drill mud and so forth. Locations of the sensors and/or transducers can be either downhole close to the bit or on the machine. Taking account of the mechanical friction along the rod and the leak/floss at the rod joints, it is desirable to install the sensor as close to the bit as possible to obtain the most representative response of the drilling operation.

Some compromise must be appreciated for the limited spaces and unfavorable conditions for accurate measurement around the bit or in the core barrel. Thus, from the points of versatility, easiness, and cost-effectiveness, it may be stated that the instrumentation on the machine is favorable as the first attempt.

### 6 TYPICAL EXPERIMENTAL RESULTS

At Central Research Institute of Electric Power Industry (CRIEPI) in Japan, basic research has been undertaken to develop technology of instrumented drilling since 1980’s (Yoshida et al., 1989). The typical feature is that the data logged at

![Figure 4: Results obtained in alternating layers](M: mudstone, S: sandstone, T: tiol)
stationary/rotational elements are conveyed through cabling/wireless transmission to the PC respectively so that the drilling process can be monitored on the real-time basis.

So far, several attempts have been made in the field to measure various drilling parameters on the drilling machine above the ground surface. The obtained results have demonstrated successful measurements of some fundamental drilling parameters. These measurements include the thrust force and the rotational torque applied to the drilling rod, the drilling rate, the rotation rate of the drilling rod, and the pressure of the supplied drilling liquid. By the system developed under the cooperation of CREEPI and Kiro-Ibara Consultani Ltd., continuous change of the records can be monitored on the real-time basis on the spot (Kaneko & T royalty, 1999).

6.1 Example 1

Figure 4 demonstrates the typical results obtained in the sedimentary soft rock ground of Pliocene deposits. The ground is typically comprised of alternating layers of fine sandstone and mudstone. Some representative properties are shown in Table 1. The drilling was made perpendicular to the bedding which dips approximately 24° from the horizontal. A 60mm double tube sampler with sleeve (IOS, 1995) and a surface-set diamond bit were used for rotary core sampling. The drilling rate was attempted to be kept constant, approximately 30m/min.

Continuous records of the thrust, the torque, the penetration rate and the pressure of the drilling mud were successfully obtained. In the left column of Figure 4, the geological formation observed from the retrieved core is presented. It should be noted that the change of the thrust and the torque are very sensitive to the geological formation. In the highly indurated mudstone layer, 6 to 8 times higher values of the thrust and the torque were measured as compared to less cemented sandstone. Moreover, in the depths from 14.8m to 15.0m, these forces gradually decrease as the layer becomes more tuffaceous, which clearly demonstrates the sensitive response of the measured forces to the rock type. Even an inclusion of a tiny sandy block of a few centimeters in the thick mudstone layer shown by the arrow in the depth scale at 14.59m can be identified in the recorded data. Furthermore, thin layers of the order of a few centimeters can be confidently identified as demonstrated in the record around 14.5m depth. Hence, it may be justified to conclude that sensing ability by this instrumented drilling system to detect various geological defects in rock ground is expected to be promising.

6.2 Example 2

Figure 5 illustrates the drilling system used at another site of sedimentary soft rock ground. The deposit was a Pliocene thick layer of uniform silstone. In addition to the case described in the previous example, the vibrations of the core barrel
were measured by using accelerometers. This was attempted to evaluate the extent of the damage and disturbance imposed on the core samples. Since the measurement should be made in the depth of drilling and electromagnetic waves do not transmit very well through saturated ground, the measured data should be stored in a memory card coupled immediately above the core barrel. The recorded data were retrieved after the drilling was completed. As a consequence, the data could not be monitored in real-time, and should be analyzed later. Although the measured results have not been analyzed fully, the measurement is considered as successful but still need to be examined carefully.

Figure 6 demonstrates the typical results obtained by the instrumentation on the machine in the same way as the previous case shown in Figure 4. This time, the operational condition was varied in stages. The thrust force was kept constant for lengths of 350m and drilling. It is noted that the recorded data clearly demonstrated the drilling machine was controlled as intended.

6.3 Current study at CRIEPI

For basic research, laboratory tests have also been conducted under the controlled condition (Hattori et al., 1990). The same hydraulic-foam drilling machine as used in the field tests is placed on the cover of the laboratory pit. The model ground, contained in the triaxial cell to apply independent pressures on the vertical and horizontal sides of the specimen, is placed in the bottom of the test pit. Through a series of laboratory experiments using artificial soft rocks, it has been confirmed that the drilling parameters are considered as most promising telltale signs of the ground condition as well as drilling operations.

A new core tube has just been manufactured which is heavily instrumented equipped with CPU for recording data and also for communication with the PC on the ground. This device will enable measurement of the drilling parameters which are more representative of the ground than those measured by the instrumentation on the machine.

7 SUMMARY

The instrumented drilling, in itself, is nothing more than a fundamental technique of data management for the development of sophisticated intelligent drilling technology. For high-level utilization of the information obtained by the instrumented, the related technology must be developed in accordance. These include the technology for real-time data processing and for feedback system to control/optimize the drilling operation. For this purpose, the expertise of sensing, processing and analyzing exercised by well-experienced foremen should be studied and analyzed.

Drilling parameters are strongly dependent on the drilling tools, machine operation, drill management and so forth. To elucidate the complicated process of drilling, it would be of great importance to accumulate experiences and to analyze high quality data collected in a systematic fashion.

In the end, the author wants to emphasize that this technique of instrumented drilling summarized in Figure 7 is a key technology which will lead to the fully automated and non-attended drilling in the future.

REFERENCES


Use of Japanese sampler in Champlain sea clay

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ABSTRACT: The Japanese fixed piston sampler has been tested in a sensitive Champlain sea clay deposit. A series of laboratory tests has been performed on samples obtained with this sampler and the results are compared with those of similar tests performed on samples taken with the Laval sampler used with two different procedures: the conventional and the modified. It appears that the Japanese sampling technique provides sample quality similar to the conventional Laval sampling and slightly lower than the modified Laval technique.

1 INTRODUCTION

In the recent years, the Japanese sampler has been compared to different samplers on various soft soil deposits (Tanaka et al. 1996; Oka et al. 1996; Lune et al. 1997). The Site Investigation Laboratory of the Port and Harbour Research Institute, Japan (PHRI) initiated a research program in collaboration with the Geotechnical Group of Laval University in order to compare the quality of samples taken with the Japanese sampler and the large diameter Laval sampler on a well documented Champlain sea clay deposit: Louisville. This research was mainly initiated for two reasons:

1. The Japanese design standard defines, for practical purposes, the undrained shear strength of fine grained soils as half the peak strength (q) obtained in an unconfined compression test (UC), which is very sensitive to sample quality. It was thus necessary to assess the quality of the Japanese sampler in various clay deposits and in particular in a very sensitive one.
2. The quality of a sample depends on the strain induced by the sampling tube compared to the strain at failure of the soil. Knowing that the strain at failure in sensitive clays is small, sampling from a sensitive deposit represents good conditions for assessing the quality of a sampler.

In order to evaluate its quality, the Japanese sampler was compared with the Laval sampler, used with two different techniques: the conventional one described by La Rochelle et al. in 1981 and a modified procedure (La Rochelle et al. 1997). The testing program involved observation of sample disturbance using X-ray pictures as well as different laboratory tests. This paper compares the data obtained from the two samplers and discusses the geometrical features of sampling tubes which are of primary importance when sampling a soft soil.

2 TESTED CLAY AND TESTING PROGRAM

The site of Louisville, which consists of a 60m thick deposit of high plasticity clay, has been described in numerous theses (Lehdonn 1981) and papers (Hamouche et al. 1995a). This Champlain Sea Canadian clay deposit is very homogeneous, with a constant clay fraction of 80%. The average plasticity index, I_p, is 45% and the natural water content decreases slightly with depth, from 56% below the 1.8m thick weathered crust to 60% at 20m. The value of the liquidity index I_L also decreases from 1.6 to 0.9 between 1.8 and 20m.

The undrained shear strength measured with the field vane increases linearly with depth, from 18 kPa at 1.8m to 70 kPa at 20m. The overconsolidation ratio, OCR, decreases from 5 at 1.8m to 2 at 20m. The average sensitivity, S, determined with the Swedish fall cone, is about 20.
Table 1: Dimensions and features of the samplers

<table>
<thead>
<tr>
<th></th>
<th>length (mm)</th>
<th>diameter (mm)</th>
<th>thickness (mm)</th>
<th>apex angle</th>
<th>area ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laval sampler</td>
<td>600</td>
<td>200</td>
<td>5</td>
<td>5º</td>
<td>10</td>
</tr>
<tr>
<td>Japanese sampler</td>
<td>1000</td>
<td>75</td>
<td>1.5</td>
<td>6º</td>
<td>8</td>
</tr>
</tbody>
</table>

The Japanese sampler was used in Louisville to take samples every meter from 6 to 20m. The tubes were carried to the laboratories of Université Laval and PPRR where the samples were pictured using X-rays, extruded and immediately waxed. Different laboratory tests were performed on these samples and the test results were compared with similar tests performed on samples taken with the Laval sampler. The comparison was restricted to 3 depths (around 7, 12 and 20m) which are the depths chosen by La Rochelle et al. (1997) for evaluating the modified sampling procedure. After an examination of the samples using X-ray pictures, different triaxial and consolidation tests were carried out; KS consolidated undrained compression tests, unconsolidated undrained compression tests, unconfined compression tests as well as conventional oedometer tests and constant rate of strain oedometer tests (CRS).

3 DESCRIPTION OF SAMPLERS

The Laval sampler, shown in Fig.1a, has been developed by La Rochelle et al. (1981). In a clean borehole, prepared by drilling and using a bentonite mud, the sampler is lowered and the sampling tube is pushed into the ground. An overcoring system is then activated to remove the soil around the sampling tube. Finally the soil at the base of the sampling tube is sheared by twisting the rods from the ground surface and the whole system is retrieved. In 1994, the Geotechnical Group of Laval University examined the possibility of improving this procedure. Two aspects were considered: 1) to change the low density bentonite mud initially used during the drilling by a denser bentonite-barite mud; 2) to proceed to the overcoring system just about 5cm behind the cutting edge of the sampling as it is being pushed down into the soil in order to facilitate the outward displacement of the soil below the tube. A comparative study performed by La Rochelle et al. (1997) shows that the modified sampling technique provides better quality samples than the conventional one described in 1981.

Fig. 1: Working principle of a) Laval sampler; b) Japanese sampler
4 X-RAY OBSERVATIONS

A series of X-ray pictures of the Japanese sampling tubes was taken before the extrusion of the clay from the tubes. The waxed samples obtained with the modified Laval sampling procedure were also pictured using X-rays. The pictures show that the tube sampled using the Japanese technique are somewhat affected by cracks. The visible cracks are horizontal and seem to cover the whole section of the tube. The cracks observed in samples obtained with the modified Laval procedure are exclusively radial, starting at the edge of the sample and are between 1 and 3cm long. The central part of the Laval sample, at least when the modified technique is used, seems to be not affected by any visible cracks.

5 TRIAXIAL TEST RESULTS

5.1 CK, UC tests

At the depths considered in the present study, the coefficient of earth pressure at-rest is close to unity (Hamouche et al. 1995b), K’ consolidation is then equivalent to an isotropic consolidation. A total number of 22 CIU tests were performed at Laval University on samples taken with the Laval sampler (conventional and modified) and with the Japanese sampler. The samples were consolidated under an isotropic effective stress equal to the vertical effective overburden pressure, and then sheared at a strain rate equal to 1.4 × 10^{-6} s^{-1}. Two aspects were examined in order to compare sampling quality: the volumetric strain during consolidation and the peak strength conditions.

5.1.1 Consolidation volumetric strain

Lunne et al. (1997) proposed a criterion for evaluating sample quality based on the value of the ratio Δσ_v after consolidation to in situ stresses in a triaxial cell. Figure 2 presents a profile of the volumetric strain for Louisville clay after consolidation to σ_v’. For the three investigated depths, the volumetric strain increases with depth and, according to the classification proposed by Lunne et al. (1997), the quality is excellent to very good for all samples which is the highest quality class in the Lunne et al.’s evaluation. However, the modified Laval sampling method seems to provide better quality samples with the lowest consolidation volumetric strains.

5.1.2 Shear behaviour

During compression, the failure occurred at typical axial strains between 0.5 and 1.3% and equal to 0.8 in average, as shown on Fig. 3a. The lower limit of the strain range is represented by the values obtained with the modified Laval sampling. Figure 3b presents a profile of the peak strength values. The modified Laval sampling gives slightly higher undrained shear strength, particularly for the deepest samples. The other techniques give in average comparable results.

5.2 Unconfined compression tests

A series of unconfined compression tests were also performed. The samples were set in the triaxial cell without any confinement and compressed at a strain rate of 1%min. Figure 4 presents a profile of the undrained shear strength. There is no clear trend and the scatter in the results does not allow any comparison in sampling quality.

The scatter is thought to be due to the following reasons:
1) the tests were performed on samples from different locations inside the sampling tube. It has been shown that samples from the middle third of the tube are less disturbed than samples from the upper and lower third parts of the tube (La Rochelle et al. 1973, 1981). The results shown here may reflect this fact.
2) While sampling and extruding from the tube a brittle material such as Louisville clay, a micro-crack network may develop in the sample due to stress release. This network could not be seen with the x-ray pictures but could control the behaviour of the sample under unconfined compression tests, creating a large scatter in the results.

On Fig. 4 are also presented results obtained from unconsolidated undrained compression tests (UU tests). In these tests, a confining pressure is applied reducing the effect of the cracks and also reducing the scatter of the test results. It can be observed that the strength measured in UC or in UU tests is generally smaller than that measured in CIU tests, even if these latter tests are performed at slower strain rate.

6 OEDOMETER TEST RESULTS

Series of conventional oedometer tests as well as CRS tests have been performed at 3 depths (around 7m, 12m and 20m).
The recompression index $c_r$ is a better parameter for comparing sampling quality since it reflects the degree of disturbance. Recompression index values obtained for the three tested depths show a little scatter but a clear tendency for $c_r$ to increase with depth forming an average value of 0.03 at 7m to 0.05 at 20m. In average, the conventional Laval and Japanese sampling techniques give values which are very close and slightly higher than the modified Laval sampling technique.

A series of CRS tests in which the knee in the curve is generally better defined has also been performed for the same depths. The tests were performed according to the Japanese standard with a strain rate of 0.02%/min.

The yield stress obtained with the Laval sampler is clearly higher, confirming the results based on the recompression index obtained from standard oedometer tests.

7 DISCUSSION

From the present comparative study, it appears that the Japanese sampling standard gives sample quality comparable to that obtained with the Laval

Fig 3: C/U tests results a) strain at failure; b) undrained shear strength
edge angles (Table 1). From previous studies (Tanaka et al. 1996, Oka et al. 1996, Lunne et al. 1997), it also appears that the Japanese fixed piston sampler is superior in quality to other fixed piston samplers. The reasons for that have not yet been identified but could be a combination of the following parameters:

1. The cutting edge is very sharp with an angle of 50° which minimizes the disturbance during the intrusion of the tube into the ground.
2. The tube is thin (area ratio = 0.08). This fact also minimizes the displacement of soil particles during sampling.
3. The stainless sampling tube is very smooth, minimizing the friction between the sample and the tube.

8 CONCLUSION

The Japanese fixed piston sampler was used in a sensitive soil deposit. It comes out from this study that the Japanese sampling method provides samples of quality comparable to the conventional Laval sampler. According to the NGI classification the quality is very good to excellent, which is the highest rank in the NGI classification. The modified Laval sampling technique provides samples of slightly better quality than the conventional Laval and Japanese procedures. The quality of the Japanese sampler is thought to be due to its geometrical and physical aspects and in particular to its small area ratio, its sharp apex angle and its smoothness.

9 ACKNOWLEDGEMENTS

The authors would like to acknowledge S. Paré and J-Y. Julien for their help during the field work as well as J-P. Dussault for the preparation of the laboratory equipment.

REFERENCES


Leblond, P. 1981. Mesure et caractéristiques des argiles Champlain. M.Sc thesia, Civil Engineering Department, Laval University, Canada.


The use of rotary sounding for new quality control of soil improvement

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ABSTRACT: In order to study the appropriateness of the rotary sounding method, a simple and highly reliable method of judging the strength of ground and suitable for use with the deep mixing method, a working machine was used to perform laboratory experiments and in-situ experiments. The results have revealed that rotary sounding can continuously measure ground strength and provides relatively good correlation with unconfined compressive strength, and it is possible to continuously measure the quality of the improved soil body in the vertical direction by means of rotary sounding.

1. INTRODUCTION

The deep mixing soil stabilization method, which is a method of solidifying soft soil by mixing stabilizing agents such as cement and lime with the soil, has been applied to an increasingly wide range of work as a result of the introduction of improved and newly developed stabilizing agents and execution machinery, and in recent years, it has frequently been employed for large scale soil improvement work. Check boring is usually performed to assess the quality of soil improved using the deep mixing soil stabilization method. Boring and removal of cores is performed from 2 to 4 times during each execution or one out of every 500 times. Unconfined compression tests are performed on the cores according to the length of the improved soil body or the constituents of the soil. This method is not trouble free: [1] it is difficult to obtain good quality average core specimens so it is also difficult to clarify continuous strength properties, [2] it requires a great deal of work, time, and expense and [3] it is not clear how to set appropriate check boring frequencies. A more reliable execution control method is necessary, particularly for use with large scale work.

And because the composition of soft soil improved using the deep mixing soil stabilization method is frequently complex, there are cases in which, depending on the soil stratification, it is impossible to obtain the forecast strength even following a thorough study of the blend of the improvement agents used to improve the soil. There is a high probability of the existence of extremely soft sections in organic soil with a particularly high water content, and in such cases this section may become critical for slippage stability. To resolve these problems, a soil inspection method which can perform continuous measurements of the strength of the entire improved soil body is necessary to detect extremely weak parts of the improved soil.

To achieve this goal, rotary sounding has been developed to provide a simple and reliable soil

<table>
<thead>
<tr>
<th>Prepared by</th>
<th>Boring Speed Equation</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Swanson, W.H</td>
<td>( R = 0.25 \sqrt{W} )</td>
<td>D: Bit diameter</td>
</tr>
<tr>
<td></td>
<td>( R = 0.25 \sqrt{W} )</td>
<td>k: Stability coefficient</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N: Penetration speed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S: Soil strength</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D: Depth</td>
</tr>
<tr>
<td>Kiehl, N.H</td>
<td>( R = 0.25 \sqrt{W} )</td>
<td>F: Drift parameters</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P: Borring speed at atmospheric pressure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \rho ): Density of soil</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( D ): Diameter of the boring surface</td>
</tr>
</tbody>
</table>

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strength inspection method suited for use with the deep mixing soil stabilization method. With rotary sounding, a soil inspection method which judges the strength of soil from boring resistance on a sensing rod, the measurement rod at the tip continuously measures and records the water pressure and the boring resistance (thrust, torque) generated by the tip bit when boring conditions (boring speed, rotation speed) are constant.

This report describes the principle of rotary sounding, system configuration and laboratory experiments, the results of experiments at a number of locations.

2. PRINCIPLE OF ROTARY SOUNDBING

Rotary sounding is used as oil well drilling control technology. There are four principal conditions related to the boring speed during drilling:

1. Soil quality conditions: Strength and category of the rock, boring speed, permeability, pore water pressure.
4. Operating control conditions: Bit thrust, bit rotation speed.

Because the soil conditions depend upon the purpose of the boring, it is necessary to select the other three conditions in order to optimize the planning and control of the boring. And because the circulation and bit conditions are selected in advance, it is important to set the operating control conditions in order to perform the boring as economically as possible.

A boring speed equation is one formula which finds the optimum values for the operating control conditions. Table 1 shows principal boring speed equations which have been proposed.

These various formulae reveal that if it is assumed that the bit shape is fixed and slurry conditions do not change very much, the principal parameters for the boring are the boring speed, thrust, rotation speed, torque, and soil strength.

Rotary sounding is used to quantitatively judge soil strength by experimentally finding the relationship of the soil strength-unconfined compressive strength or the N value or example-with the other parameters. The following is the ground strength relational expression for rotary sounding.

\[ q_s = K \cdot R^n \cdot a^b \cdot W^c \cdot T^d \] (1)

where \( R = \) Boring speed; \( n = \) Rotation speed; \( W = \) Thrust; \( T = \) Torque; and \( a,b,c,d = \) Constants.

And \( K \) in the boring speed equation is called the drillability constant, and its value varies according to the soil quality, shape of the bit, amount of abrasion, viscosity of the circulating water, etc. Consequently, it is possible to convert the soil strength under unconfined elements by varying \( K \).

3. ROTARY SOUNDBING SYSTEM

As seen in Figure 1 and Figure 2, a rotary sounding system consists of a sensing machine and sounding rod, an above-ground measurement device, and a
data analysis device (analysis software).

The sensing machine, a machine especially designed for rotary sounding and equipped with a mechanism that can automatically control boring conditions, measures the boring speed and rotation speed during boring, while above ground it measures the thrust (insertion resistance) and torque.

The sensor on the sensing rod measures the thrust, torque, and water pressure generated at the end bit during boring in order to eliminate the effect on the measurement values of the friction and elastic deformation of the outer surface of the rod. These measurement parameters are electrically amplified and stored in the measurement memory at fixed intervals (3, 4, 5, or 10 seconds). The measurement memory houses a smoothing circuit to reduce random noise such as the vibration of the machine.

The bit is designed so that a uniform load is constantly applied by the soil, and shaped to provide good boring performance and to efficiently dispose of the soil. It includes a water feed hole to prevent slime etc. from adhering to the blade. The three bit shapes shown in Figure 3, drag, two-cone, and three-cone, are now provided. The type used depends upon the strength and other properties of the soil.

4. LABORATORY TESTING OF THE ROTARY SOUNDING SYSTEM

A number of improved soil models with varying strength were prepared by blending kaolin clay or river sand with cement, and the properties of the boring resistance in different types of soil were studied.

The test conditions are shown in Table 2. The interrelationship between the boring speed R, thrust W, and torque T measured during rotating insertion of the kaolin clay specimen is shown in Figure 4 for each specimen. In every graph, there is a trend indicated by a straight line, and as the average unconfined compressive strength (q,) of a specimen increases, the straight line shifts to the left. This indicates that the measurement items obtained from rotary sounding are in a mutual exponential function relationship, and the shape of the soil strength relational expression shown in (1) is established.

The soil strength relational expression was found for a case where a kaolin clay specimen was bored using a two-cone bit. Because a proportional relationship was observed between the thrust W and the torque T, dimensional analysis was performed using formula (1) assuming that d = 0, and the value of the constant was calculated.

The following result was obtained.

\[ q' = 1.96 \times 10^2 \cdot R^{1.13} \cdot W^{0.26} \]  
(Two-cone bit in kaolin clay)  
\[ q' = 3.85 \times 10^3 \cdot R^{1.13} \cdot W^{0.33} \]  
(Two-cone bit in river sand)

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Figure 5 presents a comparison of the equivalent unconfined compressive strength ($q_u$) calculated from formula (2),(3) with the unconfined compressive strength ($q_u$) at the same depth. This reveals that it is possible to represent the unconfined compressive strength with a certain degree of precision using formula (2),(3).

5. EXAMPLE OF APPLICATION TO THE DEEP MIXING SOIL STABILIZATION METHOD

This section introduces two examples of the application of rotary sounding on ground improved using the deep mixing method in order to study the applicability of rotary sounding at actual work sites.

5.1 Application Example 1

To investigate the applicability of rotary sounding on actual ground, rotary sounding was executed at a work site where the ground was improved by the deep mixing method. The ground was on a river reservation at the toe of the slope of a super levee. The stratification of the natural ground was, as shown in Figure 6, beginning at the surface, alluvial sandy soil, alluvial sandy mud soil, and diluvial clay soil. The ground was improved between 1.0 and 30.5m from the surface on the upstream side (section A) and between 1.0 and 27.5m from the surface on the downstream side (section B). Twenty-eight days after execution, core samples were obtained and a two-cone bit was used to carry out rotary sounding of 11 improved soil samples including three column-shaped samples of the same improved soil. Four of the samples were taken in section A and 7 in section B. During the execution, in principle, the boring speed was maintained at 0.50 cm/s and its rotation
speed at 60 rpm, with the boring speed, rotation, thrust, and torque measured at five second intervals. The measured values obtained were substituted in the ground strength estimation equation obtained from laboratory experiments to calculate \(q'\). In this case, the substitution was performed on ground strength estimation equation (3) for a case where a two-cone bit was used on a sample taken where the natural ground was river sand.

The results are presented in Figure 7. Because the measured values were obtained almost continuously in response to the shape of the ground, the converted unconfined compressive strength \(q'_u\) was also calculated continuously. The solid line which continues on the right edge of the figure is \(q'_u\). A comparison with the laboratory experiments referred to above reveals that although there is a high correlation between \(q'_u\) and \(q_u\), because \(q'_u\) was obtained continuously, it is possible to determine the ground strength of low strength parts where sampling using core boring is difficult to perform.

5.2 Application Example 2

Rotary sounding was performed in ground improved using the deep mixing method. Parameters with a high contribution rate were identified by multiple correlation analysis to obtain the following formula.

\[
q'_u = 6.39 \times 10^4 \cdot W^{0.801}
\]  

(4)

Figure 8 shows the equivalent unconfined compressive strength obtained from formula (2), (3) and (4). In formula (2) for a case where the specimen is kaolin clay in an area where the natural ground is clay soil, there is a high correlation with unconfined compressive strength. This is believed to be a result of the fact that the shear properties when the improved ground suffered compressive failure caused by the bit differ according to the grain size of the natural ground. And formula (4) reveals a somewhat high correlation with the unconfined compressive strength throughout the improved ground.

6. STRENGTH ANALYSIS OF GROUND IMPROVED BY THE DEEP MIXING METHOD

Present quality control used with the deep mixing soil stabilization method and other solidification work is premised on core boring, and is performed by comparing the test strength with the average value of the unconfined compressive strength for each stratum of the natural ground. But because the judgment that enough samples have been taken to permit an evaluation is left up to an on-site manager’s decision that an evaluation can be made based on the material at the soil sampling site, there is a danger of differences between people influencing the final results. So the results of the measurements described in 5.1 were used to study the strength distribution of ground improved using the deep mixing method of soil stabilization in order to study the feasibility of a more objective quality control method.

Figure 9 presents the results of categorizing the equivalent unconfined compressive strength \(q'\) obtained by performing rotary sounding for each stratum of the natural ground and preparing a histogram for each ground strength 0.5 MPa.

The average value of the converted unconfined compressive strength is largest for diluvial humus and smallest for diluvial clay. This is believed to be a consequence of the fact that while in diluvial humus, the quantity of hardening material was high based on an assessment of the soil quality and strength of the natural ground, it was lower in diluvial clay. But in diluvial humus, the strength is broadly scattered and the probability of it being distributed below the design strength is impossible to assess based on the size of the average value. So the standard deviation was introduced as an index representing the scale of the scattering.

An approach to quality control standards for a number of design and execution guidelines for the deep mixing method is represented by the following formula.
where: $q_{ck}$: Design reference strength, and $q_{ck}$: In-situ average strength.

The design unconfined compressive strength at this site is 0.59 MPa. When the converted compressive strength average value and standard deviation are substituted in formula (5) for different types of natural ground, the value of $k$ is 1.15, 1.33, 1.49 and 1.30 for alluvial sandy soil, alluvial clay soil, diluvial lumus, and diluvial clay respectively, revealing that it is almost constant at all strata of the natural ground. If it is assumed that the strength of improved ground is normally distributed, the probability of $q_{ck} \leq q_{ck}$ ranges from 6.8% to 12.5%.

When solidification work which satisfies existing quality control standards is measured continuously by rotary sounding, the probability of $q_{ck} \leq q_{ck}$ is shown to be about 10%, and this is believed to be one scale when the quality of improved ground is continuously assessed.

7. CONCLUSIONS

The rotary sounding method has been developed as a quality assessment method for the deep mixing soil stabilization method, and basic laboratory testing and on-site testing have been performed. The results have revealed that soil strength can be continuously measured, that the correlation with unconfined compressive strength is relatively good, and that it is possible to continuously control the quality of the improved soil body in the vertical direction by means of rotary sounding. And this has opened the door to research on a method of distinguishing soils of different categories based on the cutting sound during rotation of the cone discussed above, and it is seen as a useful method of promoting the computerization and automation of soil exploration methods. In the future, tests of various kinds of soil should be carried out, data accumulated, and the range of soils where this method can be applied should be clarified.

REFERENCES

Sugimura, Ushio, Komoizaki, Terao, Chida, Improved Soil Quality Assessment Experiment Using Rotary Sounding (Part 1), Japan Society of Civil Engineers, Fifty-Seventh Annual Conference, 1996.
Minakashi, Tsukada, Improved Soil Quality Assessment Experiment Using Rotary Sounding (Part 2), Japan Society of Civil Engineers, Fifty-Seventh Annual Conference, 1997.
Study of a soil assessment method based on sounds generated by rotary sounding

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ABSTRACT: This report considers the practicality of a soil assessment technique based on sounds generated during rotary sounding by applying the technology used in a shield tunneling. Laboratory experiment, model ground experiment and numerical analysis have been executed. The results of them have revealed that it is possible to quantitatively assess the difference in the sounds recorded for different soil types to estimate their grain size distribution.

1. INTRODUCTION

The principal objective of ground exploration is to clarify conditions of the foundation on which a structure is constructed in order to contribute to the design and the execution both in scale and number. The increasing of construction projects in large cities has increased the need for high quality of ground information in every detail [1].

In response to this demand, exploration methods have been developed in an effort of improve precision by combining multiple measurement items of the information[2][3][4]. One of them is the rotary sounding[5]. In this method the drilling conditions such as drilling speed, and rotation speed are kept constant during a boring. And the drilling resistance of bit load, drilling torque at end of the drill bit and a slurry pressure are measured in order to estimate the unconfined compressive strength of the ground. A full automatic system of the method can perform continuous measurements in the depth direction without human error, and it can be used for a wide range of ground including soft ground, bedrock, or even improved ground. The soil assessment however requires the use of other methods. Resolving this problem would make it more useful method.

In shield construction work on the other hand, it is vital to detect obstructions and changes in the soil at the cutting face in order to boost the safety and efficiency of work. Research for this problem has been rewarded with the development of a method to detect changes using the sound generated from the cutter bit cuts the soil[6][7][8].

This report considers the practicality of a soil assessment technique based on sounds generated during rotary penetration by applying the technology used in a shield tunneling. The rotary sounding method employing new measuring technique could improve the reliability of civil engineering structures and the contribute to the rationalization of design and the execution.

2. ROTARY SOUNDING ATTACHED BY ACCELERATION SENSOR WITH SMALL SPECIMEN

2.1 Test Method

As shown in Figure 1, the test apparatus is equipped with a penetration rod (diameter: 40 mm) rotated (at 80 rpm) by an electric motor, and an air cylinder that raises and lowers the penetration rod with compressed air (0.5 kgf/cm²) to perform rotating penetration of small specimens at a penetration speed of 1 mm/sec. The nine types of soil specimens shown in Figure 2 were used for this testing. The specimens were formed in a mold with a diameter of 350mm and height of 350mm. Three layers were compacted...
sounding was detected by an acceleration sensor equipped with an internal pre-amp, resonance of 15 kHz the sensor is housed inside a cutting bit (drag bit, diameter of 40 mm) and attached to the tip of the penetration rod. The sound is recorded by Digital Audio Tape-recorder through a signal measurement device. During the experiment, the signal was observed on an oscilloscope and the sound was monitored by the human ear through the speaker.

2.2 Test Results and Considerations

a) Volume

Volume is represented by the amplitude of the measured signal. Figure 3 shows the representative signal wave forms for three kinds of soil (gravel, sand and clay). Because the amplitudes for sand and clay are smaller than that of gravel, the amplitude is set at 30 times for sand and at 400 times for clay.

In Figure 3, the signal wave forms of each soil type are almost similar however the amplitude of gravel is highest, that of sand lower, and that of clay lowest. The volume of the sounds heard from the speaker during the experiment conforms with this sequence. This indicates that the amplitude is an important information for the assessment of soil quality.

b) Tone Quality

When a Fourier transformation is performed on the measured wave signal to obtain the mean square, its power spectrum is obtained. This represents the distribution of the frequency component, and represents the tone quality as the frequency characteristics chart.

Figure 4 presents the frequency characteristics charts for the three soil categories. The scale of power spectrum is also set as 30 times for the pit sand and 400 times for the clay.

The frequency characteristics charts indicate that the dominant frequency ranges for every soil category are close to 3 kHz, 6 kHz, and 11 kHz. But the overall form of the frequency distribution varies between soil categories, with the dominant frequency close to 1 kHz in the clay. In other words, the frequency characteristics of the cutting sound varies depending on the soil property.

c) Energy and Spectrum Moment

The signal measured during penetration must be used as a parameter in order to use the cutting sound for quantitative evaluation of soil properties. So the numerical values representing volume and tone quality were determined and differences in cutting sound were studied.

Figure 1. Experiment System of Small Specimen Test

Figure 2. Grain Size Distribution of Small Specimen

Figure 3. Typical Wave Forms (Gravel, Pit Sand, Clay) separately by a 2.5 kgf hammer with falling 30cm, fifty times.

The cutting sound generated during rotating
As Figure 4 presented above indicates, the higher the volume, the greater the power spectrum, and the more powerful a certain frequency component, the greater the power spectrum for this frequency. In other words, because the sum of the power spectra (area) is believed to represent the volume and the distance to its gravity seems to represent the tone quality, they are referred to as the energy and the spectrum moment respectively, and are determined as the numerical values representing the volume and tone quality.

Figure 5 plots the experimental results with the energy of each soil represented by the horizontal axis and their spectrum moments by the vertical axis. This figure reveals that the larger the grain size of soil, the larger the range in which its energy is distributed, and that energy is an important parameter which can play significant role in the assessment of soil.

The energy of gravel and sandy gravel, pit sand and silty sand, clay soil and Kanto Loam are similar but their spectrum moments differ. The grain size distribution of these soils is similar, with the gravel component of gravel and sandy gravel, the fine sand component of pit sand and silty sand, and the clay component of clay loam and Kanto loam are the main constituent of each type of soil. Since the spectrum moments of these soils differ, it is clear that spectrum moment is also a parameter which can be used to perform soil assessments. These results show that the cutting sound can be used to assess soil quality.

3. ROTATING PENETRATION TESTING OF MODEL GROUND

3.1 Purpose of the Testing

This test was performed to study the usefulness of the technique using boring machine. In the boring, water is supplied to remove the soil. So during this testing, water was supplied and a study performed to make sure that it did not interfere with the soil quality assessment.

3.2 Testing Method

In order to create model ground similar to actual complex ground, as shown in Figure 6, three kinds of soil—Kanto loam, river sand, and gravel—were used to form two kinds of model ground in soil tanks. The two models, called model ground 1 and model ground 2, were compacted with a rammer at 25 cm intervals.
Assuming that differences in bit shape would effect the cutting sound, the two kinds of bit were used. One is a drag bit which causes shear failure of the soil at the cutting surface, and the other is a roller bit that causes compression failure of the soil. The test was performed for four cases using an ordinary boring machine with penetration speed of 1 cm/sec., rotating speed of 60 r.p.m., and water supplied volume of 20 l/sec. A sensor was installed inside the bit, and as the bit rotated to penetrate the ground, the cutting sound was recorded by the Digital Audio Tape-recorder.

3.3 Experiment Results and Considerations
Like the small model test, the energy and spectrum moment of the cutting sound for each stratum is calculated, and the results against the depth are shown in Figure 7.

This figure shows that both the energy and spectrum moment change depending on the soil quality and that it is possible to assess the soil quality, even in multiple strata. In model ground 2, an attention was focused on the possibility of correctly identifying a loam stratum with little energy in case of penetrating into soft layer, and it indicated change under the effect of soil quality, revealing that there is no such a difference between. The effects of differences in bit shape are apparent in the spectrum moment, mainly in the gravel stratum, but little difference is observed in the other strata.

Figure 8 represents these results in the form of the energy - spectrum moment relationship chart, indicating that each soil type forms a unique group, just as in the case of the small model test. The points plotted vary depending on whether water was or not supplied, but the extent of the variation does not exceed the range for each soil type. This means that water does not interfere for the assessment of the soil type.

These results imply that this technique can assess the soil type regardless of the bit shape and water, and that it is possible to apply this technique to an ordinary boring machine.

4. NUMERICAL ANALYSIS BY A NEURAL NETWORK

4.1 Outline of the Analysis
Ordinary computer program can not process with adding new information while running the program. Human decision absorbs a knowledge and an experience to increase their capability to judge, so even when dealing with phenomena which does not fit with their accumulated knowledge and experience, they can arrive at judgments of some kind. Neural network is a numerical analysis method which simulate this kind of superior human judgment capabilities. In this study, the Back Propagation Network, (BPN) has been used.

This method uses a hierarchy network to model a human neurons and forms a network by learning the weight of the connections between the neurons from training data. The flow is shown in Figure 9. At the initial learning, random numbers are provided to the weights, and training data is input. After the learning is completed and the weights are determined, and used as the initial value. The analysis data is provided into the input level of the hierarchy, and a number of 0 to 1 is input to each neuron of the
use even more parameters which reflect the unique properties of each category. Consequently, the input data must be numerical values which accurately reflect differences in soil quality. As described above, differences between soil types are well represented by the volume and tone quality of the cutting sound and these are represented as amplitudes and frequency characteristics.

So, amplitude distribution analysis was performed and its frequency was treated as the amplitude parameter. As shown in Figure 10, the amplitude distribution can be treated as a normal distribution. So the plus side is enough to obtain the distributor. The amplitude of 0 to 0.15 V is 0 to 0.15 V of amplitude is divided into 20 parts.

With the frequency characteristics chart, on the other hand, it is possible to treat the power spectrum in the same way as the frequency in the amplitude distribution analysis. The power spectrum calculated by dividing 0 to 20 kHz into 20 equal parts is used as the frequency data.

4.3 Analysis Results

a) Results of Analysis of the Small Specimen Testing

Table 1 shows the results of analysis performed using the amplitude parameters and the frequency parameters. The columns of analysis present the soil categorized along with the output and the maximum output value is enclosed in a dark line.

The analysis that was performed with nine kinds of training data shows that gravel, sandy gravel and gravelly sand are categorized as sandy gravel. Bit sand and silty sand are also categorized as silty sand, and clay soil and Kanto loam are as Kanto loam, as well. In other words, analysis data of nine categories is categorized into only five categories: sandy gravel, river sand, silty sand, Kanto loam, and clay, and these coincide with the regions indicated by dot lines in Figure 5.

So, training data for these five categories was learned and the analysis was performed for 5 categories. It is necessary to add the other parameters in order to improve the judgement, however, this judgement capability is sufficient for the practical use.

From the above, it has been concluded that it is possible to perform quantitative judgments of soil quality by performing numerical analysis using a neural network based on amplitude parameters representing sound volume and frequency parameters representing tone quality.

b) Results of the Analysis of the Model Ground Testing
Table 2. Results of Model Ground (1)

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Density (g/cm³)</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Shear Strength (MPa)</th>
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Table 3. Results of Model Ground (2)

<table>
<thead>
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<th>Stratum</th>
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</table>

Table 2 presents the results of using the same way as in small specimen analysis to calculate the amplitude parameters and frequency parameters from the cutting sound at each stratum during the boring on the model ground. In this analysis categories are three, and half of this data was used as input data and the other half was used as analysis data. The analysis correctly categorize the soil in every stratum and the output values are high without any error in categorization at both the learning stage and the analysis stage. This is probably because there are not many soils (categories) to be judged compared to the test of small specimens and their characteristics are accurately reflected in the parameters. Since the values obtained from the drug bit testing are higher than those obtained with the roller bit, it has been concluded that it is easier to assess soil with a roller bit.

Table 3 presents the results of learning data obtained from model ground 1 and analyzing data from model ground 2. The output values decline slightly in this case however the soil is assessed correctly and it is possible to analyze data for different strata.

In summary, by learning appropriately the training data, it is possible to assess soil using unknown data obtained from complex strata.

5. CONCLUSIONS

From the tests with small specimens and model ground, it is found that the penetration generates different on the type of soil, and a numerical analysis such as a network can divide the signal into the group of soil. These results show that this technique is useful to determine the soil type in the rotary sounding. Although the tests performed on the natural ground is not described here because of limited space, this technique can obtain a sensitive and continuous date in direction of the depth.

A further study is needed to improve the precision, for instance, the use of the other statistical method for quantitative analysis, a selection of more useful parameter of the signal and the accumulating various kinds of signals obtained from natural ground.

REFERENCES

Geophysical testing – Seismic techniques
Delineation of densified sand at Treasure Island by SASW testing

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ABSTRACT: Areas of improved and unimproved soil near berthing Pier 1 at Treasure Island, California, were investigated by the Spectral-Analysis-of-Surface-Waves (SASW) test. The upper 12 m of sand fill beneath the approach to the pier had been densified by a vibrating probe technique in 1985. The area of improved soil performed well during the 1989 Loma Prieta earthquake, while sinkholes, sand boils and cracks formed in the adjacent unimproved areas. The SASW tests were conducted on a 240 m-long alignment that extended across the area of improved soil. Average shear wave velocities determined for the densified and un densified sand fill below the water table were 192 m/s and 167 m/s, respectively. Two simplified liquefaction assessment procedures based on shear wave velocity correctly predicted no liquefaction for the densified sand, and marginal liquefaction for the undensified sand.

1 INTRODUCTION

Liquefaction of loosely deposited granular soils is a major cause of damage in earthquakes. Delineation of weak soil layers and prediction of their liquefaction potential are key inputs to the engineering design of new or retrofitted structures. This information is also essential for reliable estimation of economic losses during future earthquakes. When projects extend for great distances, such as lifelines and large building complexes, cost-effective evaluations of extensive areas are required. Screening techniques based on geology, hydrology, and soil conditions show promise for identifying areas requiring more rigorous analyses. However, even these areas requiring further analyses can be quite large.

One promising technique for spatially evaluating the liquefaction susceptibility of granular soils is the Spectral-Analysis-of-Surface-Waves (SASW) test. This test is an in situ seismic test for determining small-strain shear wave velocity, \( V_s \), profiles of soil deposits and pavements (Stokoe and Naizar, 1985; Stokoe et al., 1988; Gucunski and Woods, 1991; Stokoe et al., 1994). The SASW test does not require boreholes, and has the advantage of providing broad areal coverage. Testing can be performed at sites where minimal disturbance is required and where soils are difficult to sample. The use of \( V_s \) as an index of liquefaction potential is justified since both \( V_s \) and liquefaction potential are influenced by many of the same factors (e.g., void ratio, effective confining pressure, stress history, and geologic age).

Thus, the SASW test is well suited for profiling large areas with the objective of developing two- or three-dimensional images of the subsurface.

In March 1996, SASW tests were conducted across an area of densified sand at Treasure Island, California. The site, called the Improved Soil Area (ISA) herein, is located on the south-eastern corner of the island, as shown in Figure 1. The principal objective of these tests was to evaluate the ability of the SASW test to rapidly delineate stratigraphy and assess liquefaction resistance of the densified and undensified sands over a significant lateral extent. This paper presents the two-dimensional stiffness profile at ISA along with the liquefaction evaluation.

Figure 1. San Francisco Bay showing locations of the Improved Soil Area (ISA) and Fire Station Site (FSS) at Treasure Island.

*This paper is a U.S. Government work and, as such, is in the public domain of the United States of America.*
1.1 Treasure Island

Treasure Island (TI) is a man-made island constructed in 1936-37. It was formed by hydraulic filling
behind a perimeter rock dike. The perimeter dike
served to contain the hydraulic fill, and was raised in
sections over the previously placed fill. Currently,
the island is occupied by the U.S. Navy.

In 1991, TI was selected as a national geotechnical
experimental site. Much of the work to date
centers around a ground response experiment (de
Alba and Farris, 1996). Six accelerometers and eight
piezometers are operating at various elevations near
the fire station (see Figure 1). Inclinometer casings
are in place at the fire station and at two sites at the
perimeter of TI, including the Improved Soil Area.

1.2 Improved Soil Area

The Improved Soil Area, shown in Figure 2, is
effectively level and capped by a 127 mm thick layer
of asphalt. The upper 12 m of soil is sand fill initially
deposited in a loose to medium dense state during
hydraulic filling. Grain-size distribution curves for
six samples taken from the fill are shown in Figure 3.
Samples above a depth of 6 m contain as much as
17% fines (silt and clay). Below 6 m, samples
contain 1% to 4% fines. The fill is underlain by 3 m
of native silty clayey sand followed by 27 m of soft to
stiff clay with interbedded sand layers. The clay is
underlain by alternating layers of very stiff sandy clay
dense sand. Sandstone and shale bedrock occurs
at a depth of 87 m at the fire station (de Alba and
Farris, 1996). It is assumed that the bedrock surface
slopes upward from the fire station to the sandstone
rock forming Yerba Buena Island (see Figure 1). At
the time of SASW testing, the water surface in the
bay was about 2 m below the ground surface.

Because of concern for the seismic instability of the
waterfront slope, the fill beneath the approach to
Pier 1 was densified to a depth of 12 m by a vibrating
probe technique in 1983. The area penetrated by
the large metal-tube probe was 23 m wide and 97 m long,
as shown in Figure 2. From construction drawings by
Foundation Contractor Inc., initial tests were
conducted at the northwest corner of the improved
area to determine the optimal probe spacing.

Subsequent production probes were performed to
produce a final 1.90 m or 2.24 m probe spacing in a
triangular grid pattern. Gravelly material was inserted
through holes in the wall of the probe and vibrated
into the ground. Curves 4-6 shown in Figure 3 are
for samples taken from the improved zone.

Following the 1989 Loma Prieta earthquake (Mw =
7.0), no signs of ground disturbance were observed
in the improved area, while sinkholes, sand boils and
cracks were seen in the adjacent unimproved areas
(Geomatics, 1990; Mitchell and Wentz, 1991).

Figure 2. Improved Soil Area showing locations of
structures (de Alba and Farris, 1996) and tests.

2 SASW TEST

The SASW test is based on the principle that the
extent of the soil profile sampled by surface waves
varies with frequency (wavelength). Thus, if
stiffness varies with depth, surface waves of different
frequencies will propagate at different velocities.

Field tests were performed by placing two receivers
on the ground surface a distance D apart, as illustrated
in Figure 4. A truck-mounted seismic vibrator (or
vibrators) weighing 180 kN was used as the source
for spacings over 8 m. For shorter spacings, handheld
hammers and dropped weights were used. The
two receiver signals were recorded, and transformed
into the frequency domain using a FFT signal
analyzer. From the two frequency domain records,
the coherence and the phase of the cross-power
spectrum were computed.
Where data quality was good in the phase plot of the cross-power spectrum, Rayleigh wave phase velocity, $V_R$, and corresponding wavelength, $\lambda_R$, were calculated for each frequency by:

$$V_R = D/(2\pi f)$$  \hspace{1cm} (1)$$

$$\lambda_R = V_R/\pi$$  \hspace{1cm} (2)$$

where $\Phi$ is phase difference in radians, $f$ is frequency in cycles per second, and $\pi$ is a constant of about 3.14. A plot of $V_R$ versus $\lambda_R$ was assembled with the results for all receiver spacings. This plot is called the experimental dispersion curve.

Shear wave velocity profiles were obtained through an iterative process of matching the experimental dispersion curve to the theoretical dispersion curve. To begin this iterative process, initial properties (shear and compression wave velocities and total unit weights) and layer thicknesses were assumed. A theoretical dispersion curve was calculated for the assumed horizontally layered profile using the three-dimensional computer model by Rotevat et al. (1991). The assumed properties (primarily $V_s$) and layer thicknesses were adjusted until satisfactory agreement between the theoretical and experimental dispersion curves was obtained. Agreement between the two dispersion curves was assessed visually and by a maximum likelihood method formulation (Jeh 1996). The $V_R$-values and layer thicknesses for the final theoretical dispersion curve were then used to represent the actual profile of the site.

3 RESULTS

Experimental dispersion curves obtained for receivers spacings of 7.6 m, 15.2 m, and 30.5 m are plotted in Figure 5. The dispersion curves for test arrays in the improved area (solid symbols) are distinctly separated from the dispersion curves for test arrays in the unimproved area (open symbols). The dispersion curves for two arrays located 40-50% within the improved area (+ symbols) lie between the open and solid symbols, as shown in Figures 5b and 5c. Values of $V_R$ for the improved area are as much as 90 m/s higher than values of $V_R$ for the unimproved area at a wavelength of 3 m. This difference in $V_R$-values decreases to about 15 m/s at a wavelength of 30 m. Between wavelengths of 5 m and 24 m, the average difference in values of $V_R$ is 31 m/s.

Figure 5. SASW experimental dispersion curves.
SASW test array 4c is located next to a sinkhole formed by liquefaction during the 1989 Loma Prieta earthquake. Values of $V_g$ for test array 4c are among the lowest measured, as shown in Figures 5a and 5b. This observation was also expected, since array 4c lies closer to the waterfront slope (see Figure 2) where overburden pressures in underlying soil are lower.

Shear wave velocity profiles for SASW tests conducted in the improved and unimproved areas are shown in Figures 6 and 7, respectively. Values of $V_g$ for the improved area are about 94 m/s higher than values of $V_g$ for the unimproved area at a depth of 1 m (226 m/s versus 132 m/s). At a depth of 13 m, the difference between $V_g$-values is about 7 m/s (193 m/s versus 186 m/s). This trend is similar to the measurements of $V_g$ (see Figure 5) and the Standard Penetration Test N-values given in Figures 6 and 7.

A depth of 13 m agrees well the reported depth of densification of 12 m. Between the depths of 2 m and 13 m, average $V_g$-values for the undensified and densified fill are 167 m/s and 192 m/s, respectively.

Assembling the $V_g$- and $V_p$-profiles presented in Figures 5, 6 and 7 leads to the two-dimensional velocity profiles shown in Figure 8. Several test setups near the southern end of the improved area permit good resolution of the boundary separating densified and undensified sands. At the northern end of the improved area, however, the number of test setups are limited and the agreement between the velocity profiles and the lateral limit of vibrating probes is rather poor. Nevertheless, the zone of densified sand is clearly identified in both profiles.

Since the process for obtaining $V_p$-profiles is not computationally intensive, two-dimensional profiles, such as the one shown in Figure 8a, could be completed during field testing. For the measurements presented in this paper, field testing was completed.

Figure 6. Three shear wave velocity profiles for SASW tests conducted in the improved area.

Figure 7. Seven shear wave velocity profiles for SASW tests conducted in the unimproved area.

Figure 8. Two-dimensional velocity profiles.
within a 7-hour period. This time could be reduced once a routine is established. Thus, similar two-dimensional profiles with lengths of 500 m to 1000 m could be generated in a day.

The process for obtaining a single V_s profile is computationally intensive, often requiring more than 8 hours of computer time to complete.

4 LIQUEFACTION ANALYSES

Several liquefaction assessment procedures based on V_s have been proposed during the past decade. These procedures are evaluated by Andrus and Stokoe (in press) using liquefaction and non-liquefaction case histories from 17 earthquakes and over 43 sites. From the case histories, modifications to earlier procedures are recommended. Shear wave velocity measurements from the ISA provide one of the first opportunities to apply these new procedures.

The recommended procedure follows the format of the penetration-based procedures, where penetration or V_s is correlated with the cyclic stress ratio (CSR). The CSR at a particular depth in a level soil deposit can be expressed as (Seed and Idriss, 1971):

$$\tau_{cyclic} = 0.65(\sigma_{vr0}/(\sigma_{vr} - \sigma_{cv}))$$

in which \(\tau_{cyclic}\) is cyclic shear stress generated by the earthquake, \(\sigma_{vr0}\) is peak horizontal ground surface acceleration, \(\sigma_{vr}\) is initial effective vertical stress, \(\sigma_{cv}\) is total vertical stress, \(g\) is acceleration of gravity, and \(\tau_{v}\) is a shear stress reduction factor with a value less than 1. Based on the data of 0.16 g and 0.11 g recorded in the x and y directions at the fire station during the 1989 earthquake (Brady and Shukal, 1994), an average value of 0.14 g is used in the analyses. Vertical stresses are estimated using total unit weights of 17.3-18.9 kN/m$^3$ and 19.5-21.2 kN/m$^3$ for soils above and below the water table, respectively.

The shear wave velocity is corrected with respect to a reference stress, \(P_r\), by (Robertson et al., 1992):

$$V_{s1} = V_s(P/P_{r0})^{0.25}$$

where \(P_{r0}\) is typically 100 kPa and \(P_{r0}\) in kPa.

Liquefaction in the unimproved soil most likely occurred where \(V_{s1}\) and fines content are lowest, and where CSR is greatest. These conditions occur between the depths of 6 m and 12 m.

Resistance to liquefaction caused by magnitude 7.5 earthquakes can be defined by (Andrus and Stokoe, in press; modified from Dobry, 1996):

$$\tau_{cyclic}/g = a(V_{s1}/100) + b[1/(V_{s1c} - V_{s1}) - 1/V_{s1c}]$$

where \(\tau_{cyclic}\) is cyclic shear stress resisting liquefaction, \(V_{s1c}\) is critical value of \(V_{s1}\) which separates contractive and dilative behavior, and "a" and "b" are factors with values approximately equal to 0.03 and 0.5, respectively. The value of \(V_{s1c}\) is about 220 m/s for uncremented soils with fines content less than 5%. For magnitude 7.5 earthquakes, Equation 5 is multiplied by a scaling factor of about 1.2.

Using Equation 5, the boundary separating liquefaction and non-liquefaction for magnitude 7.5 earthquakes is drawn in Figure 9. Also plotted in Figure 9 are average values of \(V_{s1}\) and CSR for the critical layer. The data for the improved area correctly lies in the region of no liquefaction. For the unimproved area, the data lies on the boundary, and marginal liquefaction is predicted. Located close to the perimeter of the island, sloping ground may have contributed to the amount of liquefaction effects. In addition, lateral ground displacement was only about 80 mm. Thus, a prediction of marginal liquefaction for the unimproved area is considered correct.

Another method relating liquefaction potential and \(V_{s1}\) has evolved from the strain approach by Dobry et al. (1982) and the analytical studies by Stokoe et al. (1989). By combining Equations 3, 4, and 5, a relationship based on \(V_{s1}\) and \(\eta_{max}\) is obtained in the form of (Andrus and Stokoe, in press):

$$\eta_{max}/\eta = f_1(\sigma_{vr}/V_{s1}/100) + f_2[1/(V_{s1c} - V_{s1}) - 1/V_{s1c}]$$

where \(f_1 = 1.1\times 10^{-1}\) and \(f_2 = (7.3\times 10^{-2})\), and \(z\) is depth to center of the critical layer in meters. This formulation assumes the water table is located midway between the ground surface and the center of the critical layer, and the total unit weights of soil above and below the water table are 17.3 kN/m$^3$ and 18.9 kN/m$^3$, respectively. The boundary for magnitude 7.5 earthquakes and depth of 9 m is shown in Figure 10. Liquefaction behavior predicted by this method is similar to the method based on \(V_{s1}\) and CSR.
CONCLUSIONS

The zone of densified sand adjacent to Pier 1 at TI was correctly identified in V_s and V_p-profiles obtained from SASW tests. Velocities measured in the improved area were about 30 m/s greater than velocities measured in the unimproved area. Two liquefaction assessment procedures based on V_s correctly predicted no liquefaction for the improved area, and marginal liquefaction for the unimproved area. This study further supports the usefulness of in situ V_s for predicting liquefaction potential, and demonstrates the potential of the SASW test for rapid delineation of weak layers. For large study areas, a cost-effective investigation program might be to first develop profiles of V_s in the field (assuming an approximate sampling depth equal to 1/3 to 1/2). The V_s-profiles would then be used to select locations for determining V_p-profiles and sites for borehole sampling and penetration testing.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance of Richard Parks, Naval Facilities Engg. Command, for scheduling tests and supplying site data. The help of Brent Rosenblad with field work and Sung-Ho Joh with data reduction is also greatly appreciated. We also thank Glenn Rix, Georgia Tech., for his review.

REFERENCES


Dobry, R. 1996. Personal communication.


Quantitative interpretation of steep basement reverse faulting from seismic reflection data

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ABSTRACT: The orientation of basement faults is important for evaluating the potential for fault reactivation. The seismic reflection technique is well suited for imaging faults in stratified media where they are associated with offset or terminating reflectors. In contrast, steep faults in crystalline basement rocks scatter energy away from receiver arrays and are typically not associated with offset markers. Steep reverse faulting in basement beneath stratified cover is propagated only partially through the cover as a discrete fault surface. Above the fault tip the strain is accommodated by monoclinal folding. This fault propagation fold can be characterized by the interlimb angle and the inclination of any unfaulted layer. These parameters may be related to the dip of the fault and the basement offset. These relationships allow characterization of steep basement faulting which is poorly imaged on seismic reflection data from parameters that may be easily measured from the better imaged cover.

1 INTRODUCTION

The orientation of fault surfaces in basement rocks is an important consideration when evaluating the mode or potential for fault activation due to an applied regional stress field. However, imaging and interpretation of steep basement faults with seismic reflection data can be severely limited by the steep dip of the structure which results in scattering of acoustic energy away from the receiver spread. This problem is exacerbated in crystalline basement rocks by the fact that there are in many cases no sub-horizontal events offset by the fault. Therefore, no strain markers can be seen on the seismic section by which the presence of a steep fault plane may be inferred. In contrast, seismic reflection data are typically well suited to imaging acoustic events in horizontally stratified cover sequences. In settings where steep basement faults have been active beneath horizontally stratified cover the presence and general location of steep faulting in basement can be inferred from the deformation imaged on the seismic data in the cover sequences. This deformation is commonly manifested as fault propagation folding.

McConnell (1994) gives a list of characteristics associated with folding of cover rocks resulting from basement involved faulting, specifically in cases where the axial surfaces of the folds remained fixed during folding and the forelimbs rotated to steeper orientations as deformation proceeded (i.e. not ramp-flat geometry). In this situation, deformation of the cover sequence results in folding characterized by axial surfaces which intersect the fault at or near the basement-cover contact and which diverge from the fault up-section (Fig.1). Folds also exhibit heterogeneous thickness changes characterized by thickening of limbs in synclinal hinge zones and thinning of forelimbs adjacent to anticlinal hinges. Faulting of the cover strata is localized in the steep fold forelimbs or near the antcline axial plane and rarely cuts through the syncline hinge zone resulting in preservation of footwall synclines. McConnell (1994) used these observations as the basis for a kinematic model of fold evolution in cover strata above steeply dipping reverse basement faults, assuming that fold hinges remain fixed during deformation and that beds in the forelimb changed inclination and thickness as deformation progressed. The basic geometric relationships used by
McConnell (1994) are illustrated in Figure 1. This analysis yields predictable relationships between the dip of the basement fault and the fold geometries in the cover.

In particular, McConnell (1994) gives quantitative relationships between the dip of the basement fault (\( \alpha \)) and the partial interlimb angles of the fold in the cover strata (\( \gamma_a \) and \( \gamma_h \)) and between the inclination of any unfaulted layer (\( \beta \)) and the vertical throw on the fault (\( H \)).

\[
\gamma_a = \tan^{-1} \left( \frac{1}{2 \cot \alpha - \cot \gamma_h} \right) \tag{1}
\]

and

\[
\beta = \tan^{-1} \left( \frac{H}{\cot \gamma_a - \cot \gamma_h - H \cot \alpha} \right) \tag{2}
\]

These equations may be rearranged to give:

\[
\alpha = \tan^{-1} \left( \frac{2}{\cot \gamma_a + \cot \gamma_h} \right) \tag{3}
\]

and

\[
H = n \left( \cot \gamma_a - \cot \gamma_h \right) / \left( \cot \beta + \cot \alpha \right) \tag{4}
\]

Cast in this form, these equations provide simple but extremely powerful tools for interpreting seismic reflection data from cover sequences over basement faults. The input parameters for these equations are determined from features that can easily be identified and measured from the seismic section. That is, fold axial surfaces, and interlimb angles determined from subhorizontal events in the cover sequence, can be used to quantitatively constrain features invisible on the seismic section in the basement. Also, since the measurements are determined by extrapolating linear features over several traces, resolution constraints applicable to determining offset from features on adjacent traces (1/4 wavelength) are not limiting and distortions due to localized statics effects are minimized.

In the following discussion we describe and use this technique to interpret the dip and offset of steep basement faults from the geometry of fault propagation folding in stratified cover imaged on seismic reflection data from the Coastal Plain in South Carolina.

### 2 SEISMIC DATA SET

The seismic data shown in Figure 2 were acquired in the vicinity of the Pea Branch fault (Suies and others, 1993) by Conoco Inc. in 1989 and subsequently reprocessed by Domaracki (1995). These data are migrated with a velocity of 2000 meters per second and displayed at a 1 to 1 horizontal to vertical scale. This results in true angular relationships on the figure so that interlimb angles can be directly measured from the section. The nominal wavelength of this data is 50 Hz; which, for a velocity of 2000 meters per second, would give a resolution limit of 10 meters at 1/4 wavelength. Datum is 80 meters with a 50 millisecond full shift.

Several prominent laterally extensive events between 100 and 400 milliseconds (Fig. 2) mark the reflective Coastal Plain cover sequence. The lowermost event at the base of the Coastal Plain cover sequence (basement; Figure 2) at about 0.4 s can be confidently identified based on downhole sonic information as the top of unweathered crystalline basement (Domaracki, 1995). Events A, B, and C can be identified based on downhole sonic information as the top of Cape Fear basal...
3 GEOMETRIC ANALYSIS OF FAULT PROPAGATION FOLDING

The measured angular relationships exhibited by the partial interlimb angles and inclinations of the unfaulted layers are used as input parameters for equations 3 and 4 in order to calculate the dip of the basement fault and the corresponding offsets for each of the horizon events (Table 1). All events give consistent values for the dip of the basement fault of 80 degrees. However, calculated offsets for each horizon generally decrease for earlier events. This indicates that the fault was an active structure as deposition of the sediments occurred and that each horizon records only that part of the offset since deposition.

Note that the location along the fault where the basement seems to behave as discrete fault blocks appears to occur below the event that marks the top of unweathered basement. This observation indicates that the shallow levels of unweathered crystalline basement are probably highly brecciated and exhibit bulk semi-ductile behavior at this scale so that deformation is distributed in a wide zone of cataclasis. However, the fault tip itself appears to have propagated up section at least as far as the top of unweathered basement.

4 DISCUSSION AND CONCLUSIONS

Based on the preceding analysis the dip of the Pen Branch fault in the basement has been constrained to 80 degrees. This analysis was accomplished based only on partial interlimb angles of the fault related folding in the deformed cover sequence overlying the basement fault which are the only features that can be reliably seen on the seismic section.

Geometric analysis of fault related folding in cover sequences overlying high angle reverse faulting in crystalline basement can be useful for determining the dip of the basement fault segment. However, this technique may also offer some advantages over drill core information in calculating

Table 1. Fold and fault parameters

<table>
<thead>
<tr>
<th>EVENT</th>
<th>( \gamma )</th>
<th>( \gamma' )</th>
<th>( \beta )</th>
<th>( \alpha )</th>
<th>( u )</th>
<th>( H )</th>
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<td>A</td>
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<td>80°</td>
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<td>90°</td>
<td>3°</td>
<td>80°</td>
<td>332</td>
<td>9</td>
</tr>
</tbody>
</table>
offset in the layered cover sequence. The values for the offsets calculated from the fold geometry, in our example, are consistent with those reported by Snipes and others (1993) as determined from core analysis from borings over this same structure. However, the fold geometry and associated deformation of layers require that the location of the boring data be carefully considered if used to analyze this type of structure. Note that if borings are located between the hanging wall anticline and foot wall syncline hinges, so that they sample the inclined limb, offsets determined from these borings will decrease as the distance between the borings is decreased. If only two borings were available for analysis and a discrete fault plane were assumed between them the inferred amount of throw on the fault would be too low. This situation would also result in the inferred throw becoming greater with depth possibly leading to the conclusion that the fault was active during deposition in fact the borings are only sampling the increasing limb dip with depth. Also note that any boring penetrating the thickened foot wall syncline would indicate thickened section relative to the hanging wall strata. This relationship could also be easily misinterpreted as evidence for growth faulting if the thickening in the syncline due strictly to deformation were not accounted for. Knowledge of and accounting for these phenomena would require several closely spaced borings if other independently determined information were not available (for instance seismic reflection data).

In our example the decreasing offset in the shallower parts of the cover sequence can be demonstrated to be real and are an indication that this structure represents a fault that was active during deposition of the cover sequence (growth fault), at least as determined from analysis of events A through C.

REFERENCES


Experimental procedures for detection of underground objects by the SASW test

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ABSTRACT: This paper presents experimental implementation of a modified SASW test, called the SWOD method, for detection of underground objects. The SWOD method procedures were originally developed from numerical simulations of the SASW test. Laboratory tests were conducted in a soil bin with and without an embedded rigid object. The results obtained correlate well with results from numerical simulations and demonstrate the ability of the SWOD technique to detect underground obstacles.

1 INTRODUCTION

The spectral analysis of surface waves (SASW) method is a nondestructive seismic method typically used in evaluation of elastic moduli profiles of horizontally layered systems like soils and pavements (Stokoe et al., 1994). Results of numerical simulations of the SASW test (Gucinski et al., 1996a and 1996b) and experimental studies (Al-Shayan et al., 1994) point to significant effects of soil nonhomogeneities, such as buried underground objects, soil lenses and layer dipping, on the shape of the dispersion curve obtained from the test. Backcalculation of the elastic modulus profile from the dispersion curve affected by soil nonhomogeneities can lead to misinterpretation of the profile.

Two major effects of underground obstacles on the dispersion curve were observed: strong fluctuations in the dispersion curve and a general increase in the phase velocity depending on the rigidity of the obstacle. These observations led to the development of a modified SASW test, called surface waves for obstacle detection (SWOD) method (Ganjii et al., 1997). Numerical simulation of the SWOD method demonstrated potential of the method in detection of the position of buried objects (distance from the source and depth of embedment), and to a certain extent characterization of the object (rigidity, shape and size).

This paper presents results of ongoing research on the practical implementation of the SWOD method. In particular, the work presented involves the development of an experimental setup and its laboratory implementation. Results of laboratory testing are presented and compared to those from numerical simulations.

2 SWOD METHOD

Detection of an underground obstacle using the SWOD technique involves testing similar to SASW testing, but with a small receiver spacing. The impact source is fixed at one location, and a closely spaced receiver pair is moved away from the source, evaluating the dispersion curve at every location (Figure 1).

Numerical simulations of the test indicate that
fluctuations in the dispersion curve increase as the receiver pair approaches the obstacle, and that they vanish as the pair crosses the obstacle. This phenomenon is more pronounced as the receiver spacing decreases, as demonstrated in Figure 2 for an elastic half-space with a square cross section cavity. The cavity was of unit dimensions and a unit depth of embedment, with the near edge 15 units from the source. The receiver spacing was set at one quarter of the size of the obstacle throughout the simulation. The normalized frequency in the figure represents the ratio of frequency in Hz and the shear wave velocity of the half-space, multiplied by a unit distance.

3 EXPERIMENTAL SETUP

The SWOD technique was implemented and evaluated in a soil bin 2 m long, 1.5 m wide, and with a 1.05 m deep soil. The soil was a medium to fine sand, placed and compacted in seven 15 cm layers. The average dry unit weight of the compacted sand was 17.5 kN/m³, with about 6% moisture content at the time of compaction. Average void ratio of sand was 0.62.

A 1 m long concrete beam, of 15x15 cm cross section, was embedded across the bin, as shown in Figure 3. The depth of embedment of the bottom of the beam was 30 cm. The position of the beam, the source and twelve near receiver locations were defined in a way to minimize effects of waves reflected from sides and the bottom of the bin. The receiver spacing for all receiver positions was kept constant and equal to 5 cm. Because of the relatively high frequency range of interest, 200-600 Hz, and a small receiver spacing,
accelerometers were used. The soil bin and the test setup are shown in Figure 4.

Dispersion curves for the receiver pair locations in front and above the obstacle show fluctuations similar to those observed in numerical results. This is illustrated in Figure 5 by dispersion curves for near receiver locations 6 and 7, and in the plot of the dispersion surface in Figure 6. It can also be observed in Figure 6, that phase velocity fluctuations vanish as the receiver pair passes the obstacle. Measured phase velocity fluctuations, of the order of 50%, correspond well to the same obtained from numerical results and experimental results by Al-Shaye et al. (1994) for a soil with very soft obstacle (cavity).

During the course of testing, significant effort was given to minimization of effects of waves reflected from the wall and the bottom of the bin. However, it is believed that those effects were not completely eliminated, as it can be observed at near receiver locations 1 to 3. A new 4.2 m diameter, 2 m deep, soil bin with an absorbing boundary layer of saw dust is now being constructed. It is expected that quantitatively much flatter results will be achieved in the new bin due to reduced reflections and the ability to do testing in a lower frequency range than in the current bin.

CONCLUSIONS

The proposed SWOD technique was implemented in an experimental setup and its operation verified in a soil with a buried rigid obstacle. The results confirm
numerical results that buried objects can cause significant surface wave reflections, thus producing fluctuations in the dispersion curve measured by a closely spaced receiver pair positioned in front and above the obstacle. While a good qualitative match was achieved between the experimental and numerical results, quantitatively better results are expected to be achieved in a larger bin and in the field. These are the future tasks of this ongoing research.

REFERENCES


ACKNOWLEDGMENT

This material is based on work supported by the National Science Foundation under Grant No. CMS-9622140.
SASW control of a vacuum consolidation on a sludge disposal

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ABSTRACT: For several years a vacuum consolidation was applied on a disposal of dredged sludge for
fastening the autoconsolidation; reclamation of the area in this way could start in a much earlier phase.
The disposal with an initial thickness of ± 8 m was covered with a sandlayer of 0.5 m and topped with an
HDPE cover. The vacuum consolidation caused an average settlement of ± 2 m. Since initially it was not
allowed to cut open the HDPE for environmental reasons, it was decided to use the non-destructive
Spectral-Analysis-of-Surface-Waves method for evaluation of the degree and homogeneity of the
consolidation. The paper presents the results of the evaluation of the stiffness of the sludge by the SASW
method which lateron were compared with laboratory test results. This SASW campaign showed that the
consolidation was quite homogeneous in the disposal, although still a very low stiffness was measured.

1 INTRODUCTION

The Spectral-Analysis-of-Surface-Waves (SASW) method is a non-destructive seismic method which
has been used in situ to determine the elastic moduli of soils and pavements at low level of
strain and the variation of these moduli with depth.

The test is based on the dispersion of Rayleigh-waves which mean that Rayleigh waves of
different wavelengths propagate at relatively dif-
ferent depths. If the medium of propagation is
vertically inhomogeneous, then the different
wavelengths propagate at different phase vel-
cities. This variation of phase velocity with
wavelength is called a dispersion curve and is
related to the structural stiffness of the medium of
wave propagation.

Once the field dispersion curve has been
determined, it is used to calculate the stiffness
profile at the site using an inversion algorithm.
Inversion allows detailed profiles of shear wave
velocity to be determined at sites with very simple
to very complex stiffness profiles.

Shear wave velocities of each layer of the
system are directly converted into a stiffness
modulus. These relationships form the bases for the
use of seismic methods to assess in situ material
parameters.

Any significant stiffness change due to for
instance the vacuum consolidation can be directly
determined without recourse to empirical
correlations by measuring the dispersion curve.
Because field testing involves measurement of
surface waves, all testing is performed on the
ground surface. This non-invasive nature of the
SASW test and the fact that it is based on stress
wave propagation makes it ideal for the evaluation
of the consolidation of the sludge underneath the
HDPE cover. The easy handling of the test, its
non-invasive character and the simplicity of the
required field equipment assured that the SASW
test was quite cost-effective.

2 THE SASW METHOD

2.1 Theoretical background

Surface waves used in the SASW method are the
vertically polarised Rayleigh waves. These are
seismic waves that travel along the exposed surface
of any solid system. These waves have particle
motion that decreases with depth into the system.
The depth of wave motion is determined by the
wavelength (or frequency). At low frequency, henc-
e long wavelength, waves extend deeper into the
system than at high frequency, hence short wavelength, waves. This property is illustrated in fig. 1.

![Diagram](image)

Figure 1. Distribution of vertical practical motion with depth for two surface waves of different wavelengths

Rayleigh waves with short wavelengths propagate through the surface layer; their velocity will only be determined by the properties of that layer. On the other hand, longer wavelength Rayleigh waves propagate through the top several layers, and their velocities will be determined by the combined properties of the layers through which they propagate. Conclusion, in layered media, the velocity of propagation of a surface wave depends on the frequency (or wavelength) of the wave. This variation of velocity with frequency is called the dispersion. Therefore, all layers in the profile can be sampled simply by generating surface waves over a wide range of wavelengths (i.e. a wide range in frequencies) and the velocities will vary with the stiffness and thickness of the layers in the system. The objective in the field testing of the SASW method is to measure this surface wave dispersion.

Surface wave velocity \( V_s \) of a material is closely related to the shear wave velocity \( V_g \) of the material. The surface wave propagates at a velocity slightly less than the shear wave velocity. The relationship between surface wave velocity and shear wave velocity depends on Poisson’s ratio \( \nu \).

For values of Poisson’s ratio between 0.1 and 0.3, surface wave velocity can be approximated by:

\[
V_s = 0.9 V_g \quad \text{for} \ 0.1 < \nu < 0.3
\]

From the theory of elasticity, values of the shear modulus can be calculated from shear wave velocity and mass density \( \rho \):

\[
G = \rho V_s^2
\]

\[
E = 2G(1 + \nu)
\]

where \( G \) is shear modulus and \( E \) is Young’s modulus.

2.2 Testing procedure

The general configuration of the source, receivers, and recording equipment is shown in fig. 2. Surface waves are generated by applying a dynamic vertical load to the ground surface. The propagation of these waves along the surface is monitored with two receivers placed at distances \( d_1 \) and \( d_2 \) from the source.

![Diagram](image)

Figure 2. Source-receiver configuration

The most common types of sources are either simple hammers (small, hand-held hammers or sledge hammers) or drop weights varying from 200 to 1500 N. Electromagnetic vibrators in conjunction with sinusoidal or random input motion can also be used as sources.

A dual channel Fast Fourier Transform (FFT) dynamic signal analyser is used to record and analyse the motions at both two transducers. The ability to calculate transforms rapidly in the field,
is an essential part of the SASW method, allowing operators to immediately assess the quality of the data being collected and, if necessary, modify the arrangement of source and receivers or other test parameters accordingly. This data can be easily transferred to a microcomputer for further analysis.

2.3 Analysis procedure

For each source/receiver spacing, the time histories recorded by the two receivers, \( s(t) \) and \( y(t) \), are transformed to the frequency domain resulting in the linear spectra of the two signals, \( X(f) \) and \( Y(f) \). The cross power spectrum of the signals, \( C_{xy}(f) \) is then calculated by multiplying \( Y(f) \) by the complex conjugate of \( X(f) \). In addition to the cross power spectrum, the coherence function and auto power spectrum of each signal are also calculated. It must be emphasized that all of these frequency domain quantities are calculated in real-time by the waveform analyzer. The key data are the phase of the cross power spectrum and the coherence function. The coherence function represents a signal-to-noise ratio and should be nearly one in the range of acceptable data.

The phase of the cross spectrum represents the phase difference of the motion at the two transducers. The surface wave velocity (\( V_s \)) and the wavelength (\( \lambda_s \)) can be determined from the phase of the cross spectrum (\( \phi_{xy}(f) \)) using the following expressions:

\[
\phi_{xy}(f) = 0 \quad \text{or} \quad 2\pi f
\]

where the phase angle is in radians and the frequency, \( f \), is in Hertz. The surface wave phase velocity, \( V_s \), is determined using:

\[
V_s(f) = (d_x - d_y)/\phi_{xy}(f)
\]

and the corresponding wavelength of the surface wave is calculated form:

\[
\lambda_s = V_s/f
\]

The result of these calculations is a dispersion curve (\( V_s \) versus \( \lambda_s \)) for a given receiver spacing. Individual dispersion curves for all receiver spacings are assembled for the composite dispersion curve for the site. In case of a infinitely uniform layer at constant material stress, the dispersion curve is a vertical line at constant value of Rayleigh wave velocity versus depth. For a layered system in which stiffness changes with depth, an inversion process is required to obtain the stiffness profile from the measured dispersion curve. This requires a velocity profile to be assumed, and a theoretical dispersion curve to be calculated for that profile. The theoretical dispersion curve is then compared to the measured one, and the assumed profile is adjusted in an attempt to improve the match. This procedure is repeated until the theoretical and measured dispersion curves closely match at which time the assumed profile is taken to represent the stiffness profile in the material system.

Application of inverse theory to surface wave testing has increased the accuracy of resulting wave velocity profiles and has significantly expanded the variety of sites of which the SASW method can be successfully applied.

3 EVALUATION OF THE VACUUMCONSOLIDATION

3.1 SASW measurements

After three years of vacuumconsolidation on a disposal of dredged sludge, the question was put forward whether the consolidation could be stopped and the area reclaimed. The disposal had an initial thickness of ± 8 m and was covered with a sand layer of about 0.5 m. To maintain the vacuum the disposal was topped with an HDPE cover of 2 m thickness and submerged with 0.5 m water. During consolidation, settlements of about 2 m occurred.

For environmental reasons it was not allowed to open the HDPE cover, so it was decided to use the SASW method for evaluation of the already obtained degree and homogeneity of the consolidation.

However, there was some doubt on the applicability of the SASW test on top of the HDPE; finally the environmental agency approved to open the cover at two places. After pumping off the water mass, nine SASW profiles were performed on the disposal in one day: two directly on the sand and seven on top of the HDPE cover. Figure 3 shows the dimensions of the disposal and the places and directions of the SASW profiles. In order to assure a good contact between the HDPE cover and the sand, small but
heavy steel plates were put on the surface underneath the geophones. These two seismic
geophones were consecutively placed at spacings of 0.5; 1; 2; 4; 8 and 12 m. Elastic stress waves
were then generated by the impact of a sledge hammer on an equal distance from the first re-
ceiver at the corresponding receiver spacing.

Figure 3. Plan of sludge disposal and SASW pro-
files

For each receiver spacing, five or more signals
were averaged in the frequency domain where the
phase of the cross-spectrum and coherence were
also calculated. The phase delays and receiver
spacing were then used to calculated the Rayleigh
wave velocities as a function of frequency using
equations (3), (4) and (5). The procedure was
repeated for all the spacings listed above. At the
end of testing, dispersion points from the indi-
vidual spacings are combined into a single
dispersion curve for that well chosen location.

To evaluate the stiffness of the profile, the
dynamic shear modulus and young modulus were
calculated using equations (1) and (2).

As an example, figure 4 is showing the experi-
mental and theoretical dispersion curve of the
measurement at profile F together with the
corresponding layer thickness and stiffness. On the
horizontal axis one can see the phase velocity
which is an indication for the stiffness of the
material; the wavelength on the vertical axis can
be directly related to depth.

The phase velocity is the highest at the small
wavelengths because of the stiffness of the top
sand layer. As greater depths the phase velocity is
decreasing because of the low stiffness of the
sludge. Finally the bottom of the disposal and the
natural soil profile are causing again an increase in
the phase velocity. The theoretical inversion is
showing a sand layer of 0.65 m thickness with a
dynamic shear modulus of 24 MPa. Underneath
there is a 5.4 m thick sludge layer with a maxi-
mum shear modulus of 3 MPa. The natural soil
profile shows a G1 modulus of 80 MPa.

The two SASW measurements performed
directly on the sand showed that the presence of
the HDPE cover had no influence on the final
results. The shear wave velocity of the sludge
varied from 37 m/s to 47 m/s within the nine
measurements profiles.

Conclusion of this measurement campaign was
that the previous consolidation was quite homo-
ogeneous in the disposal; a maximum shear mo-
dulus of 2 to 3 MPa however is still a too low a
value after 3 years of vacuum consolidation.
Therefore it was decided to perform a CPT test
and a boring with undisturbed soil sampling at the
place were the HDPE was cut open. Laboratory
tests confirmed the still quite low deformation and
shear characteristics of the sludge material.
3.2 Laboratory and in situ tests

Figure 5 shows the results of the electrical cone penetration test: below the sand layer very low $q_c$ values are measured with an average value of 0.25 MPa. Three undisturbed samples were taken at a depth of 1 m, 4 m and 7 m and the physical and mechanical laboratory tests were performed. Some of these results are also mentioned in figure 5: the volume weight of the sludge is 14 kN/m$^3$ and the water content varies between 60 and 70 %. A plastic limit of 39 % and a liquid limit of 82 % give a plasticity index of 43 %. The undrained cohesion out of the vane tests is 5 to 9 kPa.

![Figure 5. Laboratory and in situ test results on a sludge disposal](image)

The results of this CPT test and laboratory tests can be compared with the measurement results of the SASW profile E which shows a shear wave velocity of 37 m/s in the sludge. Assuming a value for Poisson's ratio of 0.3, which is perhaps somewhat low, and using equations (1) and (2), leads to a maximum shear modulus of 2 MPa and a constrained modulus M of 7.2 MPa. The constrained modulus out of the oedometer test results suggested 1.4 MPa; the maximum modulus therefore appeared to be about 5 times higher. This is of course due to the higher strainlevels in an oedometer test in comparison with the very low strainlevel in the SASW test.

Mayne and Rix (1993) estimated $G_{max}$ out of the cone resistance $q_c$ as:

$$G_{max} = 2.78 q_c^{0.315}$$

With an average $q_c$ of 0.25 kPa, $G_{max}$ in such way becomes 4.41 MPa which is overestimating the measured value of 2 MPa by 120 %. This had to be expected because of the wellknown generally valid difficulty of linking cone resistance to soil deformation parameters.

4 CONCLUSIONS

The SASW method is a valuable tool for characterising geotechnical sites and soil improvement techniques. Sample data of the case study presented herein illustrates the testing and analysis procedures at a site with very different material properties and stiffness.

A vacuum consolidation on a dredged sludge disposal was controlled by SASW testing and showed a low but quite homogeneous stiffness of the sludge. These conclusions were confirmed by the laboratory tests.

Shear wave velocities of each layer of the system were directly converted into a stiffness modulus. The easy handling of the test and the simplicity of the required field equipment moreover assured that the SASW test was quite cost-effective.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the valuable assistance of Silt N.V. during the in situ testing.

REFERENCES

Mayne, P.W. and Rix, G.J., $G_{max}$-$q_c$ relationships for clays, Geotechnical Testing Journal, Vol 16, No 1, March 1993, pp. 54-60.
The use of seismic geophysics in the characterization of a weak rock site

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ABSTRACT: The paper describes aspects of the site investigation of a weak rock site carried out in preparation for the construction of a retaining wall. Emphasis is given to the role of the geophysical surveys which complemented the more conventional characterisation techniques used. It is shown that, despite several factors which rendered the site a particularly challenging one for seismic testing, the geophysical component of the investigation contributed to both the characterisation and understanding of the site. Site-specific and general problems associated with the use of seismic methods to determine stiffness profiles in weak rock are examined and discussed.

1 INTRODUCTION

In design, weak rocks fall between the camps of soil and rock engineering, and designers are often overly cautious when assigning design parameters to these materials. Designs for structures in weak rocks are therefore typically over-conservative and uneconomic. Appropriate characterisation of the stiffness properties of weak rocks is a fundamental step in the resolution of this problem. This paper considers the practical application of one category of potential characterisation techniques, seismic geophysics, in the determination of design stiffness parameters for weak rocks.

A stiffness profile derived from seismic geophysical testing represents the mass stiffness at very small strain; that is, it incorporates the effects of fractures and other discontinuities on the overall behaviour of the material. In consequence of the bondless nature of weak rock deposits, such materials can be expected to behave in a linearly elastic manner for relatively high stress excursions from the in-situ stresses. It is therefore anticipated that a stiffness profile derived from geophysics will represent a realistic estimate of the in-situ ground stiffness, and so may be used in the geotechnical design of structures in weak rocks and in the numerical modelling of deformations around such structures.

The construction of an instrumented wall retaining weak rock offered a valuable opportunity to investigate these suppositions. Seismic surveys were conducted in addition to the "conventional" ground investigation, with the aim of determining whether stiffness profiles derived from geophysics provided a better estimate of design parameters for structures in weak rocks. This paper describes the techniques used to obtain the stiffness profile. Several aspects of the site made it a technically demanding location for geophysical surveying, and emphasis is placed on the practical difficulties and potential pitfalls associated with the techniques used. The assessment of the applicability and usefulness of the derived stiffness parameters, which will be carried out via back-analysis of the movements of the instrumented wall, is not within the scope of the present paper.

2 PROJECT AND SITE

The construction of the retaining wall is part of a road scheme, started in 1995, to link the M6 motorway with Coventry city centre. The new road, a dual carriageway, runs north-south through Coventry (Warwickshire) and will pass through residential areas to the east of the city centre. Approximately 1.1 km of the road is being constructed in a cutting of an abandoned railway line. The cutting, believed to be 80-90 years old, is required to be widened at its base in order to accommodate the new road (Figure 1). For this, retaining walls having a total length of 1.8 km are
being used. The section of wall of interest comprises a contiguous bored-pile wall with a stabilising toe slab to minimise deformations. Inclinometers, embedded strain gauges and pressure cells have been installed within the wall section, in order to monitor the post-construction deformation behaviour of the wall.

The ground conditions at the site are complex and are marked by strong vertical heterogeneity. The geology comprises loose ground overlying glacial deposits and moderately to highly weathered mudstones, sandstones and siltstones. These sequences form part of the Bromsque Sandstone Formation, of Triassic age, and the Coventry Sandstone Group, of Carboniferous age. The sedimentary strata display marked variation in the degree of weathering; the material varies locally between residual soil and competent rock. The groundwater table is several metres beneath the base of the cutting although, during trial pitting, seepage from perched water was experienced into the side slopes. Table 1 summarises the strata descriptions from the boreholes situated nearest to the northern (BH 220) and southern (BH 229) end of the survey area. Further details of the site geology are described in Davies & Barton (1998).

### Table 1. Summaries of strata descriptions from boreholes in the vicinity of the seismic survey area.

<table>
<thead>
<tr>
<th>BH 220</th>
<th>DESCRIPTION SUMMARY</th>
<th>thickness (m)</th>
<th>level (mAOOD)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>moderate ground (not cored)</td>
<td>3.6</td>
<td>94.6</td>
</tr>
<tr>
<td></td>
<td>glacial till (not cored)</td>
<td>1.0</td>
<td>91.8</td>
</tr>
<tr>
<td>MODERATELY weathered SANDSTONE</td>
<td>2.1</td>
<td>96.6</td>
<td></td>
</tr>
<tr>
<td>very soft CLAY</td>
<td>0.4</td>
<td>82.7</td>
<td></td>
</tr>
<tr>
<td>moderately weathered SANDSTONE</td>
<td>0.1</td>
<td>87.35</td>
<td></td>
</tr>
<tr>
<td>slightly weathered SANDSTONE</td>
<td>0.2</td>
<td>86.83</td>
<td></td>
</tr>
<tr>
<td>moderately weathered SANDSTONE</td>
<td>3.2</td>
<td>86.46</td>
<td></td>
</tr>
<tr>
<td>very soft CLAY with gravel bands</td>
<td>3.4</td>
<td>81.93</td>
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</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>BH 229</th>
<th>DESCRIPTION SUMMARY</th>
<th>thickness (m)</th>
<th>level (mAOOD)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>clay yard and gravel fill (not cored)</td>
<td>4.3</td>
<td>96.47</td>
</tr>
<tr>
<td>MODERATELY weathered SANDSTONE (not cored)</td>
<td>1.9</td>
<td>88.22</td>
<td></td>
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<tr>
<td>highly weathered SANDSTONE</td>
<td>0.4</td>
<td>87.82</td>
<td></td>
</tr>
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<td>0.3</td>
<td>87.17</td>
<td></td>
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<tr>
<td>slightly weathered SANDSTONE</td>
<td>3.2</td>
<td>84.15</td>
<td></td>
</tr>
<tr>
<td>slightly weathered SANDSTONE</td>
<td>2.3</td>
<td>81.92</td>
<td></td>
</tr>
<tr>
<td>slightly weathered SANDSTONE</td>
<td>0.6</td>
<td>81.32</td>
<td></td>
</tr>
<tr>
<td>slightly weathered SANDSTONE</td>
<td>0.3</td>
<td>81.27</td>
<td></td>
</tr>
<tr>
<td>soft to very soft CLAY</td>
<td>1.4</td>
<td>79.52</td>
<td></td>
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<tr>
<td>slightly weathered SANDSTONE</td>
<td>0.6</td>
<td>79.22</td>
<td></td>
</tr>
<tr>
<td>moderately weathered sandy</td>
<td>0.6</td>
<td>79.22</td>
<td></td>
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<td>slightly weathered SANDSTONE</td>
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<tr>
<td>slightly weathered SANDSTONE</td>
<td>0.5</td>
<td>79.07</td>
<td></td>
</tr>
</tbody>
</table>

3 SITE CHARACTERISATION METHODS

**In-situ** methods were the predominant characterisation techniques used at the site, principally because weak rocks are difficult to sample. The characterisation methods used are considered here within the categorisations of non-geophysical and geophysical.

3.1 Non-geophysical methods

Initial ground investigations in 1988 and 1989 along the corridor of the road included boreholes, trial pits, in-situ tests, and laboratory tests on cored samples. These investigations provided information for the general project design. A further investigation was carried out in 1993, in order to obtain specific information for use in the design of the proposed retaining walls and bridge abutments. The later investigation employed rotary cored boreholes, trial pits and slit trenches on the cutting slopes, at the planned locations of the structures. These explorations yielded, *inter alia*, an indication of the interface between overlying drift and the weathered rock, and detailed weathering profiles of the rock. Values for the small-strain stiffness properties of the soil and rock were derived using self-boring pressure meters (both Cambridge and weak rock types) and a high pressure dilatometer. The laboratory testing included classification tests (moisture content, Atterburg limits, particle size distribution), one-dimensional consolidation tests on soil samples, and point load and unconfined compressive strength tests on rock samples.

![Figure 1. Schematic of the test site, looking north.](image-url)
3.2 Seismic geophysical methods

The 1993 ground investigation included seismic refraction surveys along the base of the cutting, to investigate the underlying rock. Additional seismic surveys were later carried out by the University of Surrey on the crest of the cutting at the location of the proposed instrumented wall (Figure 1). The space available on the cutting crest was only about 30 m by 2.2.5 m, which significantly restricted both access and potential survey configurations. Five collinear boreholes were installed at the crest. Each borehole was 20 m deep, with a grouted plastic liner of internal diameter 100 mm. Borehole vertically surveys were commissioned, so that the three-dimensional positions of downhole instrumennts could be determined to within at least 10 mm.

Six established seismic tests were deployed, each of which can provide a stiffness-depth profile. They were: (i) parallel crosshole, with vertically-polarized shear waves; (ii) continuous surface waves (refer to Matthews et al., 1996); (iii) downhole seismic profiling, using shear waves; (iv) spectral analysis of surface waves (Nazar and Stokoe, 1984); (v) shear wave refraction; and (vi) upheole seismic profiling. The principal equipment used for borehole-based work were a vertically-polarized shear-wave source and three-component geophones. The continuous surface wave source was a frequency-controlled vibration unit. The impact surface source was a "hammer and anvil" arrangement. Vertically-oriented surface geophones were used in the upheole and surface wave surveys. All data were recorded on a digital seismograph.

Of the six techniques applied at the site, only methods (ii) and (iii), as listed, gave comparable, self-consistent data. Various site-specific difficulties affected all the tests, but certain methods were more severely influenced than others. The most significant problem was the high level of ambient groundborne vibration or "noise". Movement of heavy plant on the old track bed, which was used as a haul road for construction vehicles, and nearby traffic on a busy public road both contributed to this problem. There was further significant corruption of recorded signals, particularly of many of the crosshole observations, by interference of 50 Hz frequency (see Figures 2(a) and 2(b) and, for comparison, Figure 3). This may have been due to groundborne vibration from a mains electricity substation located across the cutting. Transformers emit airborne noise, as a continual hum, and unless properly isolated, groundborne vibrational energy also. Alternatively, the 50 Hz "noise" on the seismograph recordings may have an electrical artefact. A nearby power line may have induced an alternating current in the "takeout" cable used to link the geophone sensors and the seismograph recorder. This problem is often encountered by geophysicists, especially in urban areas. Use of a notch filter at 50 Hz was found to be ineffective. Harmonics can be present in the seismic record and, moreover, notch filters can distort the sought-for signal (see, for

![Figure 2](image2.png)

Figure 2. (a) Example of a crosshole seismic record corrupted by noise at 50Hz; and (b) the frequency spectrum of the signal.

![Figure 3](image3.png)

Figure 3. Example of a downhole seismic record with a high signal-to-noise ratio.
example, Butler and Russell, 1993). The local topography of the site was also problematic. The presence of reflections from the free face of the cutting, which could interfere with direct wave arrivals, was suspected. Moreover, the sharp-angled geometry of a cutting is in itself troublesome, as it can cause localised amplification of waves through focusing by topographic features (Ohtsuki et al., 1984).

Results from the continuous surface wave (CSW) and downhole surveys are shown in Figure 4, as a profile of vertically-polarised shear wave velocities against depth. CSW tests actually provide measurements of Rayleigh wave velocities, and these must be corrected to shear wave values: the Rayleigh wave velocity in a material is, depending on Poisson’s ratio, about 90-95% of the shear wave velocity.

Both sets of results in Figure 4 are characterised by a relatively narrow clustering of data points. Given the problems of noise at the site, this is significant as it suggests that the appropriate seismic signals were consistently and successfully observed in each case. In Figure 4, the feature at about 4.5 m to 5 m below ground level corresponds with the deepest extent of the made ground and glacial tills noted in the records for nearby boreholes (Table 1).

4 DISCUSSION

Figure 5 represents the field results of Figure 4 in terms of shear modulus. In deriving these elastic moduli from the velocity data, it has been assumed that the ground has a uniform mass density of 2200 kg/m³. Figure 5 also shows the design envelope that was used in the design of the retaining walls. This generalised stiffness profile has two regimes. The upper, low stiffness model is applied to the glacial tills and very weak, weathered rocks that may contain bands of residual soil; the high stiffness model is applied to weathered rocks that may include weak and stronger bands. The design stiffness profile was developed before the seismic surveying took place and was based, in the main, on the pressuremeter and dilatometer results. These data, from tests taken across the project, are also included in Figure 5. Pressuremeter data are from the first unload-reload loop, averaged across the three arms, and are not strain corrected.

It is evident from Figure 5 that the stiffnesses from the seismic tests generally exceed those from the pressuremeter and dilatometer tests. This behaviour has been previously noted in soil, where stress-strain behaviour is markedly non-linear and stiffness is therefore strongly dependent upon the strain level at which it is measured, but in a weak rock good predictions of mass stiffness have previously been obtained from carefully-conducted geophysical tests (e.g. Matthews, 1993). Indeed, where saturating of weak rocks is widely spaced it has been found that tests carried out on relatively small volumes of ground yield higher stiffnesses than do geophysical tests, because the latter are able to reflect the reduced stiffness of the rock mass which results from the compressibility of the joints or fractures (Clayton et al., 1994).

The lower stiffnesses produced by pressuremeter and dilatometer testing probably have resulted from one or more of the following:

- a small component of non-linear stress-strain behaviour;
- in-situ anisotropy. Taking this factor in isolation, the field data indicate the unusual situation of horizontal stiffness (from the pressuremeter and dilatometer tests) being less than the vertical stiffness (obtained from seismic testing). It is possible that stress relief and weathering in the
area of the railway cutting could account for this behaviour;

- borehole disturbance, where pressuremeter or dilatometer tests were carried out in boreholes;
- bedding between the side of the test pocket and the pressuremeter sensor, which might be significant for a stiff material such as a rock, particularly where the rock is coarse grained.

Whatever the cause(s) of the differences, it is hoped that, in the future, instrumentation of the retaining wall will provide back-analysed values of stiffness which can be compared with those obtained from the various in-situ tests.

It can be seen (Figure 5) that the geophysical testing allowed many more determinations of stiffness to be made than could economically be achieved using conventional methods. Despite the very large number of stiffness determinations, the scatter of results obtained from seismic testing is acceptably small, even though the data were obtained in a particularly challenging environment. Qualitatively, it might be judged that the variance of the seismic results is far less than that of the pressuremeter and dilatometer results. Such a conclusion would be unwarranted, as these tests were carried out in a number of locations and, unlike the seismic tests, were not confined to a single part of the site.

5 CONCLUSIONS

Six methods of seismic surveying were applied at a "live" construction site. Several significant problems affected the viability of seismic testing at the site, including groundborne vibration from traffic and construction plant, limitations of access and associated restrictions on survey sites and geometries, electromagnetic interference and the specific topography of the site. Despite these obstacles, two of the six methods of testing applied gave self-consistent results: the continuous surface wave method, and downdip seismic profiling using vertically-polarised shear waves.

Shear moduli derived from the seismic surveys were compared with the results of self-boring pressuremeter (Cambridge and weak-rock types) and dilatometer tests. The seismically-derived stiffness values generally exceed those from the latter tests. It is suggested that this difference may be attributable to various factors, acting severally or in combination:— the presence of a small component of non-linearity; anisotropy, perhaps due to stress relief at the face of the cutting; the effects of disturbance on tests in boreholes; the effects of bedding within the test pocket of the pressuremeter.

ACKNOWLEDGEMENTS

Site work was funded by the Engineering and Physical Science Research Council (Grant GR/K74395), as a project in association with colleagues at the University of Southampton who will undertake the back-analysis of the instrumented wall section. Permission to publish field data was kindly given by Babtie Group on behalf of Coventry City Council.

REFERENCES

Clayton, C.R.I.; M.A. Gordon, M.C. Matthews (1994) Measurements of stiffness of soils and weak rocks using small strain laboratory tests

483
and field geophysics. Proc. Int. Symp. on Pre- 
failure deformation Characterisation of 
Geomaterials, Sapporo, Japan, vol. 1, pp. 229-
234, Balkema

Davies, T.J., M.E. Barton (1998) Precise geological 
characterisation for the wider application of 
high quality site data First Int. Conf. on Site 
Characterisation, Atlanta

Matthews, M.C. (1993) The mass compressibility of 
faulted chalk PhD thesis, University of 
Stirrey

The use of surface waves in the determination 
of ground stiffness profiles Proc. Inst. Civil 
Engineers: Geotechnical Engineering, vol. 119, 
no. 2, pp. 84-95

velocities from spectral analysis of surface 
waves Proc. 8th World Conf. on Earthquake 
Engineering, vol. 3, pp. 31-38

of topography and subsurface inhomogeneity on 
seismic Rayleigh waves Earthquake Engineering 
and Structural Dynamics, vol. 12, pp. 37-58
Geophysical surveying methods for soft soil

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ABSTRACT: Geophysical surveying methods are becoming common in geotechnical problems involving very small strains. It has been reported that the wave velocities from different geophysical surveying methods may differ. This has raised questions as to the validity of the results. In this paper two geophysical surveying methods, the crosshole method and the downhole method, using both P and S waves will be discussed. Surveying results from two soft soil sites in Singapore will be presented and the dynamic moduli determined from the wave velocities will be compared. Finally comparisons with deformation modulus values derived from conventional soil tests will be made.

1 INTRODUCTION

Traditionally, geophysical techniques are used to map the ground subsurface particularly for inhomogeneities. Current understanding and advances in geophysical techniques has led the geotechnical engineer to exploit geophysical techniques in site investigation. One important physical property that can be determined by geophysical techniques is the ground stiffness. It has been widely accepted now that stiffness changes with strain levels and that geophysical techniques can yield the shear modulus at very small strain levels. This 'maximum' shear modulus \( G_{\text{max}} \) together with laboratory measurements of shear moduli at higher strain levels produces a shear modulus and shear strain relationship. Many studies have shown that in a number of field situations, soil behavior is governed by soil properties at small strains typically less than 0.1% (Simpson, et al. 1979; Burland, 1989). In the past, it is common to make field measurements of shear modulus at small strains using in situ loading tests. In the last decade or so, advances in geophysical survey technology has made geophysical surveys economical and effective to be considered in addition to existing site investigation methods.

The common geophysical methods used are seismic refraction surveys, seismic reflection surveys and the borehole seismic survey methods. Two other methods which have become increasingly popular in recent time are the surface wave method and ground penetration radar method. This paper will discuss the use of two borehole seismic survey methods, crosshole and downhole, at two soft soil sites in Singapore. Some concerns had been raised on the variability of the measured parameters when different seismic survey methods are used (Anderson, 1980; Powell and Butcher, 1991; Butcher and Powell, 1995; Rickelton et al., 1995). The paper will examine these concerns for the P and S wave velocities measured using both the crosshole and downhole methods made at the two soft soil sites. Finally, comparisons will be made with shear moduli determined from cyclic triaxial tests for larger strain levels.

2 GEOLOGY OF SITES

The geology of Singapore can be divided into four major formations: Jurong formation, Bukit Timah granite, Old Alluvium and Kallang formation. A geological map of Singapore is shown in Figure 1.
The two soft soil sites are located on the eastern part of the island in the Kallang formation. These two soft soil sites, labeled Site A and Site B, are indicated in Figure 1.

The soft clay of marine origin is the most distinguishing feature of the Kallang formation and has been well documented by Tan and Lee (1977). The marine clay frequently appears as an Upper Marine Clay and a Lower Marine Clay separated by a thin layer of intermediate stiff silty clay. The maximum thickness of the marine clay layer recorded is 35 m (Tan, 1983). The Upper Marine Clay is generally described as very soft to soft while the Lower Marine Clay is generally described as soft to medium stiff due to overconsolidation effects. The marine clay is usually normally consolidated to slightly overconsolidated. Due to the depositional history, the Upper and Lower Marine Clays have an average undrained shear strength of about 10 kPa and 40 kPa respectively. The intermediate stiff clay layer is overconsolidated with an undrained shear strength of 60 to 120 kPa. Tan (1983) had summarized the results of numerous laboratory tests on the Upper and Lower Marine clays.

The borehole logs of Site A and Site B are shown in Figure 2. At Site A, the Upper Marine Clay is about 10.5 m thick followed by a 1.7 m thick intermediate stiff clay and a 14 m thick Lower Marine Clay. At Site B, the Upper Marine Clay is about 9.8 m thick followed by a 3.3 m thick intermediate stiff clay and a 8.9 m thick Lower Marine Clay. The characteristics of the Upper and Lower Marine Clays at Site A and Site B are summarized in Tables 1(a) and (b) respectively. Index properties tests were also conducted for the intermediate stiff clay at Site B.

The intermediate stiff clay has a plastic limit of 20, plasticity index of 43, natural water content of 26.7%, bulk unit weight of 2.05 Mg/m³ and a specific gravity of 2.72. The marine clay properties are in general agreement with those reported by Tan (1983).

### Table 1a. Characteristics of marine clays at Site A.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Upper Marine Clay</th>
<th>Lower Marine Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic Limit</td>
<td>24</td>
<td>22</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>69</td>
<td>64</td>
</tr>
<tr>
<td>Natural w (%)</td>
<td>63</td>
<td>53</td>
</tr>
<tr>
<td>Bulk Unit Wt (Mg/m³)</td>
<td>1.69</td>
<td>1.76</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.68</td>
<td>2.68</td>
</tr>
</tbody>
</table>

### Table 1b. Characteristics of marine clays at Site B.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Upper Marine Clay</th>
<th>Lower Marine Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic Limit</td>
<td>25</td>
<td>22</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>65</td>
<td>59</td>
</tr>
<tr>
<td>Natural w (%)</td>
<td>62</td>
<td>50</td>
</tr>
<tr>
<td>Bulk Unit Wt (Mg/m³)</td>
<td>1.69</td>
<td>1.76</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.73</td>
<td>2.71</td>
</tr>
</tbody>
</table>

### 3 SEISMIC SURVEYS

#### 3.1 Downhole Tests

The downhole tests were conducted using a single borehole. The borehole was drilled and a PVC pipe...
was grouted to the borehole wall to ensure a good acoustic coupling. The borehole geophone consists of an inflatable rubber diaphragm and a vertical sensor and a horizontal sensor. When lowered to the correct depth, the rubber diaphragm was inflated to wedge the geophone against the borehole wall. A schematic diagram of the downhole test setup is shown in Figure 3. The source is located at the ground surface about 3 m from the borehole. P-waves were generated by striking a metal plate vertically with a heavy hammer. The P-wave was monitored using the vertical sensor. S-waves were generated by striking a wooden plank, orientated perpendicularly to the borehole and weighted down by heavy weights, horizontally at both ends. This causes one wave form to be the reverse of the other to be detected in turn by the horizontal sensor. Readings were taken as the borehole geophone were lowered at 1 m intervals throughout the depth of the borehole.

3.2 Crosshole Tests

In the seismic surveys at each site, three boreholes in a straight line at a center to center spacing of 3 m were used. Two ‘detector’ boreholes were initially drilled and its wall lined with a PVC pipe which was subsequently grouted to the borehole wall to ensure a good acoustic coupling. Drift loggings were performed on these two boreholes to determine the inclination and therefore the actual distances between the two boreholes. The third ‘source’ borehole was then drilled to the first measurement depth. A borehole geophone is then lowered into each of the detector boreholes to the first measurement depth. A core barrel is then lowered to the bottom of the source borehole and a P-wave is generated by striking the head of the core barrel as shown by the schematic in Figure 4. The source borehole is then advanced forward by 0.5 m and a pressuremeter is lowered to the measurement depth and inflated. S-waves were generated by striking the head on top of the rods upwards and downwards. The process was then repeated at 2 m intervals throughout the depths of the boreholes.

3.3 Seismic Survey Results

The results of the downhole and crosshole seismic surveys for Sites A and B are shown in Figures 5 and 6, respectively. Discussions, however, will be restricted to the marine clay layer which is the primary concern of this paper. The P-wave velocities from the downhole and crosshole seismic surveys showed less than 1% variation for Site A in all the marine clay layers. At site B, the P-wave velocities showed less than 1% variation for the Upper Marine Clay and Intermediate stiff clay layers and less than

Figure 3. Schematic setup of downhole seismic survey.

Figure 4. Schematic setup for crosshole seismic survey.
2.5% variation in the Lower Marine Clay. The S-wave velocities from the downhole and crossectional seismic surveys showed variations of 4%, 8.5% and 6% in the Upper Marine Clay, Intermediate stiff clay and Lower Marine Clay respectively for Site A. At Site B, the variations are respectively 2.5%, 8% and 13%. The S-wave velocity variation for the Intermediate stiff clay is consistent for both sites and the S-wave velocity variation for the Lower Marine Clay is consistently higher than the variation in the the Upper Marine Clay. The variation is due to the different S-waves used in the downhole and crossectional seismic surveys. In the downhole seismic survey, the S-wave is horizontally polarized at source and for the crossectional survey, the S-wave is vertically polarized at source. Another possibility of differences is due to anisotropy and the difference in stress history. The Intermediate stiff clay layer is overconsolidated and the lower marine clay layer has a much higher strength than the upper marine clay.

These observations are in general agreement with those reported by Powell and Butcher (1991) and Butcher and Powell (1995) for heavily overconsolidated London clay. The variations are, however, smaller than those reported in their papers.

The shear modulus \( G \) may be computed from the shear wave velocity \( V_s \), and the density of the soil \( \rho \) determined using:

\[
G = \rho V_s^2
\]  

(1)

It can be easily shown from Equation (1) that a variation of \( \pm \% \) in the shear wave velocity will lead to a variation of \( \pm \% \) for the shear modulus. Thus the shear wave velocity variation between downhole and crossectional seismic surveys will lead to shear modulus variations of 8%, 17% and 12% in the Upper Marine Clay, Intermediate stiff clay and the Lower Marine Clay respectively for Site A. Similarly shear modulus variations for Site B will be 5%, 16% and 26% in the Upper Marine Clay, Intermediate stiff clay and the Lower Marine Clay respectively.

4 CYCLIC TRIAXIAL TESTS

Undisturbed samples using thin-walled sampling tube of 76 mm diameter were obtained from the two sites. Cyclic triaxial test were performed on some of the samples. For Site A, soil samples at depths 12 - 12.65 m (mid-depth of the Upper Marine clay layer) and 24 - 24.9 m (mid-depth of the Lower Marine clay layer) were tested. For Site B, soil samples at depths 15 - 16 m (mid-depth of the Upper Marine clay layer), 21 - 22 m (mid-depth of the Intermediate stiff clay layer) and 33 - 34 m (mid-depth of the Lower Marine clay layer) were tested. The soil specimens were first consolidated to their in situ effective stresses and then cycled at strain levels of \( \pm 0.01\% \), \( \pm 0.02\% \), \( \pm 0.05\% \), \( \pm 0.1\% \) and \( \pm 1\% \). The soil specimens were tested at each strain level for 40 cycles at 0.5 Hz. At the end of 40 cycles at each strain level, the soil specimen was reconsolidate back to its initial effective stress before starting the next 40 cycles at the next strain level.

5 SHEAR MODULI AND SHEAR STRAINS

Typically, the shear strains associated with seismic surveys are less than 0.001% (Woods, 1978). The shear moduli determined from in situ shear wave
measurements are usually denoted as the maximum shear modulus $G_{\text{max}}$ (Matthews et al., 1996). The ratios of $G_{\text{max}}$ (shear modulus at 0.01% strain determined from the cyclic triaxial tests) and $G_{\text{max}}$ determined from downhole and crosshole shear wave measurements are tabulated in Table 2. It has been reported that the ratio of $G_{\text{max}} / G_{\text{max}}$ for clay is generally between 0.5 and 0.8 (Tatsumak and Shihaya, 1991; Clayton et al. 1994; Mukahi et al., 1994). From Table 2, it can be observed that the Intermediate stiff clay and the Lower Marine Clay agreed with the reported range.

Table 2. Ratios of $G_{\text{max}} / G_{\text{max}}$ for Singapore marine clays.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Site A</th>
<th>Site B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Marine Clay</td>
<td>0.42 (0.40)</td>
<td>0.34 (0.33)</td>
</tr>
<tr>
<td>Intermediate stiff clay</td>
<td>0.58 (0.51)</td>
<td></td>
</tr>
<tr>
<td>Lower Marine Clay</td>
<td>0.55 (0.63)</td>
<td>0.89 (0.78)</td>
</tr>
</tbody>
</table>
* Values in parenthesis indicates $G_{\text{max}}$ from crosshole tests.

A relationship between shear modulus and shear strain for clay has been suggested by Hardin and Druveich (1977). This relationship is given by:

$$G = \frac{G_{\text{max}}}{1 + \gamma_k^2}$$  

(2)

where $\gamma_k$ is the hyperbolic strain given by

$$\gamma_k = \left[\frac{\gamma}{\gamma_k^0} \right]^{1 + \frac{a}{b}}$$  

(3)

where $\gamma_k$ is the shear strain, and $a$ and $b$ are constants.

The relationship given in Equation (2) may be used to form a shear modulus - shear strain curve using the $G_{\text{max}}$ determined from the seismic surveys and the shear moduli determined from the cyclic triaxial test. The relationships are shown in Figures 7(a) and (b) for Site A and Site B, respectively. Figures 7(a) and (b) indicate that Equation (2) is suitable for the Intermediate stiff clay and the Lower Marine Clay and is less suitable for the Upper Marine Clay. The figures also showed that Equation (2) is not sensitive to $G_{\text{max}}$ for shear strains greater than 0.01%.

6 CONCLUSIONS

Downhole and crosshole seismic surveys were carried out at two soft marine clay sites in Singapore. The variations in the P-wave velocities were insignificant. However, more significant variations in the S-wave velocities were noted. The degree of variation is dependent on the soil types with the greater variations being found in the Intermediate stiff clay and the Lower Marine Clay. The variation in shear wave velocities will translate to a variation of two times in magnitude for the shear moduli. The Hardin and Druveich relationship for clay was used to give the shear modulus - shear strain curve using the $G_{\text{max}}$ from the seismic surveys.
and the shear moduli determined from cyclic triaxial tests. Despite the differences in $G_{max}$ from the downhole and crosshole seismic surveys, the relationships show no significant variations in shear moduli for shear strains greater than 0.01%.

ACKNOWLEDGMENTS

The work described in this paper is partially funded by a research grant from the National Science and Technology Board of Singapore, Grant No. NSTB 170/1.

REFERENCES


Geophysical/geotechnical characterization of upper Borrego Valley

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ABSTRACT: Borrego Valley is a small alluvial valley located near Palm Springs, California. This is the location of the Borrego Valley Downhole Array (BVDA), a dense, 3-D array of earthquake sensors designed to measure the response of the upper Borrego Valley to earthquake shaking. Extensive geophysical and geotechnical studies have been done at this site to characterize the geology and physical soil and rock properties of this site. Results from surface, borehole, and laboratory measurements were combined to create a master geotechnical/geological model of the upper Borrego Valley. Wave propagation modeling in progress uses this master model for various 1-D, 2-D, and 3-D modeling of earthquake shaking in the valley.

1 INTRODUCTION

Borrego Valley is a small alluvial valley 110 km northeast of San Diego, California (see Figure 1). In the upper (northern) part of this valley is the Borrego Valley Downhole Array (Kusko et al., 1997), an earthquake measurement array consisting of surface and downhole strong motion accelerometers (SMAs). An extensive investigation using geophysical and geotechnical methods was conducted in the upper Borrego Valley to develop a density and seismic velocity model for the shallow alluvial basin. The density and velocity structure model was needed in order to simulate 3-D earthquake wave propagation in the valley.

The study site is located in an alluvial valley bound to the west and east by the San Ysidro and Coyote mountains, respectively. The geophysical survey area has an aerial extent of about 20 square kilometers and occupies portions of the Clark Lake and Borrego Palm Canyon, California, United States Geological Survey (USGS) 7.5 minute quadrangles. Generalized geology in the vicinity of the site is summarized in Figure 2. Several boreholes drilled to depths of over 230 meters at the site revealed that alluvial sediments consist of coarse grained sand and gravel. Crystalline rocks of three distinct ages occur in the study area and include regionally metamorphosed pre-batholithic rocks, plutonic rocks of the Cretaceous Southern California batholith, and a series of cataclastically deformed rocks of post batholithic age named the Santa Rosa cataclastic...
zone (Sharp, 1967.) The northwest to the southeast trending Coyote Creek fault, is located along the western side of Coyote mountain (Figure 2.) This fault is a branch of the San Jacinto fault zone, one of the most active faults of the San Andreas fault system in southern California.

2 FIELD INVESTIGATIONS

2.1 Land Surveying

Seventy gravity stations were surveyed along the horizontal array using a total station system with
estimated horizontal and vertical accuracy of about 1 centimeter (cm). An additional 557 gravity stations were surveyed on an approximately 100-by-400-meter grid using a dual frequency differential GPS system operated in kinematic mode. These stations are estimated to have 1-2 cm horizontal accuracy and 2-3 cm vertical accuracy. All other geophysical measurements were tied to the surveyed gravity stations. The locations of all geophysical stations are shown in Figure 3. This figure also shows the locations of the SMAs along the horizontal array and the location of the BVDA which is denoted as the Main Station.

2.2 Borehole Measurements

Borehole measurements have been made in a 220 meter (m) borehole at the Main Station, a 30m borehole at Station K, a 100m borehole at Station J, and a 270m water well (Oasis Well) drilled within the survey area. The locations of these boreholes are shown on Figure 3.

P and S-wave velocity measurements were made at 1-2m intervals using the suspension method (Njögbor and Imai, 1994). Borehole velocity measurements at the BVDA Main Station are presented in Figure 4. In this log, P and S-wave velocities generally increase with depth in the alluvial sediments. S-wave velocities range from about 300 meters per second (m/s) near the surface to 750 m/s at a depth of 225 meters. P-wave velocities range from about 450 m/s near the surface to 1,150 m/s immediately above the water table at 95 meters. Within the saturated zone, P-wave velocities range from about 1,850 m/s at a depth of 100 meters to 2,200 m/s at a depth of 225 meters.

At a depth of about 230 meters P and S-wave velocities abruptly increase at the basement surface. Long and short normal resistivity, SP, and natural gamma logs were also obtained in the Main Station and Oasis Well boreholes. These data were used to constrain the TDEM modeling. Soil and rock samples were obtained from several of the boreholes and laboratory tested for index properties, including density. Dynamic triaxial testing was done on selected soil samples to measure the nonlinear soil strength and dynamic damping.

2.3 TDEM Survey

Thirty-five TDEM soundings were made in the upper Borrego Valley at locations shown in Figure 4.

Figure 4. Oyo Suspension Log at Main Station

Figure 5. Model at TDEM sounding at Station 2484

3. Twenty-four of these soundings were conducted along the horizontal array and the remaining 11 soundings were conducted north of the array along the west and east sides of the valley.

TDEM sounding data were modeled using 1-D forward and inverse algorithms. Where possible a three layer resistivity model with layers for unsaturated alluvium, saturated alluvium, and basement were fit to the field data. Modeled depths to ground water and basement at sounding locations near the Main Station and Oasis Well agreed very well with actual depths from the boreholes. A typical TDEM model (Station 2484, near the Main Station) is presented in Figure 5.

2.4 Seismic Surveys

A seismic reflection survey was conducted along 16
Figure 6. Basement Elevation Contour Map

Figure 7. Groundwater Elevation Contour Map
lines (SR1-SR16) at locations shown in Figure 3. Each line consisted of 48 geophones spaced 20 feet (6.1 meters) apart for a total line length of 940 feet (286.5 meters). A seismograph source was used, with three shot points per line - a center shot and forward and reverse end shots.

Seismic refraction data were processed using the slope intercept method (Telford et al., 1990). This simple interpretation method assumes that all subsurface velocity layers are planar and that no lateral velocity variations exist, a valid assumption for the water table beneath a short line.

Interpreted P-wave velocities of unsaturated sediments varied from about 325 m/s near the surface to as high as 1,150 m/s immediately above the water table. Interpreted P-wave velocities of the saturated sediments determined from the seismic refraction data varied from about 1,800 to 2,250 m/s. These velocity ranges agree with those determined from suspension logs at the Main Station (Figure 4) and Oasis well.

Seismic reflection was attempted along the horizontal array. Small (10kg) explosive sources were used in addition to a weight drop source. Only the water table was imaged by this reflection survey; more source energy would have been required to image the bedrock interface.

2.5 Gravity Survey

An extensive microgravity survey was conducted to image the bedrock in the survey area. A LaCoste & Romberg Model D gravity meter was used. Gravity measurements were made at 557 stations along a 100- by 400-meter grid and 70 stations along the horizontal array. The locations of the gravity stations are shown in Figure 3.

An absolute gravity base station established by University of California, Riverside (UCR) at the Peg Leg Smith Monument (PEGLEG), located about 10 km southeast of the survey area, was used for absolute gravity control during this investigation. Five local base stations were established within the survey area and tied to PEGLEG with 5 measurement loops. All gravity measurement loops began and ended at a local base station so that corrections for instrument drift could be made.

The gravity data were reduced and modeled using the following steps: convert measurements from dial divisions to relative gravity in milligals, average multiple measurements at each station, apply instrument height correction, remove tidal effects, apply drift corrections, calculate absolute gravity, calculate theoretical gravity, apply free air correction, apply Bouguer correction, apply terrain correction. The result of applying the above corrections is the complete Bouguer anomaly.

Gravity data from this investigation were combined with regional data from the State of California database prior to calculating the regional gravity field. The residual gravity anomaly was calculated by subtracting the complete Bouguer anomaly data from the regional field.

A depth inversion of the residual gravity anomaly was made using algorithms developed at the University of California at Riverside. The algorithms generate a depth model assuming a hyperbolic increase in density with depth. The hyperbolic density-depth function and resulting effective density is determined using the method of Litinsly (1989) with input information being the density contrast between near surface sediments and basement, the known depth to basement at a point and associated residual gravity anomaly, and residual gravity anomaly where the basement outcrops (depth equal to zero). The known depth to basement at the Main Station was used as control for the modeling. Errors in calculated depth using this method are expected to be on the order of 15 percent.

2.6 Synthesis

Results of the surveying, borehole measurements, TDEM soundings, seismic surveys, and microgravity survey were combined to create a master model of the upper Borrego Valley. This model, with a grid spacing of 25m, consisted of:

- surface elevations
- basement elevation (Figure 6)
- water table elevation (Figure 7)
- P-wave velocity
- S-wave velocity
- density

Surface elevations were controlled by the survey results and extended using the USGS digital elevation model (DEM) for this region. Basement elevation was controlled by the microgravity survey and constrained by borehole and TDEM results. Water table elevation was estimated using results of borehole, seismic refraction/reflection, and TDEM surveys. Velocities were estimated throughout the grid based upon borehole surveys. Densities were estimated throughout the grid using measured sample test values and the hyperbolic model used in the gravity inversion.
3 DISCUSSION

State-of-the-art geophysical and geotechnical methods were used to create a master model of the upper Borrego Valley. By synthesizing the results from several different types of surveys, an accurate picture of the structure of the valley and of the physical properties was created. Such a synthesis is needed, as single geophysical methods may contain bias or other errors unrelated to the structure.

This extremely detailed site characterization provides an accurate master model with sufficient resolution for modeling of earthquake wave propagation in the valley. Simulations will be compared with actual earthquake data recorded by the Borrego Valley Downhole Array.

4 ACKNOWLEDGEMENTS

This study was part of an ongoing study of earthquake response of soil sites funded by Kajima Corporation and the Nuclear Power Engineering Corporation of Japan. Assistance by Takuaki Konno and Setsuo Iizuka is especially appreciated.

REFERENCES


GEOSIS: Integrated approach of geotechnical and seismic data for offshore site investigations

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J. Meunier – IFREMER, Brest, France

ABSTRACT: The objective of the GEOSIS Project is to improve the correlation between geophysical and geotechnical data by integrating offshore seismic and geotechnical investigations. In the framework of this project, two new techniques for site investigation were developed and tested, i.e. multichannel very high resolution seismic surveying and vertical seismic profiling in geotechnical boreholes. These techniques are now finalized and can be used on field. They allow the processing software, routinely used in oil exploration surveys, to be applied, in particular VSP and reservoir assessment tools. The results of such site investigation enable first to derive geotechnical data from seismo-acoustic measurements and secondly to quantitatively extrapolate consistent geotechnical data around a borehole. In this way, they significantly improve the understanding of the stratigraphy. These techniques should stand as a major improvement in future offshore site investigation practice.

INTRODUCTION

Site investigation practice in the offshore oil industry today involves two steps. An initial seismic survey is generally performed. It is followed by a geotechnical investigation including one or several boreholes at the location of the future structure. These two surveys are quite distinct and the resulting seismic and geotechnical data are generally difficult to integrate and cross-correlate, involving engineering issues whenever the structure location is some distance away from geotechnical boreholes.

As an illustration, to date, high resolution seismic data are of single channel type and uses still frequently analog recording. Therefore seismic sections are presented as a function of time without possibility of processing. In contrast, geotechnical data are processed in terms of depth. Thus, due to this different approach, time or depth, a seismic section cannot be directly compared with a geotechnical profile. Moreover, the precise knowledge of the seismic velocity of each layer would be extremely important for the integration of seismic and geotechnical data and for improving the extrapolation of soil data around a geotechnical borehole.

The GEOSIS Project was initiated by a group of French research organizations (IFP and IFREMER), oil companies (Elf and TOTAL), and offshore contractors (ISM and GEODIA), all companies affiliated with CLAROM (Club for Research Activities on Offshore Structures). It aims at improving the correlation between geophysical and geotechnical data by integrating offshore seismic and geotechnical investigations.

It was found that it was necessary to develop two techniques (Nauroy et al., 1994a, b):
- multichannel very high resolution seismic surveying,
- vertical seismic profiling in geotechnical boreholes.

I. IMPROVED SOIL INVESTIGATION TECHNIQUES

1.1. Multichannel very high seismic surveying

In 1990, at the beginning of the project, specifications of a complete very high resolution multichannel system including streamer, source and recorder were defined. The expected goal was to obtain a spatial definition of about one metre with a penetration into the soil of about 100 metres. Now, a 24-channels system is available (Marsset et al., 1994). It was used on several sites, including an oil field.

Multichannel processing improve spatial resolution and depth penetration. Moreover, it gives access to an estimation of the P-wave velocities of the soil (Meunier et al., 1994, 1997).
1.3 Vertical Seismic Profiling in geotechnical boreholes.

Besides the multichannel very high resolution seismic surveying, another way to obtain the velocities is to perform a Vertical Seismic Profiling. A seismic signal transmitted from a source located at the surface is recorded by a probe containing a set of geophones successively located at different depths inside the borehole.

Each trace on the Figure 1 corresponds to a position of the VSP probe inside the borehole. VSP profiles recorded in reservoir seismic surveys provide several types of information.

First, the velocities can be obtained by picking of the first arrivals. Secondly, further processing of the subsequent reflected seismic events after the first arrivals allows the down-going waves and the up-going waves to be separated, the primary reflections and the multiple reflections to be distinguished. In fine, seismic sections on the site passing through the borehole can be calibrated.

The Vertical Seismic Profiling techniques can be adapted to offshore geotechnical boreholes. P-wave source can be immersed over the side of the vessel for example or close to the sea bottom. S-wave source can be installed below the seabed frame to obtain a good coupling with the soil.

The receivers can be of two types. The first type is a standard VSP probe, which contains a set of three geophones, one vertical and two horizontal. This kind of probe must be correctly coupled to the soil. This implies to operate it under open hole conditions inducing of course the limitations of the measurement with the standard VSP probe to hard soils where the borehole stability is guaranteed. The second type is a seismic cone penetrometer with also a triaxial set of geophones. The seismic cone is basically usable in any formation where the penetration of the tip is possible (Robertson, 1986, De Lange, 1991). Moreover the jacking of the seismic cone with a several ton force gives a good coupling with the soil, allowing a P-wave frequency bandwidth up to 1200 Hz.
1.3. Offshore field tests

These techniques were first tested offshore Monaco, on a site with a water depth of 50 m. Towards the end of 1994, the French oil company TOTAL also used the GROGIS techniques on a Far East field. The surveys included very high resolution seismic campaigns and several geotechnical borings including seismic cone testing for VSP profiling.

The picking of the first arrivals on P and S-wave signals allows travel times to be estimated and time to depth relationship to be obtained (Fig. 2). The geotechnical logs such as CPT log can be recompensed as a function of the two-way travel time and thus directly compared to the surface seismic section (Nauroy et al. 1994).

The use of both P and S-wave velocities (V_p and V_s) presents an obvious interest for the geotechnical engineer. Dynamic bulk and shear moduli or Young's modulus and Poisson's ratio can be derived from V_p and V_s (Nauroy, 1995). The extra cost due to the use of these techniques is estimated to represent less than 20% of standard geotechnical soil surveys.

2. DATA PROCESSING

2.1. VSP Processing

Despite the presence of the drill string, P-wave signals recorded by the vertical geophone of the seismic cone are of high quality with a good signal to noise ratio. This good quality allows the standard VSP processing procedure used in oil exploration seismic surveying to be applied.

Figure 3 shows the results of such processing applied to seismic cone data.

The vertical axis represents the two-way travel time of the standard seismic section. The horizontal axis represents the position of the probe below the mudline. The soil description is given above. At the start of each trace, the small horizontal line represents the picking of the first arrival. It provides a direct correlation between true depth and the
position of the reflector expressed in time, without assuming velocity values. It is a classical processing used in oil industry.

This chart is a useful tool for determining the geotechnical meaning of each seismic event. It is obvious that powerful reflectors do not necessarily always correspond to identified geotechnical interfaces.

For example, on Monaco site, some indurated, but very thin beds (without geotechnical interest) located in the upper layer give strong seismic reflectors. On the other hand, the geotechnical interface between the lower layers does not clearly appear as a seismic event (Fig. 3).

Using this chart, the surface seismic sections are calibrated and it is possible to present a picture of the subsoil where each layer corresponding to different nature of soil characteristics.

The velocity laws obtained from the multi channel seismic data can be calibrated. A good estimation of velocity values along the seismic section can give the thickness of each layer and also the geotechnical characteristics through the use of relationships between geotechnical and seismic data.

2.2 Correlation between seismic and geotechnical data

One of the objectives of the GEOSIS project was to study the possible correlation between the P and S-wave velocities or impedances and geotechnical engineering properties of soils.

Impedance profile can be obtained by using a sonic log (if any) and density measurements or by inversion of the VSP data.

For example, Figure 4 shows the correlation which was found between P-wave impedance and cone resistance qc on Monaco field. This correlation is not surprising since both cone resistance and impedance can be considered as parameters that integrate bulk properties of the soil. However, this relationship is probably highly site dependent. Other site results have to be examined on a case by case basis rather. General rules cannot be drawn presently.

2.3 Towards a CPT cross section

The good quality of VSP data obtained in geotechnical boreholes makes it possible to consider the application of stratigraphic deconvolution techniques routinely used in oil reservoir studies.

The inversion techniques used consist in transforming a surface seismic section in a map acoustic impedance by using a wavelet previously calibrated with VSP data recorded in a borehole located in the same plan as the seismic section.

Initial tests with this technique were made on a surface seismic section obtained in offshore Monaco. We first construct an acoustic impedance cross section, the "a priori stratigraphic model" obtained from the interpretation of the seismic section (Fig 5). Each trace of the impedance section is transformed in reflection coefficients, then convolved by a wavelet obtained by inversion of the VSP data. By this way, a synthetic section can be obtained. This synthetic seismic section is compared to the real section. Then a residual section is calculated which is used to correct the initial model and so on. The objective is to minimise the residual section. This
Fig. 5 Structural model obtained from the interpretation of surface seismic section, Monaco site

Fig. 6 Example of acoustic impedance cross-section obtained by inversion.

Work was performed by using the IFP software INTERWELL.

After inversion, a map of acoustic impedance is obtained (Fig. 6). If relationships are known between acoustic impedance and standard geotechnical parameters, such as CPT cone resistance for example, a cross section of such parameters can then be obtained. Work is presently continuing on this subject and initial results are very encouraging.
CONCLUSIONS

Both multichannel seismic surveying and vertical seismic profiling techniques are necessary for the integration and the calibration of geotechnical and seismic data. Standard acquisition and processing procedures used in exploration seismic surveying can be used. These techniques open the way to the extrapolation of soil data around a geotechnical borehole with an extra cost estimated to represent less than 20 % of standard soil surveys.

At this stage of the project, specifications for industrial integrated soil surveys can be proposed including tools operations, data recording and processing methods in order to obtain an accurate time-depth relationship and an efficient procedure for the identification and the meaning of the reflectors.

ACKNOWLEDGEMENTS

The "GEOSIS" research project is sponsored by CLAROM (CSUB for Research Activities on Offshore Structures) in France. CLAROM, IFP, IFREMER, ELF, TOTAL, GEODIA and ISM are acknowledged for granting the permission to publish this paper.

REFERENCES


Simultaneous inversion of surface wave velocity and attenuation

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ABSTRACT: The Spectral Analysis of Surface Waves (SASW) test is a non-invasive field technique which
uses the dispersion properties of surface waves to determine the shear modulus profile at a site. Recently, the
SASW test has also been used to estimate the shear damping ratio profile. In this paper, we present a
systematic and efficient procedure to simultaneously determine both the shear modulus and damping ratio
profiles from SASW test data. Simultaneous inversion of surface wave velocity and attenuation data is
superior to the traditional uncoupled analysis. First, it considers the effect produced by material damping on
the phase velocity of seismic waves; secondly, it improves the well-posedness of the inverse problem
associated with the interpretation of surface wave data. Application of the new procedure is illustrated at
Treasure Island NGES. The shear wave velocity and shear damping ratio profiles obtained from simultaneous
inversion compare well with other independent experimental data.

1 INTRODUCTION

The use of surface (Rayleigh) waves for geotechnical site characterization has several advantages over more conventional seismic methods
such as crosshole and downhole tests. The most attractive feature of surface wave tests is that they
are non-invasive which permits the tests to be performed more rapidly and at lower cost than most
other invasive methods. At sites where subsurface conditions (e.g. gravelly soils) or environmental
concerns (e.g. solid waste landfills) hinder the use of boreholes and probes, surface wave tests
constitute an excellent choice for an in situ site investigation.

The most common application of surface wave methods is to determine the shear wave velocity
profile at a site. Recently, Rix et al. (1997) developed a method to calculate near-surface values
of material damping ratio from measurements of the spatial attenuation of harmonic Rayleigh waves.
However, until now the two problems of determining the shear wave velocity and shear damping ratio profiles at a site have been considered
separately and therefore uncoupled. The objective of this paper is to present a different approach to the
problem, where Rayleigh wave velocity and attenuation data are inverted simultaneously. The
following sections describe the theoretical basis of
this new procedure and illustrate its application to
data acquired at the Treasure Island National
Geotechnical Experimentation Site (NGES).

2 THEORETICAL BACKGROUND

Rayleigh wave propagation in elastic, vertically
heterogeneous media is described mathematically by
the following linear eigenvalue problem (Aki and
Richards, 1980):

$$\frac{df(y)}{dy} = \alpha(y) f(y)$$

(1)

where $f(y)$ is a 4-element column vector composed
of the horizontal and vertical components of the
displacement and stress eigenfunctions, $\alpha(y)$ is
a 4 x 4 matrix whose elements are functions of the
wavenumber, mass density, and elastic moduli of a
soil deposit. These two latter quantities are function
of the depth y. Equation 1 must be supplemented
with appropriate boundary conditions, which are
vanishing stress at the free surface of the half-space,
and the radiation condition at $y \to \infty$.
2.1 Uncoupled Analysis

The formulation of the boundary value problem associated with the propagation of surface waves requires an assumption regarding the constitutive law used to describe the mechanical behavior of the medium. Linear elasticity is the most obvious choice and is of interest for applications where only the shear wave velocity profile is to be determined. Inelastic constitutive laws were first considered by Audomson et al. (1965). They developed an approximate solution which is applicable in weakly dissipative media; in their solution the surface wave dispersion and the attenuation curves are computed separately. This uncoupled analysis forms the basis of the procedure implemented by Rix et al. (1997) to determine the material damping ratio profile of a site.

The uncoupled analysis suffers two important limitations. The first is that it disregards the experimental evidence that seismic wave velocities of dissipative media differ from those of corresponding elastic media (Schwab and Kropff, 1971), and therefore should not be determined independently. The second limitation is related to the well-posedness of the two separate inverse problems of determining the shear wave velocity and shear damping ratio profiles. Simultaneous inversion of both dispersion and attenuation data not only results in a single inverse problem, but it also adds a useful internal constraint. This constraint is justified by the Cauchy-RIemann equations which are satisfied by the Rayleigh phase velocity once it is viewed as an analytic, complex-valued function of the complex variable shear wave velocity. This constraint is embedded in the formalism of the non-linear inversion for the complex-valued shear wave velocity profile.

2.2 Coupled Analysis

A simple constitutive model which accounts (from a phenomenological point of view) for the most important dissipation processes occurring at small deformations in geological materials is linear viscoelasticity. An attractive feature of this theory is the so called “elastic-viscoelastic correspondence principle” (Christensen, 1971). According to this principle, the solution of a linear elastic, boundary value problem can be easily converted into the solution of the corresponding boundary value problem in linear viscoelasticity. In particular, for steady state harmonic problems the elastic solution can be extended (by analytic continuation) to the viscoelastic case by considering complex-valued elastic moduli. This procedure is equivalent to using complex-valued seismic wave velocities:

$$V_s = \frac{V_s}{(1 + iD_s)}$$

where $D_s = \sigma, P, R$ denotes the shear, compression, and Rayleigh wave phase velocities, respectively. In this paper $V_s$ is also denoted by $\varepsilon_s$.

From the mathematical point of view, application of the correspondence principle to the elastic eigenproblem (Eq. 1), leads naturally to a complex eigenproblem for the viscoelastic case. The technique used to solve the complex eigenvalue problem is the generalization of the method developed by Chen (1993) and Hinada (1998) for finding the normal modes in a multilayered elastic half-space. A crucial step of this generalization is the computation of the roots of the Rayleigh dispersion equation (or Rayleigh secular function). In the elastic case, the Rayleigh secular function is real-valued and its (real) roots may be obtained by a root-bracketing technique combined with iteration.

In the viscoelastic case the Rayleigh secular function is complex-valued, and therefore a different technique is needed to find its (complex) roots (Abd-ElBaal, 1970, Lai, 1998). We have employed an elegant algorithm which takes advantage of the analytic (holomorphic) properties of the Rayleigh secular function with respect to $\varepsilon_s$ in the domain of interest. This algorithm is based on Cauchy’s residue theorem from complex analysis written in the form (Abd-ElBaal et al., 1970):

$$G_N = \frac{1}{2\pi i} \oint \frac{z^N}{f(z)} dz = \sum_{\nu=0}^{N} P_{\nu} S_{\nu}$$

where the integral sign denotes integration along a positively oriented, closed contour $\Gamma$. $1/(f(z))$ is an analytic function inside and on $\Gamma$ except at the points $z_\nu$ where it may have isolated singularities; $z \in C$. $P_{\nu}$ denotes the residue of the function $1/(f(z))$ at the point $z_\nu$. Finally, $m$ denotes the total number of isolated singularities of $1/(f(z))$ inside $\Gamma$ and $N = 0$. If now we identify $f(z)$ with the Rayleigh secular function $R(\varepsilon_s)$, and the contour $\Gamma$ as the boundary of the region $\mathcal{D}$ where the roots of $R(\varepsilon_s)$ are located, we have the basis of a procedure that can be
used for determining the roots of $\mathcal{P}(\xi_0)$. First, we evaluate the complex numbers $G_{0r}$ via the contour integral defined by Eq. 3. An admissible parameterization of the contour $\Gamma$ is required for the actual implementation of the algorithm (Lai, 1998); the numerical integration is carried out using a classical, fifty-point Gauss-Legendre quadrature formula.

From the knowledge of the numbers $G_{0r}$ it is possible to find the coefficients of the complex polynomial $p_r(z)$:

$$p_r(z) = c_0 + c_1z + c_2z^2 + \ldots + c_{m-1}z^{m-1} + z^m$$  \hspace{1cm} (4)

by solving the linear system of equations that can be constructed using the modified Newton identities (Abd-Elale et al., 1979):

$$\sum_{r=0}^{m} G_{0r} \cdot c_r + G_{1r} = 0 \quad r = 0, \ldots, m - 1$$  \hspace{1cm} (5)

The zeros of $p_r(z)$ coincide with the zeros of $\mathcal{P}(\xi_0)$ inside $\Gamma$ which are the (complex) Rayleigh wave phase velocities associated with the solution of the complex eigenproblem.

2.3 Solution of the Inverse Problem

The non-linear (complex) functional relationship existing between the Rayleigh and the shear wave phase velocities for a specified set of frequencies can be written as follows:

$$\bar{\xi}_k = \mathcal{E}_k(\bar{V}_s)$$  \hspace{1cm} (6)

where $n_f$ is the number of frequencies, and $\mathcal{E}_k$ is an $n_f \times 1$ vector of complex Rayleigh wave phase velocities which are computed using Eq. 2 and the expression:

$$D_k = \mathcal{\xi}_k \cdot \mathcal{\xi}_k^*$$  \hspace{1cm} (7)

where $\mathcal{\xi}_k$ is the Rayleigh wave attenuation coefficient and $\omega$ is the circular frequency. If the soil deposit is characterized by $n_l$ homogeneous layers then $\bar{V}_s$ is an $n_l \times 1$ vector containing the complex shear wave velocities of the layers also defined by Eq. 2. Equation 8 is an explicit definition of the nonlinear forward problem associated with the prediction of the experimental complex Rayleigh wave phase velocities from an assumed complex shear wave velocity profile.

To solve the implicit inverse problem represented by Eq. 6, we expand the latter in a Taylor series about an initial complex shear wave velocity profile $\bar{V}_s$ and ignore higher order terms. The result is the linearized form of Eq. 6 about $\bar{V}_s$:

$$\frac{\partial \mathcal{E}}{\partial \bar{V}_s} \cdot \delta \bar{V}_s = \frac{\partial \mathcal{E}}{\partial \bar{V}_s} \cdot \delta \bar{V}_s$$  \hspace{1cm} (8)

where $\delta \bar{V}_s$ is the $n_f \times 1$ vector formed by the complex Rayleigh wave phase velocities obtained solving the forward problem defined by Eq. 6 with $\bar{V}_s = \bar{V}_s$. The term $\frac{\partial \mathcal{E}}{\partial \bar{V}_s}$ is the $n_f \times n_f$ Jacobian matrix whose elements are the complex partial derivatives $\frac{\partial \xi_{k,m}}{\partial V_{a,l}}$, for $k = 1, n_f$; $l = 1, n_l$. The partial derivatives are obtained using the variational principle of Rayleigh waves extended by analytic continuation (Aki and Richards, 1980; Lai, 1998), which leads to a closed-form expression for $\frac{\partial \mathcal{E}}{\partial \bar{V}_s}$. Equation 8 forms the basis of a classic least squares algorithm to determine the complex-valued shear wave velocity profile. We have chosen a more robust approach based on a constrained, weighted least squares algorithm (Constable et al., 1987) whose strategy can be summarized as follows: given a set of experimental complex Rayleigh phase velocities $\mathcal{E}_k$ and their associated uncertainties $\sigma_k$, find the optimum values of $\bar{V}_s$ that maximize the smoothness of the resulting complex shear wave velocity profile while predicting the experimental phase velocities with an acceptable accuracy. This approach is motivated by the observation that inversions performed with classic least squares techniques often lead to physically unreasonable profiles of model parameters. This algorithm requires a definition of smoothness or its converse roughness of a candidate solution (in our case the complex $\bar{V}_s$ profile). In a layered soil profile with complex-valued model parameters, roughness may be defined by the following expression (Lai, 1998):

$$R_k = \sum_{a=1}^{n_l} \left( \left| \mathcal{E}_k(\bar{V}_{a,l}) - \mathcal{E}_k(\bar{V}_{a+1,l}) \right| \right) \text{conj} \left( \mathcal{E}_k(\bar{V}_{a,l}) - \mathcal{E}_k(\bar{V}_{a+1,l}) \right)$$  \hspace{1cm} (9)

where $\text{conj}(\ldots)$ denotes complex conjugation.
The misfit between the measured and the predicted complex Rayleigh phase velocities may be written as follows:

$$
\epsilon' = \left| \mathbf{W} \mathbf{e}^m - \mathbf{W} \mathbf{e}_h \end{array} \right|^2 \left| \mathbf{W} \mathbf{e}^p - \mathbf{W} \mathbf{e}_h \end{array} \right| \right|^2
$$

where $\mathbf{W}$ is a complex diagonal $n \times n$ matrix:

$$
\mathbf{W} = \text{diag} \left\{ \| \mathbf{e}_1 \| / \sigma_1, \| \mathbf{e}_2 \| / \sigma_2, \ldots, \| \mathbf{e}_n \| / \sigma_n \right\}
$$

and $\left\{ \cdot \right\}^H$ denotes the Hermitian transpose. The solution of the linearized inverse problem represented by Eq. 8 consists of finding a vector $\mathbf{V}_h$ that minimizes $\epsilon'$ with the constraint that the residual error $\epsilon'$ be equal to $\epsilon^m$ (an acceptable value considering the uncertainties). The method of Lagrange multipliers is employed to solve this complex constrained minimization problem resulting in (Lai, 1998):

$$
\mathbf{V}_h = \left[ \mu I + \left( \mathbf{W}_h \mathbf{V}_h \right)^{-1} \mathbf{W}_h \right]^{-1} \left( \mathbf{W}_h \mathbf{V}_h \right)^{-1} \mathbf{W} \mathbf{e}_h
$$

where $\mu$ is the Lagrange multiplier which may be interpreted as a smoothing parameter, $\mu$ is a $n \times n$ matrix defining the two-point central finite difference operator and $\mathbf{d} = \mathbf{J}_{\mathbf{e}_h} \mathbf{V}_h + \left( \mathbf{e}_h - \mathbf{e}^m \right)$ (Constable et al., 1987). Equation 12 is used iteratively to refine the estimated complex shear wave velocity profile until convergence. The smoothing parameter $\mu$ must be chosen so that the acceptable residual error $\epsilon'^m$ is matched with a vector $\mathbf{V}_h$ composed of negative imaginary part only. This additional constraint assures that positive values of the hysteretic damping ratio are obtained.

Finally, it is important to point out that for a given frequency, the Rayleigh secular function may have multiple roots as a consequence of the existence of several modes of propagation. Thus the Jacobian matrix $\mathbf{J}_{\mathbf{e}_h}$ and the Rayleigh phase velocities $\mathbf{e}_h(\omega)$ are calculated with regard to a particular mode, in this case the fundamental mode. For the Treasure Island NGES site presented in this paper, the use of only the fundamental mode is considered appropriate. For more irregular, inversely dispersive shear wave velocity and shear damping ratio profiles, an analysis which includes the contributions of higher modes is required (Robb et al., 1991).

3 TREASURE ISLAND NGES

Treasure Island is a man-made island constructed of hydraulic fill soils in the eastern portion of San Francisco Bay. The area investigated for this study is along the western property margin of the National Geotechnical Experimentation Site (NGES). Extensive geotechnical characterization of the site was performed for the EPRI (1993) study and included soil borings, penetration tests, and in situ seismic tests. The soil conditions consist of approximately 10 m of loose, fine to medium sand underlain by 12 to 18 m of soft clay (Bay Mud). Beneath the Bay Mud are stiffer soils (Older Bay Mud) which are underlain at depths on the order of 80 m by bedrock. The water table is approximately 1.5 m below the ground surface.

4 TEST PROCEDURE AND RESULTS

At the Treasure Island NGES, the experimental Rayleigh wave phase velocities were calculated from the cross power spectrum of two receivers separated by a distance $\Delta x$:

$$
\epsilon_h(\omega) = \frac{\omega \Delta x}{0(\omega)}
$$

where $0(\omega)$ is the phase of the cross power spectrum in radians. Phase velocities from several different receiver spacings were combined to form a composite experimental dispersion curve.

The experimental Rayleigh wave attenuation coefficients were obtained from measurements of the vertical particle displacement, $|v(t,\omega)|$, at several receiver offsets. The attenuation coefficients were obtained from a nonlinear regression based on the following (Rix et al., 1997):

$$
|v(t,\omega)| = F_w Q(\omega) e^{-\alpha(\omega)}
$$

where $F_w$ is the magnitude of the source, $Q(\omega, \alpha)$ is a function which models the geometric attenuation of Rayleigh waves in a heterogeneous medium, and $\alpha(\omega)$ is the frequency-dependent Rayleigh wave attenuation coefficient.

The inversion algorithm described above was applied to the complex-valued experimental phase velocities derived from the composite dispersion and attenuation curves using Eqs. 2 and 7. Figure 1 illustrates the convergence of the inversion.
algorithm in terms of the Rayleigh wave dispersion and attenuation curves. The experimental curves are shown as dashed lines in the figure. Theoretical dispersion and attenuation curves starting with the first iteration are also shown. The theoretical curves corresponding to the sixth and final iteration are in reasonable agreement with the experimental curves.

The corresponding sequences of shear wave velocity and shear damping ratio profiles are shown in Figure 2 where the dashed lines indicate the starting models used in the inversion. The final shear wave velocity and shear damping ratio profiles are shown using bold lines.

Figure 3 shows the convergence of the algorithm in terms of the root-mean-square (rms) error between the experimental and theoretical complex phase velocities; the convergence of the algorithm after only six iterations is apparent.

Finally, the results obtained from surface wave measurements with the simultaneous inversion are compared with independent in-situ and laboratory measurements of shear wave velocity and damping ratio in Figure 4. The values of shear wave velocity compare well with other SASW and crosshole test results. Values of shear damping ratio from surface wave tests are generally less than those obtained from crosstie damping measurements, possibly because of frequency and apparent attenuation effects (Rix et al., 1997). As expected, the shear wave velocity and shear damping ratio profiles are similar to those resulting from the uncoupled analyses.
5 CONCLUSIONS

Surface wave tests are non-invasive field techniques which can be used for the determination of the shear wave velocity and the shear damping ratio profiles at a site. We have presented a systematic and efficient procedure for the simultaneous inversion of Rayleigh wave experimental dispersion and attenuation data. The simultaneous inversion is an elegant procedure which accounts for the coupling between hysteretic damping ratio and seismic wave velocity. It also enhances the degree of well-posedness of the non-linear inversion problem associated with surface wave measurements. The new procedure has been applied at the Treasure Island NGES site where independent in situ and laboratory measurements were available for comparison. The shear wave velocity and shear damping ratio profiles determined from the simultaneous inversion agree reasonably well with the independent measurements.

6 REFERENCES


Seismic velocity in cohesionless granular material deposits

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ABSTRACT: Seismic velocities within in-situ cohesionless granular material deposits obtained during reflection seismic investigations with corresponding grain size distributions and other geotechnical data are analyzed and characterized using a Percolation Theory (PT) approach. Elastic moduli, as quantified by PT, varies universally as a function of the difference between the material porosity and a threshold porosity more generally described as the percolation threshold. In-situ material deposit elastic moduli are estimated from in-situ seismic p-wave velocities, and porosities are estimated from particle distributions and measurements or estimates of in-situ material deposit densities. Two characteristic elastic modulus behavior, physical gel and chemical gel, exist in PT. Dense, well graded cohesionless granular materials tend to behave as physical gels. Looser, uniform poorly graded cohesionless granular materials tend to behave as chemical gels. Cemented soils and intact, porous rock may also tend to behave as chemical gels.

1 INTRODUCTION, EMPIRICAL OBSERVATIONS

During geotechnical investigations, primarily of stream crossings for infrastructure projects, the author has collected and correlated cohesionless granular material mean particle size D₅₀ data with refraction seismic p-wave velocity data. Details of the field investigation methodologies and previous analysis of the data is presented in Rucker (1996). This data has been subdivided into sets above and below the water table and into well graded or uniform, poorly graded particle size distributions, depending upon the uniformity coefficient Cu for each data point, as summarized in Figure 1. This paper presents a quantitatively based framework utilizing concepts and methods from Percolation Theory for the empirical relations previously presented. If this framework proves to be sound from a theoretical and engineering mechanics perspective, seismic methods may be able to provide more useful information for site characterization than is present.

2 PERCOLATION THEORY AND ELASTIC MODULUS

A detailed discussion of Percolation Theory is beyond the scope of this presentation. A brief summary and example applications in groundwater hydrology are provided by Berkowitz and Illeberg (1993). Saafier and Aharoni (1992) present the theoretical background and development of Percolation Theory, and Sahimi (1994) presents a variety of applications, including applications in rock fracture and fluid flow in porous media.
Sahimi describes percolation as occurring when a system or network becomes macroscopically open to a given phenomenon such as electrical resistance, permeability or elastic modulus. Such a network can be visualized as a lattice with points that are occupied or unoccupied, or connected or unconnected. Sahimi describes such points as sites. Occupied or connected lattice points are distributed randomly throughout the lattice with a probability \( P \). Below a critical threshold probability \( P_c \), insufficient lattice points are occupied or connected, and no complete pathways through the lattice are present for the percolation phenomenon to occur through. Above \( P_c \), sufficient lattice points are occupied or connected to provide pathways through the lattice for the percolation phenomenon to proceed. Near \( P_c \), the given phenomenon changes at a rate or magnitude that is proportional to the difference between the lattice occupation or connection probability \( P \) and the critical percolation threshold probability \( P_c \), raised to a power characteristic for the given phenomenon. The form of this relationship for elastic modulus is

\[
G = k (P - P_c)^f
\]

where \( G \) is elastic modulus and \( k \) is a proportionality constant. Having an exponent which is characteristic for a given phenomenon implies universality in percolation phenomena.

Sahimi (1994, Chapter 11) discussed and summarized percolation-based behavior of modulus in terms of the exponent \( f \). He described the behavior of a modulus \( G \) in terms of two basic types of gels, physical gels and chemical gels. In general, modulus in chemical gels is dominated by stretching or central forces acting through the elements of the network. Modulus in physical gels is additionally influenced by bond bending forces, which are similar to mutually touching particles rolling on top of one another during dekformation of the network. Sahimi reports that \( f \) is about 2.1 for chemical gels and about 3.75 for physical gels.

Applying Percolation Theory to explain some granular material behavior requires that the granular materials be described in terms of occupation or connection probability of network lattice points and that the phenomenon be described as elastic modulus. Porosity is a means to describe the probability of occupation for a network lattice. A 3-dimensional lattice can be visualized within a granular material. Lattice points falling within particles are occupied, and lattice points falling within void spaces are unoccupied. As the distance between lattice points is made small relative to the particle sizes, the percentage of lattice points that are unoccupied approach the porosity of the granular material mass. Elastic modulus of the granular material mass at low strain levels may be measured indirectly through wave propagation velocity.

3 ESTIMATING IN-SITU POROSITY AND MODULUS

Direct measurement of in-situ porosities was not practical or possible within the constraints of the site investigations from which the field data were obtained. Porosities therefore had to be estimated from available data. Knowing material dry unit weight and specific gravity, porosity could be calculated. Unit weight could only practically be measured at or near the surface in materials no coarser than gravelly sands above the water table. However, such measurements were not required as part of the site investigations, and thus was not available for the data set used in this discussion. Burnister (1963) presented a method to estimate dry unit weight from grain size distributions and relative density in cohesionless granular materials primarily in alluvial environments. Relative density was estimated utilizing standard penetration test results or the depositional environment. Burnister included characteristic grain size distribution curve types in the analysis. Results of these analyses for selected data, using a specific gravity of 2.67 to convert dry unit weight to porosity, are presented in Table 1.

Low strain modulus was estimated using the interpreted p-wave velocities. Poisson's ratio of 0.33 was assumed as a reasonable value for soils above the water table, resulting in estimated shear wave (s-wave) velocities of half the p-wave velocities. Given these p- and s-wave velocities, and the estimated unit weight of the granular materials, low strain Young's modulus was calculated (Visher, 1976). Because p-wave velocities are significantly influenced by saturation

| Table 1. Measured or estimated characteristics of selected samples referenced in Figures 1-5. |
|---|---|---|---|---|---|---|
| Sample | \( D_{50} \) | SPT | Relative Point | \( n_{Cu} \) | Density | Porosity |
| A - Colo. | 56 | 80 | 80 | 0.18 |
| B - Colo. | 19 | 14 | 14 | 0.19 |
| data above and below water table | | | | |
| C - Ariz. | 0.24 | 2.1 | 2.5 | 0.44 |
| data above and below water table | | | | |
| D - Ariz. | 0.6 | 5.8 | 10 | 0.40 |
| E - Ariz. | 0.2 | 4.7 | 16 | 0.39 |
| F - Ariz. | 0.5 | 20 | 0.4 | 0.28 |
| G - Nev. | 6 | 7.9 | 70 | |

Notes: (1) standard penetration test points a, b stream gradient = 10%
while p-wave velocities are not, Young's modulus was typically not estimated for relatively high porosity materials located below the water table. At very large \( D_{pw} \), the difference between p-wave velocities above and below the water table becomes less, as shown in Figure 1. Young's modulus was estimated for data points a and b below the water table, and for saturated sandstone at low porosities (Yin and others, 1993).

The resulting estimated moduli, although somewhat high, represent an upper bound for moduli estimates. Estimated Young's moduli compared to porosity for Figure 1 data above the water table, and data points a and b below the water table, are presented in Figure 2. Two sets of porous rock modulus versus porosity data for laboratory samples from the literature are also shown in Figure 2. Butcher (1987) presented laboratory determined modulus versus porosity for welded tuff. Yin and others (1993) presented laboratory determined p-wave velocity versus porosity for clean, saturated sandstone. Sandstone modulus values were estimated using the approach outlined above, but with a Poisson's ratio of 0.2. Yin and others also presented p-wave versus porosity data for saturated sand. This sand data was subject to the same limitations described above for high porosity data below the water table, and moduli were not estimated. Yin and others reported the existence of a "critical porosity" of about 0.39 where the p-wave velocity versus porosity trend in saturated sand and sandstones underwent a dramatic change.

Various trends in modulus-porosity relationships are apparent in Figure 2. A steep trend in the well graded granular materials indicates that modulus decreases rapidly with increasing porosity, and approaches zero at a porosity above about 0.4. Such behavior is consistent with a percolation threshold porosity below which modulus disappears. Uniform or poorly graded granular materials indicate a less steep trend of modulus decreasing with increasing porosity. The uniform or poorly graded materials data exhibit considerably greater scatter than the well graded materials, which could reflect limitations of the data or actual variations in material behavior. Welded tuff (Butcher, 1987) and saturated sandstone (Yin and others, 1993) laboratory data exhibit excellent trends of gradual modulus reduction with increasing porosity. With uniform or poorly graded materials, such behavior is consistent with a percolation threshold different from well graded materials.

4 WELL GRADED COHESIONLESS GRANULAR MATERIALS AS PHYSICAL GEL

Sahimi (1994) describes the modulus behavior of physical gels as a function of the central forces (CF) and bond bending (BB) forces acting on and between occupied points in the lattice of a percolating network. Physical gels are formed when a reversible association and no permanent chemical bond is present between particles or monomers. Sahimi cites submicrometer copperoxide-silver powder and silica aerogels as examples. BB forces are important if each point interacts with at least 6-12 nearest neighbor points in 3 dimensions. For a 3-dimensional network such as a cohesionless granular material deposit to exhibit BB forces and behave as a physical gel, points in the lattice must interact with at least three nearest neighbor points. When stress causes deformation of the lattice or cohesionless material deposit, any three particles that mutually touch roll on top of each other. Forces similar to BB forces are created from the motion and as their relative centers are displaced. The resulting modulus percolation exponent \( f \) is about 3.75 for physical gels.

![Figure 2. Estimated low-strain modulus versus porosity for data above water table and literature.](image1)

![Figure 3. Young's Modulus versus porosity for well graded granular material data as a physical gel.](image2)
The empirical relationship between elastic modulus estimated from p-wave velocity and porosity estimated from grain size distribution and relative density for the Figure 1 data sets above the water table with uniformity coefficient Cu > 6 is shown in Figure 3. Estimates ranges for ± 20 percent of the relative density and ± 20 percent of the seismic velocity are shown on selected data points. Through this data, a physical gel modulus versus porosity trend utilizing equation (1) was iteratively best-fit by hand using a spreadsheet. The critical percolation threshold porosity providing the best-fit to the data was 0.42, as shown in Figure 3. Given the inherent nature of the uncertainties in the field data and estimation procedures, the fit between the empirical data and the theoretical trend over two orders of magnitude of elastic modulus is encouraging. It should be noted that the author performed the porosity estimates several months before understanding the concept of physical gel in a percolation context.

5 UNIFORM, POORLY GRADED GRANULAR MATERIALS AS CHEMICAL GEL

Sahin (1994) describes the modulus behavior of chemical gels as a function of primarily central forces acting on and between occupied points in the lattice of a percolating network without significant contribution from bond bending forces. Chemical gels are formed when a permanent chemical bond is present between particles or monomers. Sahin cites hydrolyzed polyacrylamide and tetraethoxyorthosilicate reactions as examples. Rolling between mutually touching particles does not take place, so that bond bending forces are not important. Important forces are primarily central forces between particles or stretching forces through interparticle bonds. The resulting modulus percolation exponent λ is usually in the range of 1.9 to 2.2, with a typical value of about 2.1.

The empirical relationship between elastic modulus estimated from p-wave velocity and porosity estimated from grain size distribution and relative density for the Figure 1 data sets above the water table with uniformity coefficient Cu > 6, and for welded tuff and saturated sandstone from Figure 2, are shown in Figure 4. Estimates ranges for ± 20 percent of the relative density and ± 20 percent of the seismic velocity are shown on selected data points. Again, using a spreadsheet to iteratively fit the chemical gel modulus versus porosity trend to the data utilizing equation (1), the critical percolation threshold porosity was determined to be roughly on the order of 0.6. Data point c was considered to be an outlier and was ignored. An adequate range of moduli data was not available to estimate a threshold porosity with great confidence. However, the fit between the laboratory data for welded tuff and saturated sandstone, and the theoretical trend is very encouraging. Chemical gels are normally permanently bonded, why would uniform, poorly graded cohesionless granular materials behave like chemical gels? One conjecture is that in the loose packing conditions implied by high porosities, the particles are organized in a way that the rolling effect needed to create HB forces does not occur. If such forces are applied to the material structure, it resists in an inelastic manner by collapsing into a lower porosity structure. An oversimplified condition could be visualized as simple cubic packing of uniform spheres with porosity of about 0.48 as described by Lamb and Whitney (1969). By rolling layers of a simple cubic structure over each other, that structure would easily slip or collapse into a dense cubic packing structure with a porosity of about 0.26.

6 PERCOLATION THRESHOLD AND GRANULAR MATERIALS BELOW WATER TABLE

As described previously, estimates of elastic modulus for data below water table at relatively high porosities were not attempted. However, with the understanding that modulus and seismic velocity are interrelated, an indication of the presence of a percolation threshold in the saturating fluid phase can be demonstrated. Higher seismic velocity implies higher modulus, and lower seismic velocity implies lower modulus. Measured p-wave velocity and estimated porosities from Figure 1 data are presented in Figure 5. At relatively low and very high estimated porosities, the measured field p-wave velocities are above and relatively close to the p-
wave velocity of water. Velocities gradually increase as porosity decreases. This trend ceases, and measured field p-wave velocities fall significantly, where estimated porosities are in the vicinity of 0.4.

P-wave velocities in these saturated media are higher than in unsaturated media, as shown in Figure 1. P-wave velocities are largely a function of the saturating media, such as water, filling the pore space. To apply Percolation Theory to a saturated granular material, the relevant lattice may be the fluid-filled pore space rather than the granular particles. The bulk modulus behavior should follow that of a chemical gel, since bond bending forces would be irrelevant in a fluid. In this condition, the percolation threshold becomes 1.0-0.4, or a porosity of about 0.6 for p-wave transmission through the saturated pore spaces in the granular material volume. That threshold is similar to the percolation threshold porosity estimated for the chemical gel condition presented in Figure 4.

An alternative explanation for the low p-wave velocities below the water table would be the presence of entrained air in the water at volumes greater than about 0.1 percent. This condition causes a severe reduction in bulk modulus and p-wave velocity in the water. Its effect in relation to streambed materials is discussed by Hacker (1990). One potential weakness in this explanation is the concentration of low p-wave velocity data only in the vicinity of 0.4 porosity. This porosity is very close to the “critical porosity” of 0.39 reported by Yin and others (1993). A calculated seismic p-wave velocity and porosity relationship for quartz particles and water, based on the Wood equation as presented in Richart and others (1970), is also presented in Figure 2. At both the highest and lower porosities, the field p-wave and estimated porosity data correlate well with the Wood equation. It is in the vicinity of the apparent percolation threshold that the empirical field data are inconsistent with the Wood equation.

7 DISCUSSION AND CONCLUSIONS

Analysis based on concepts of Percolation Theory provides a quantitative framework to explain empirical correlations between field measurements of seismic p-wave velocity and particle size as shown in Figure 1. The author is using these relationships to assist with site characterization in stream environments and estimating parameters for analyses such as scour prediction and excavatability. These relationships are especially useful in cobble and boulder environments where other methods of exploration and sampling are frequently ineffective.

A Percolation Theory based analysis can provide other insights into possible granular material behavior. For example, according to the analysis presented here, a cohesionless granular material which behaves as a physical gel cannot have a porosity less than about 0.42. At a specific gravity of 2.67, the corresponding minimum dry density is about 1.55 g/cm³. Typical index properties for severely collapsing soils in the desert southwest (Beckwith, 1979), namely silty sands, clays, and sandy silts, include a dry density of less than 1.52 g/cm³. As a result of the deposition process for these soils, silt and sand grains become “tack welded” by clay packets or cementing constituents such as calcium carbonate. The “tack welds” are analogous to chemical bonding, and the soil structure is directly analogous to the structure of a chemical gel. Upon wetting the soil, the bonds between particles are dissolved or broken, and the soil mass collapses into a more mechanically stable configuration, analogous to a physical gel.

The analyses presented here are based on porosities estimated from basic geotechnical data. Further research might concentrate on further quantification, verification and refinement of these modulus-porosity relationships in soils. The large scale in-situ measurement capabilities inherent in refraction and other shallow seismic exploration techniques may then be more effectively exploited as tools for site characterization.

REFERENCES


Shallow control for high resolution seismic reflection surveys using piezocone penetrometer soundings

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ABSTRACT: Seismic reflection techniques are limited in their ability to image shallow unconsolidated sediments of the Carolina Coastal Plain. However, a combination of high resolution seismic reflection information coupled with piezocone penetrometer soundings has proven successful for correlating near surface stratigraphy and structure with deeper seismic information. An area at the Savannah River Site has been evaluated using the piezocone to constrain the shallow stratigraphy where seismic reflection data indicate structure within the deeper section.

1. INTRODUCTION

Obtaining high resolution seismic data, for the shallow or near surface geologic horizons of the Carolina Coastal Plain, is difficult due to the effects of source noise and highly variable geology. Where stratigraphy and structure of the deeper geology is acquired using seismic reflection, a technique for resolving the shallow stratigraphy is needed. The piezocone penetrometer test (CPTU) has been used to provide shallow stratigraphic control over high resolution seismic data.

An area at the Savannah River Site has been characterized using high resolution seismic reflection surveys as well as piezocone soundings and borings. In this area, deep structure was noted on the seismic records. A series of CPTU soundings were pushed to provide shallow stratigraphy over the area where structure was noted on seismic line APT2-3.

Stratigraphic calibration of both the piezocone soundings and the seismic reflection data was determined from down-hole geophysical logs and core data acquired from a deep boring drilled to basement. The CPTU interpreted stratigraphy and the deeper seismic data were merged into one surface to basement geologic model for the site. The CPTU stratigraphy was then used to evaluate the deep structure noted on seismic line APT2-3.

2. SITE CONDITIONS AND INVESTIGATION

The Savannah River Site (SRS) is located in the upper portion of the Carolina Coastal Plain of South Carolina (Figure 1). The investigation site is located approximately in the center of the SRS. The site consists of approximately 1000 feet of unconsolidated to semi consolidated Coastal Plain sediments overlying crystalline basement rock.

The area was investigated with four high resolution seismic reflection surveys, seventy piezocone soundings and one deep core boring with geophysical logging (Figure 2). An additional 13 piezocone soundings were pushed over the area where structure was noted in the deeper seismic data (Figure 2).

3. DATA INTERPRETATION

Analysis of the four seismic reflection profiles indicated an anomalous area located in the southern part of the investigation site on seismic profile APT2-3 (Figure 2). It was characterized from the seismic reflection survey as a broad warp on the basement and overlying stratigraphic units.

The shallow stratigraphy and structure in the sediments directly above the first seismic anomaly on seismic line APT2-3 was characterized from a closely spaced (approximately 100 feet) series of thirteen CPTU soundings along APT2-3 (Figure 2).

The shallow stratigraphy was determined from the deep core boring and geophysical logs and correlated with the nearest CPTU soundings (Figure 3). Cross correlation methods were then used to determine the site stratigraphy over the entire investigation area.
3.1 Deep stratigraphy

The deep stratigraphy (greater than approximately 200 feet depth) was determined directly from the deep core boring and geophysical logs. These geologic formations were correlated with nearby regional wells and considered to be representative of typical Coastal Plain geology. The deep stratigraphy throughout the investigation site was mapped using the five seismic reflection surveys. Surface contour maps of each geologic unit were used to compare the structure of the deep stratigraphic units with the shallow stratigraphy determined from CPTU data.

3.2 Shallow stratigraphy

The shallow stratigraphy (surface to approximately 200 feet depth) of the site consists of sediments deposited under a wide range of geologic environments including shallow marine, marginal marine, near shore to lagoonal, backwater environments and also low to high energy fluvial. Stratigraphic tops were interpreted from the CPTU measurements including friction ratio, tip resistance, sleeve friction and pore pressure measurements. Stratigraphic units mapped with the CPTU soundings include (from deepest to shallowest): the SanTEE Formation, Dry Branch Formation, Tobacco Road Formation and Upland Unit.

Two major erosional unconformities are present in the shallow subsurface and include the top of the SanTEE Formation. The top occurs at approximately elevation 165 feet to 185 feet Mean Sea Level (MSL) and represents a sea level regressive event. The Upland Unit is the shallow most geologic formation and consists of high energy fluvial deposits which irregularly incise into the underlying Tobacco Road Formation (Figure 4).

4 RESULTS AND CONCLUSIONS

The shallow or near surface geologic horizons of the Carolina Coastal Plain, have been characterized using the CPTU in an area where high resolution seismic data indicates structure within the deep section. The shallow stratigraphy determined from the CPTU soundings show relatively flat structure over a broad warp structure noted on the seismic. This indicates that either the anomaly is an imaging artifact, or if representative of real structure, is time bounded by the shallow stratigraphy (Figure 5).

The CPTU sounding provides resolution deep enough to tie with upper limits of the seismic data. This allows a basement to surface stratigraphic section to be constructed from the two data sets. Further, the CPTU measurements may be used to evaluate near surface conditions which can affect seismic acquisition such as highly variable soil conditions or water table gradients.
Figure 5. Correlation panel showing site stratigraphy from down-hole geophysics correlated to CPTU measurements and seismic reflection data from seismic line 3.
Figure 4. Correlation panel showing shallow stratigraphy determined from down-hole geophysics, CPTU friction ratio measurements and seismic reflection data.
Experience gained in testing pavements by spectral analysis of surface waves

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ABSTRACT: Two sections of a major highway in Croatia were tested at 462 locations by the spectral-analysis-of-surface-waves (SASW) method. The testing was a part of an investigation programme for the reconstruction project of the highway. Testing locations were about 100 m apart along the highway. SASW testing was conducted to determine the layering and the profiles of shear wave velocities of the pavement structure and its subgrade from the surface to the depth of 3 m. Experience gained during the testing is briefly described. The layering determined by the SASW method compares well to that determined in investigation pits by visual inspection.

1 INTRODUCTION

Detailed knowledge of the condition of the pavement structure and the subgrade of two sections of a major highway in Croatia was required for the planned reconstruction of the highway. Sections were 32.5 km and 13.7 km long. It was decided to use the method of Spectral-Analysis-of-Surface-Waves (SASW) to determine the subgrade layering, as well as the stiffnesses for each layer down to the depth of 3 m. SASW testing was performed approximately every 100 m along highway sections, totalling in 462 testing locations. Other investigations, performed at a much smaller number of locations, included exploration pits, borings and simple laboratory tests on recovered samples.

The major concern about the use of the SASW method in the described road project was related to the test reliability and the interpretation of results in view of the short time available for the testing programme. This paper describes the gained experience and offers some guidance in similar projects.

2 FIELD MEASUREMENTS

The SASW method provides detailed profiles of shear wave velocities in horizontally layered strata working entirely from its surface (e.g. Nazarian and Stokoe 1983, Addo & Robertson 1992, Stokoe & al. 1994, Hiltunen & Gucuksi 1994). A vertical impact at the surface generates transient Rayleigh (R) waves which propagate at different speeds in layered strata. Vibration transducers are located at known distances on the surface of the stratum. The variation of surface wave velocities with frequency for the specific site is computed by performing the spectral analysis of recorded signals at known distances. This variation is known as the dispersion curve. The layer thickness and shear wave velocity are computed by use of an analytical inversion algorithm with assumed layer density and Poisson ratio. In this algorithm a horizontally layered elastic stratum having a dispersion curve which matches the measured dispersion curve is sought by trial and error. The SASW method is advantageous over other geophysical methods working entirely from the surface, since it is the only one which accurately detects softer layers underlain by stiffer ones, a feature common to all pavement structures.

The necessary equipment for the SASW method consists of the following: surface impact generators, vibration sensors, an analogue-to-digital (AD) converter and a computer supplied with the adequate software. Each of these devices has to satisfy certain requirements to obtain meaningful and reliable measurement results.

The location of the "centre" of the R-wave displacement profile is related to the wavelength \( \lambda_d \) (at about 1/3 of \( \lambda_d \)). Therefore, to obtain reliable
measurements from the surface down to the required depth of 3 m, the surface impact should generate a signal rich in adequate wavelengths. No single impact generator was found to match these requirements. Instead, three classes of impact generators were used, each for a specific range of wave frequencies: heavy falling weights, falling hammers and light falling sticks. Heavy falling weights with masses up to 1 t and falling depths up to 3 m were used for the frequency range from about 10 to 100 Hz (Fig. 1).

Falling hammers with masses up to 10 kg were used for the frequency range from about 100 Hz to 1 kHz (Fig. 2). Light falling sticks about 1 m long were used for the frequency range from about 1 to 20 kHz (Fig. 3). Every impact generator was provided with an impact tip of different design and material. It was found that specific impact generators within a class generate signals of better quality than others depending on the condition of the asphalt surface. The quality of generated signals was tested by analysing their cross power spectra and coherences. Therefore, it was occasionally necessary to choose the most suitable impact generator in the field by trial and error.

Two pairs of vibration sensors were used: two geophones (L4-C, Marc Products) for frequencies up to 1 kHz, and two accelerometers (352A78, PCB) for frequencies up to 20 kHz. The precision of the accelerometer measurements is of particular importance at higher frequencies. It is thus, favourable that the SASW method requires only the computation of the phase difference between recorded signals, and not accurate measurements of wave amplitudes. It is thus, favourable that the SASW method requires only the computation of the phase difference between recorded signals, and not accurate measurements of wave amplitudes. Due to the variable state of the asphalt surface, the adherence of the vibration sensors to the surface was a concern. Good results were obtained when the vibration sensors were pressed into a 6 mm thick wax disc contained in a plastic ring and laid on the road surface. The wax disc was of a diameter slightly larger than that of the vibration sensor.

During the testing, the pairs of accelerometers were placed on the road surface 0.125 m, 0.25 m and 0.5 m apart, while the pairs of geophones were placed 0.5 m, 1.0 m, 2.0 m and 4.0 m apart. The placement of vibration sensors was therefore, the most time-consuming phase of the testing procedure. After some time the field crew of three who performed the testing achieved an average testing time of 8 min per location.
An external DAQ PAD-1200 AD converter from National Instruments was used to convert analogue signals from vibration sensors to digital signals that were stored on a laptop computer (486 Intel processor at 50 MHz). An in-house computer code in the LabVIEW environment from National Instruments was developed for data acquisition, storage, and the spectral analysis producing the dispersion curve. Two modes of the code were developed. The first mode performs data acquisition, data storage, and the spectral analysis for control purposes only (coherence and cross power spectrum). The second mode additionally computes the dispersion curve from all recorded data as the characteristic curve for a specific site. It was more convenient to use only the first mode of the code at the time of testing. Engineering judgement had to be used in selecting measurements of acceptable quality for the calculation of the final dispersion curve. The final dispersion curve was computed for each location later on by an experienced person. It was thus essential to make several tests with different impact generators for each testing location in order to avoid returning to a previous testing location due to poor measurements. The calculation of profiles of shear wave velocities by the inversion algorithm was performed by the computer code INVERT (Nazarian, 1984).
For each of 462 locations a dispersion curve was obtained. Figure 4 shows a typical dispersion curve obtained by field measurements. In the same Figure 3, Light falling sticks about 1 m long.

Figure 4. Typical dispersion curve obtained from field measurements.

Figure 5. Profile of shear wave velocities corresponding to the theoretical dispersion curve shown in Figure 4.

The measured dispersion curve is compared with the theoretical dispersion curve corresponding to the profile of shear wave velocities (Figure 5) obtained by the inversion algorithm. The subgrade layering was deduced from the abrupt changes in the magnitude of shear wave velocities. Figure 6 shows the resulting profile of main subgrade layers along a road section. This profile compares well with observations from exploration pits. Figure 7 shows the profile of average shear wave velocities for main layers along the same road section.
3 CONCLUSIONS

SASW has proved as a valuable, reliable and cost effective method for the road reconstruction project. In order to obtain reliable results for a wide frequency range and a variable road surface condition under a strict time schedule, a number of various impact generators were successfully used.

4 REFERENCES


Development of borehole sonar for the evaluation of soil-cement column

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ABSTRACT: This study aims to develop a quality evaluation system of improved grounds by measuring the geometry of the soil-cement columns with elastic waves. A new device based on the theory of seismic reflection was developed and the following experiments were executed to assure the detection accuracy. 1. Model study with simulated soil-cement columns. 2. In-situ experiments with a soil-cement column made by the mechanical mixing method. 3. Estimation of the geometry of model columns with different shapes. Principles and the results of these experiments are reported in this paper. Evaluation of results by experts is necessary at the present stage of the development thereby an objective method should be settled in the future.

1 INTRODUCTION

Soil improvement methods have been widely used as a base of bearing stratum and a ground stabilizer on excavations. Since the quality of the soil improvement is depending mostly upon ground conditions, evaluation of the results with respect to designed performances is importance. A major evaluation method in Japan has been the unconfined compression test for core-sampled specimens, whereas other performances such as extent or continuity of the soil-cement columns made by the jet mixing method or grouting method cannot be evaluated without in-situ inspections.

The elastic wave method has been introduced recently to evaluate the continuity of soil-cement columns whereas a method that can inspect the geometry and column diameters has never been presented.

This study aims to develop an evaluation system capable of measuring the extent and geometry of soil-cement columns by means of elastic waves. To this end, a new device based on the theory of seismic reflection was developed and the following experiments were executed to assure the detection capability:

1. Model study with simulated soil-cement columns with different shapes.
2. In-situ experiments with a soil-cement column made by the mechanical mixing method.
3. Estimation of the geometry of model columns with different shapes.

The principle and the results of these experiments are reported in this paper.

2 METHOD OF MEASUREMENT

Our measurement method was based on the seismic reflection method, a geophysical exploration method of the ground. A set of an emitter and receivers was settled in a hole drilled in the soil-cement column and the distance from the hole to the boundary between improved and unimproved soils was estimated. The elastic wave used was S-wave taking into account of the result of the past studies (Tamura et al., 1996).

2.1 Principles of measurement

Elastic waves are reflected by the boundary between two medium where respective mechanical impedance are different with each other. Therefore the reflection waves may be generated from the boundary between improved and unimproved soils. The information acquired in this measurement is the time T for an elastic wave to reach and reflect from the boundary.

(a) Simplified method

Principle of the simplified method is shown in Figure 1 and in equations (1), (2) and (3).
Figure 1. Principle of the simplified method

\[ R_i = V \cdot \left( \frac{T_i}{2} \right) \]  
\[ Y = (R_i^2 - X_i^2)^{0.5} \]  
\[ V = 2 \cdot \frac{(X_1^2 - X_2^2) \cdot (T_1^2 - T_2^2)^{0.5}}{} \]  
where \( R_i \) = distance from a emitter to a reflection point, \( T_i \) = arrival time, \( V \) = elastic wave velocity, \( Y \) = improved length, \( X_i \) = distance between center line and a emitter.

(b) Full method
Principle of the full method is shown in Figure 2 and in equations (4), (5), (6), (7)and(8).

\[ R_i = V \cdot \left( \frac{T_i}{2} \right) \]  
\[ Y^2 = R_i^2 - X_i^2 = \left( \frac{V \cdot T_i}{2} \right)^2 - X_i^2 \]  
\[ T_i^2 = 1 / V^2 - X_i^2 + V^2 = a \cdot X_i^2 + b \]  
\[ V = \left( \frac{1}{a} \right)^{0.5} \]  
\[ Y = \left( \frac{Y^2}{b / a} \right)^{0.5} / 2 \]  
where \( R_i \) = distance from a emitter to a reflection point, \( T_i \) = arrival time, \( V \) = elastic wave velocity, \( Y \) = improved length, \( X_i \) = distance between a emitter and a receiver, \( a \) = inclination of regression line, \( b \) = intercept of regression line. 

When the reflection planes are located vertically, it is recommended to use the full method rather than the simplified method to measure elastic wave velocity \( V \) and improved length \( Y \).

2.2 Apparatus

The apparatus comprises an elastic wave sonar, an elastic wave controller and a computer for data acquisition and analysis (see Figure 3).

The frequency used was 2 kHz. The sonar heads were attached at the surface and were driven by a micro-motor within a range of 20 mm. A measurement was executed by pulling the sonar into a drilled hole giving a contact between the sonar head and the surrounding soils. The sonar is shown in Figure 4.

2.3 Data processing

Analysis of the acquired data was executed by a computer for analysis. Data processing was made as follows.

(a) Stacking
Stacking recorded waves acquired under the same condition to reduce the noise.

(b) Filtering
Applying an appropriate band-pass filter to the waves to reduce the other noise.

(c) Deconvolution
Deconvolving impulses from the waves to improve resolution.

(d) Elastic wave velocity calculation
Calculating the elastic wave velocity from the reflection data.

(e) Time-distance transformation
Transforming the time into distance with the elastic wave velocity.

(f) Estimation of the boundary distance
Estimating the distance from the emitter to the reflection point.
3 RESULTS OF THE MEASUREMENT

Results of the soil-cement columns measurement, both as a model specimen and as a full scale specimen made by the mechanical mixing method, are shown here.

3.1 Model study with simulated soil-cement columns

Two types of soil-cement column model, with dimensions of 150 x 50 x 100 cm, made by mixing 45% silica sand and cement or organic water-glass binder were subjected to test in a simulated ground of saturated silica sand. The bore hole was hexagonal shape where a circle with a diameter of 100 mm can be inscribed as shown in Figure 5. Test situations are shown in Figure 6. Analysis was made according to the full method and the results are shown in Table 1 and 2.

When the direction of wave propagation was perpendicular to the boundary between improved and unimproved soils, the improved area predicted by analysis showed good agreement with the designed one, whereas in the other direction, analytical results were always smaller than the designed values. The elastic wave propagation velocities to each direction determined by the analysis were larger in soil-cement column using cement than that using water-glass binder, which may be attributed to the difference in strength of
the respective grounds. The unconfined compression strength of small specimens, with common dimensions of 5 cm in diameter and 10 cm in length and using cement or organic water-glass binder, was 2.55 MPa and 0.10 MPa respectively.

3.2 In-situ experiments with a soil-cement column made by the mechanical mixing method

An improved body with a diameter of 1 m was made by the mechanical mixing method using a cement binder. Situations of the measurement is shown in Figure 7. Actual column measurement was made at a depth of -2.6, -6.5, -11.5 and -13.6 m below the ground level and the analysis of the elastic wave measurement was based on the simplified method. After all the measurement was completed, the column was inspected for confirmation.

An example of comparison of actual size and measurement at a point of -2 m below the ground level is shown in Figure 8, where the measured improved body was smaller in size but similar in

Table 1. Measurement of improved body using cement binder

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Table 2. Measurement of improved body using organic water-glass binder

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Figures 7 and 8: Situations of measurement and comparison of measured values with real size at GL -2 m

Figure 9: Situations of measurement

3.3 Model study with simulated improved bodies (blind test)

Improved body of soil-cement with dimensions of...
Figure 10. Comparison of the predicted and the actual size
55 cm in diameter and 90 cm in length was covered not to be guessed its shape from the outside and subjected to test of predicting its shape. Three types of specimen with different shape were prepared and measured by rotating clockwise in every 15 degree, that is 24 directions as shown in Figure 9. Analysis was made by both simplified and full method and the respective results were compared in terms of precision. The results are shown in Figure 10.

The reflection patterns measured with the simplified method were interpreted empirically, and the resulting shapes of each specimen showed similar tendency to the real shapes, whereas the precise size could not be evaluated since the elastic wave velocities were unavailable due to the high rigidity of the improved bodies.

On the other hand, those measured with the full method were interpreted mechanically, and the results were poor in precision. This may be attributed to the difference in elastic wave velocity between small specimen with a velocity of 1318 m/s and the soil-cement improved body with an average velocity of 520 to 630 m/s. The unconfined compression strength of the small specimens with dimensions of 5 cm in diameter and 10 cm in length was 11.0 GPa.

4. CONCLUSIONS

Our experimental results can be summarized as follows.

1. Extent of soil improvement may be estimated by the method using elastic wave propagation.
2. Evaluation of results by experts is necessary at the present stage of the development thereby an objective method should be settled in the future.

REFERENCES

Examination of liquefaction potential by seismic tomography after the Hyogoken-Nambu Earthquake in the reclaimed land of Kobe

K. Tanimoto
Kobe University, Japan

Y. Takahashi
Fudo Construction Company Ltd, Japan

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ABSTRACT: The 1995 Hyogoken-Nambu Earthquake gave waterfront areas in and around Kobe significant liquefaction damage. Port Island is a reclaimed island lying on very soft clay. It was constructed with mountain soil, called "MASA" which originated from highly weathered granitic rocks. Shear wave velocity measurements were used to evaluate the liquefaction potential of the ground instead of laboratory testing an undisturbed sample and using penetration tests because MASA includes a large quantity of gravel and cobble. The distribution of shear wave velocity among three boorholes was analyzed using seismic tomography and a comparative study of shear wave velocities between a slightly damaged area and a heavily damaged area was carried out. This study suggests that this method of shear wave velocity analysis clearly shows differences between slightly damaged areas and heavily damaged areas. Also, it can produce accurate estimations of liquefaction potential. This was confirmed by the test result's correspondence to actual damage sustained in the earthquake.

1 INTRODUCTION

Port Island is an island of reclaimed land located about 2km from Kobe City center. It was constructed by dropping soils on the seabed from off-shore pusher barges. Its soil is called "MASA" consisting of highly weathered granitic rocks, excavated and transported by elevated conveyor belts from the mountain side near the north-western part of Kobe City.

This reclaimed land was struck by the Hyogokenn-Nambu Earthquake and suffered significant damage such as settlement and sand boils due to liquefaction as shown in Fig. 1. These sand boils were observed in open cracks on the ground and near buildings. The boiler sand particles together with pore water were spread out far from these open cracks. The quay around the island experienced great displacement sea-ward but the area behind the quay showed only a few traces of sand boils although significant subsidence occurred (Tanimoto, 1995). On the land in the central part of the island, in order to promote consolidation of soft clay and to increase the

Fig. 1 Improved area with liquefied and/or inundated area in Port Island (Tanimoto, 1995)
bearing capacity of the ground, ground improvement such as the pre-heading method and the sand drain method had been carried out. Fig. 1 shows the location of such improved areas, notice that very few sand boils occurred in these areas. Interestingly the sand-drain method is effective, to a considerable extent, in reducing the occurrence of sand boils (Sakihara et al., 1995). This effectiveness is considered to be because of the densification that occurs due to the vibrating effect of the sand drain installation.

In such gravel-rich soils, it is very difficult to estimate liquefaction potential from conventional SPT data because the data is significantly influenced not by the relative density of soil but by the difficulty of driving through gravel and cobbles. From such a point of view, in situ measurements of shear wave velocity together with seismic tomography were performed to evaluate liquefaction potential of the reclaimed land.

2 Investigation Site and its N Value of SPT

Shear wave velocity measurement with seismic tomography analysis was performed at the test site. The reclaimed land consisted of both sand drained improved MASA soil and unimproved MASA soil as shown in Fig. 2. In the unimproved areas a lot of sand boils and settlement of up to about 30cm were observed but in the improved areas settlement of only about 10cm without sand boils was found.

The N values from SPT at the investigation site are shown in Table 1, where the N value data was collected from below the water table and any N value data that was influenced by gravel and cobbles (N > 33) were omitted.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Improvement</th>
<th>No. of data</th>
<th>Average of N value</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>Unimproved</td>
<td>23</td>
<td>13.0</td>
<td>5.2</td>
</tr>
<tr>
<td></td>
<td>Sand drain</td>
<td>16</td>
<td>17.2</td>
<td>6.0</td>
</tr>
<tr>
<td>After</td>
<td>Unimproved</td>
<td>22</td>
<td>17.9</td>
<td>5.9</td>
</tr>
<tr>
<td></td>
<td>Sand drain</td>
<td>13</td>
<td>18.0</td>
<td>4.8</td>
</tr>
</tbody>
</table>

From this table, it is seen that before the earthquake N values obtained in the improved ground are higher, by about 3, than that of the unimproved ground. This shows the effectiveness of the sand drain method of ground improvement works. After the earthquake, however, the values of both unimproved and improved ground have almost no difference although both of them increased. This is considered to result from insufficient number of data and difficulty of estimation on penetration testing of gravel-rich soil such as MASA soils.

3 IN SITU SHEAR WAVE VELOCITY MEASUREMENTS

Shear wave velocity is known to be influenced by density of soil, confined stress, stress history and aging. Therefore, it is a useful parameter for estimating liquefaction potential of composite-rich ground. Especially in the case of gravel-rich ground, the investigation of the distribution of shear wave velocity by seismic tomography using inter-boreholes measurement is considered to be a more practical site investigation method for estimating liquefaction potential.

Fig. 3 Schematic figure of shear wave measurement procedure for seismic tomography.
As shown in Fig. 3, three bore-holes were used for the shear wave velocity measurement: No. 2 bore-hole was used for generating shear waves by the penetration energy of SPT, and the other two boreholes were used for reception of shear waves. In each of the other boreholes, No. 1 and No. 3, down-hole receivers with 24 vertical geophones at one meter intervals were installed. A SPT blow per every meter was given and 48 channels of data were recorded.

Shear waves are generally characterized by a wave phase change of 180 degrees when the SPT blow is struck in the opposite direction. In order to confirm shear waves formed in the observed records, both downward and upward penetrations were carried out at every meter. Both the downward and upward penetration waveform records, versus receiver depth, are shown in Fig. 4. This figure shows distinct 180 degree phase changes, although the wave from records obtained by the upward penetration have less S/N ratio and are slightly noisier than those of the downward penetration.

The arrival time of a shear wave, produced by a SPT penetration, at each depth was determined by subtracting the travel time of the wave passing through the driving rod from the time between the first break of the shear wave and the time of the blow. The time of the blow was obtained from the time record of the shock mark positioned immediately below the drive head of the SPT equipment.

First arrival times of shear waves for each source and receiver thus obtained were used as input data in seismic tomography analysis. The procedure of this analysis was to divide the objective area into rectangle cells of 1m × 2m and to make an initial model of the seismic velocity based on data obtained by suspension-PS logging. Then, theoretical travel times for the initial model were calculated by the ray-tracing method based on the Huygens’ principle. The initial seismic velocity model was iteratively modified to obtain the final velocity model by using the Simultaneous Iterative Reconstruction Technique (SIRT) until the difference in travel time became small.

The results from the tomography is shown in Fig. 5. This figure shows that the unimproved reclaimed ground mainly has relatively uniform shear wave velocities of 200–240m/s except in the lower part which has higher velocities. The improved region has a velocity of 230m/s at the surface and increases with depth up to 300m/s in the lower part of the reclaimed ground.

4 EVALUATION OF LIQUEFACTION POTENTIAL

Robertson et al (1992) proposed that normalized shear wave velocity $V_{50}$ in terms of effective stress is related to the cyclic stress ratio to cause liquefaction for $M=7.5$ as shown in Fig. 6 and defined in the following equation:

$$V_{50} = \frac{V_d}{\sigma_0 - \sigma'}$$

where $V_d$ is measured shear-wave velocity in m/s, $\sigma_0$ is atmospheric pressure, usually 100KPa and $\sigma'$ is effective overburden stress in KPa.

Selecting representative points of both the unimproved and improved areas (marked by 1 in Fig. 5), the shear velocity $V_d$ and the normalized shear wave velocity $V_{50}$ versus depth were obtained and are shown in Fig. 7. In this figure $V_d$ of the unimproved ground decreases with depth and shows a minimum value of $V_{50} = 200$ m/s in depths lower than 10m, while that of the improved ground is approximately constant at 270m/s for all depths. Contrasting $V_d$ with the proposed correlation of Fig. 6, the cyclic stress ratio of $V_{50}$ is obtained to be 0.30.
The equivalent cyclic stress ratio \( (r_m / \sigma')_L \) of an earthquake is estimated by the following equation (Tokimatsu and Yoshimi, 1983):

\[
r_m = \left( \frac{M - 1}{100} + 1 \right) \left( \frac{\sigma}{\sigma'} \right) r_s
\]

where \( M \) is magnitude of earthquake, \( \sigma_{max} \) is maximum acceleration, \( \sigma \) and \( \sigma' \) are total and effective overburden pressure, \( \sigma_s \) is a stress reduction factor expressed by equation (1):

\[
r_s = 1 - 0.015 \cdot z
\]

where, \( z \) is depth in m.

From the strong motion records of Port Island recorded during the Hyogoken-Nambu earthquake (\( M = 7.2 \)), the maximum acceleration of the horizontal component according to the historical trend was observed to be 42\% gal. The cyclic stress ratio of the earthquake, according to equation (3) and the cyclic stress ratio to cause liquefaction of the unimproved ground, obtained from Fig.6, are plotted against depth in Fig.8.

It is suggested from Fig.8 that the cyclic stress ratio to cause liquefaction evaluated from shear wave velocity in unimproved ground below 7m in depth is quite a bit less than the cyclic stress ratio caused by the earthquake. But in the improved ground the normalized shear wave velocity obtained is located beyond the graph of Fig.6 proposed by Robertson et al. (1992). Therefore it is concluded that the unimproved reclaimed ground was susceptible to liquefaction while the improved ground was not.

Fig. 6 Cyclic stress ratio to cause liquefaction against normalized shear wave velocity (after Robertson et al., 1992)

Fig. 7 Profiles of shear wave velocity of unimproved and improved ground
6 CONCLUSION

Shear wave velocity distribution of the reclaimed fills, "MASA" soils, was measured using seismic tomography method and from these results liquefaction potential was evaluated.

It is judged from the results of this evaluation that ground improvement work by sand drain may change reclaimed soils into relatively denser soils, with higher stress ratio to cause liquefaction. It is thought that because the unimproved reclaimed ground constructed by only dropping soils from pusher barges had low stress ratio to cause liquefaction. It is supposed that most of the liquefaction may be taken place in such parts of the unimproved reclaimed fills.

Based on our experience described here, shear wave velocity measurement with tomography analysis may be promising to evaluate liquefaction potential of ground such as "MASA" soils which include a lot of gravel and cobbles.

7 ACKNOWLEDGEMENT

In this study, the place for the site investigation was approved for use by the Kobe City New Urban Projects Head Office. And a lot of valuable information about site investigation results of Port Island were presented by Kobe City and Construction Engineering Research Institute, Foundation. The authors would like to express their sincere gratitude to the Kobe City and the Foundation.

REFERENCES


Fig. 8 Comparison of predicted response due to the earthquake

6 DISCUSSION OF LIQUEFACTION POTENTIAL

The in situ shear wave measurements mentioned above were performed after the earthquake, not before. The characteristics of the reclaimed soils of both unimproved and improved areas may have changed during the earthquake. And also, it was pointed out that the strong motion records at Port Island may have been attenuated because liquefaction softened the stiffness of the ground (Shibata et al 1996). From these conditions it may be difficult to make a definite conclusion about the validity of liquefaction potential evaluations from seismic methods.

The reclaimed fills below 7m in depth from the ground surface were constructed by dropping soil from offshore pusher barges and this soil was so loose that it had low cyclic stress ratio which caused liquefaction. The magnitude of surface settlement due to the earthquake, however, was observed to be up to 30cm and corresponds to less than 2% (mainly about 1%) of total thickness of reclaimed fill. This shows that damage to the ground due to the earthquake was possibly not due to flow liquefaction of the whole reclaimed fill.

The improved ground areas suffered settlement of up to 10cm corresponding to 1% of the reclaimed fills and did not suffer significant liquefaction during the earthquake. In author's study, the improved ground area is estimated to be resistant enough against liquefaction occurrence from data of shear wave velocity.
Three-dimensional soil stratification using surface waves in microtremors

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ABSTRACT: A simple method for determining two- or three-dimensional shear wave velocity profiles is presented in which frequency-wave number (F-k) spectral analyses of microtremor array records are combined with microtremor horizontal-to-vertical (H/V) spectral ratio techniques. To demonstrate the effectiveness of the proposed method, microtremor measurements are conducted using arrays at six sites in Koshino city. The F-k spectral analysis of the array records yields dispersion characteristics of Rayleigh waves, and the inversion of these data results in shear wave velocity profiles down to the bedrock at those sites. Conventional microtremor measurements are performed at about 400 stations, resulting in microtremor H/V spectral ratios over the city. Based on the shear wave velocity profiles at the six sites together with the assumption that the microtremor H/V spectra reflect those of Rayleigh waves, the shear structures at those stations are estimated. This reveals a three-dimensional soil profile of the city, together with an unknown hidden valley that crosses the central part of the city. The results of subsequent borings prove the existence of the valley, indicating the effectiveness of the proposed method.

INTRODUCTION

The shear wave profile is the key parameter evaluating dynamic response characteristics of soil deposits. It is however difficult to estimate deep soil profiles using conventional geophysical methods, particularly when two- or three-dimensional soil profiles are to be required. In addition, most of the conventional geophysical methods require boreholes, and may not be performed conveniently and economically.

The frequency-wave number (F-k) spectral analysis of microtremors, an extended version of SASW methods, may be a cost-effective alternative, considering its potential to explore deep soils without any boreholes. Recent studies (e.g., Hecke, 1985; Okaeda and Maeta, 1986; Tokimatsu et al., 1992), in fact, have shown the following:

1. Rayleigh waves dominate in microtremors.
2. F-k spectral analyses of microtremor records measured at a site could yield dispersion characteristics of Rayleigh waves.
3. The inversion of these data results in a shear structure below the site.

Furthermore, it is found that the horizontal to vertical (H/V) spectral ratios of microtremors at a site correspond to those of Rayleigh waves and thus reflect the shear structure at the site (e.g., Tokimatsu and Miyadera, 1992; Tokimatsu, 1995). This validates the assumption in the method by Nakamura and Ueno (1986) that the H/V spectral ratios correspond to the shear wave transfer function. The above findings indicate a possibility that the F-k spectral analysis of microtremors combined with the H/V spectrum method permits evaluation of two- or three-dimensional shear wave structures in an economical manner. The object of this study is to explore such a possibility based on field investigation conducted in Koshino city.

SITE GEOLOGICAL CONDITIONS AND MICROTREMOR MEASUREMENTS

Microtremor measurements are conducted in Koshino city, using arrays of sensors and with one station. Array observations were made at six sites that are labelled as JMA, ASH, SWI, KBS, KMB, and KHB, and shown in Fig. 1.

Fig. 2 shows a schematic diagram showing geologic cross section along Line A-A’ shown in Fig. 1. JMA is located on a hill covered by a thin layer of silty volcanic ash that overlies Tertiary rock, called Urahoro Group. ASH, SWI, and KBS are situated on a Holocene layer that overlies Tertiary rock, KMB and KHB lie on a Holocene layer underlain by a
Pleistocene layer, called Kushiro Group. The depth to Uraboro and Kushiro Groups generally increases westward from about 20 to 80 m.

The equipment used consists of amplifiers, low-pass filters, 16-bit A/D converter, and a 32-bit computer; all built-in a portable case; and three-component velocity sensors with a natural period of 1 or 5 s. Circular arrays of different array radii were formed at each site. Each array had either four or five three-component sensors on the circumference at equal spacing with one in the center. The minimum array radii were 3 m at all sites, and the maximum array radii were 160, 75, 30, 120, and 200 m at JMA, ASH, SWI, KBS, KMB, and KHB, respectively. Sixteen - 24 sets of digitized microtremor data consisting of 4096 points each were sampled simultaneously. The sampling rate ranged from 50 to 500 Hz, depending on the site geological condition as well as the array radius used. The details of the test procedures have been described elsewhere (e.g., Tokimatsu et al., 1992).

Conventional microtremor measurements with only one station were conducted at about 400 sites over the city as shown in Fig. 1. The equipment used was the same as above, which permits computation of HVV spectra on site while the measurements are in progress. The distance between adjacent two stations ranged from about 10 - 300 m, depending on the variation of microtremor HVV spectra with distance. At each station, microtremor ground motions were observed with a three-component sensor for about 15 minutes and digitized at equal intervals of 0.01 s. Twenty sets of data consisting 2048 or 4096 points each were made from the recorded motions and used for the spectral analysis.

SHEAR STRUCTURES FROM F-K SPECTRAL ANALYSIS OF MICROTREMORS

The high-resolution F-K spectral analysis (Capon, 1969) is performed with microtremor vertical motions measured with arrays at each site to determine dispersion characteristics. An inverse analysis using these dispersion data can yield a shear wave velocity profile at each site. In the analysis, it is assumed that the soil deposit consists of 4 to 10 layers. The details of the analytical procedures have been described elsewhere (e.g., Tokimatsu et al., 1992; Arai et al., 1996).

Fig. 3 (a)-(d) show the shear wave velocity profiles at the six sites estimated from the inverse
It seems that the layer with V over 650 m/s at KMB and KHB corresponds to the Pleistocene deposits, and at the other sites, it corresponds to Tertiary rock. The depth at which the rock occurs generally increases westward. The shear wave velocities of the overlying Holocene deposit are about 200-300 m/s for sand and sandy gravel layers, about 380-440 m/s for clay and silty clay layers, and less than 150 m/s for silty volcanic ash on the hill. The estimated structures appear consistent with the available geologic information shown in Fig. 2.

**MICROTREMOR H/V SPECTRAL RATIO**

The microtremor H/V spectral ratio used in this paper is defined as

\[
\text{H/V} = \frac{|S_H|}{|S_V|}
\]

(1)

in which \(S_H\) is the Fourier amplitude of microtremor vertical motion, and \(S_V\) and \(S_{\text{H/V}}\) are those of the two orthogonal horizontal motions. Tokimatsu (1995) indicated that (1) if the microtremor H/V spectrum of a site does not have a distinct peak, it may be a rock site or a soil site with a low impedance ratio between the bedrock and the overlying deposit, and (2) if the microtremor H/V spectrum of a site has a distinct peak, the impedance ratio of the site is moderate to high and the H/V peak period, \(T_p\), could be equal to the natural site period, \(T_s\). Thus, if \((V)_{\text{H/V}}\) is known and if the impedance ratio of the site is moderate to high, the depth to the bedrock or the thickness of the surface layer, \(D\), may be estimated from

\[
D = T_p(V)_{\text{H/V}}/4
\]

(2)

in which \((V)_{\text{H/V}}\) is the average shear wave velocity of the surface layer.

Solid lines in Fig. 4 show the H/V spectra at several sites along Lines A-A' and B-B'. At each site, the H/V spectrum has a prominent peak or
peaks, indicating that the impedance ratio of the site is relatively high. Besides, the peak period of H/V spectrum varies from place to place. Namely, it increases westward from 0.3 to 1.2 s along Line A-A', but it increases from 0.3 to 0.7 s in the middle along Line B-B'. These trends suggest that the shear wave velocity structure could vary considerably along those lines, even though no topographical variation exists except for the hill on the east end of Line A-A'. Of particular interest in the figure are the second prominent peaks occurring at B2 and B3 that cannot be clearly identified at other sites.

To investigate whether the above-mentioned trends exist within the area, spatial variations of H/V spectra along the two lines are shown in Fig. 5. In the figure, the value of H/V is indicated by gradation as shown in the legend. The figure generally confirms the findings from Fig. 4. Namely, the H/V peak period increases westward from 0.2 to 1.3 s along Line A-A'. In contrast, it changes abruptly from about 0.25 to 0.8 s in the middle of Line B-B', with the appearance of the second prominent peak at 0.25 s. This creates discontinuities of the spatial variation of H/V spectra.

Fig. 5 Spatial variations of microtremor H/V spectra along Lines A-A' and B-B'.

SHEAR STRUCTURES ESTIMATED FROM MICROTREMOR H/V SPECTRA

Figure 6 shows the relation of the depth to the bedrock with \( V_s \) greater than 650 m/s, D(m), from available boring logs and Fig. 3, with microtremor H/V peak period, \( T_p \) (s). The figure shows that there is a fairly good correlation between D and \( T_p \) defined as

\[
D = \frac{C}{T_p}
\]

in which \( C \) ranges from 50-70, depending on \( T_p \). Thus the bedrock depth in the area may be approximately estimated from Eq. (3). A comparison of Eq. (3) with Eq. (2) indicates that \( (V_s)^2 \), ranges from 200 to 280 m/s.

Fig. 6 Correlation of H/V peak period with bedrock depth.
Figure 7 shows a map indicating the contours of the H/V peak period and the bedrock depth estimated from Eq. (3). The figure suggests that an unknown hidden valley crosses the central part of the city from the southeast to the northwest, of which depth is estimated to be about 40 m. The thick sedimentary deposit of the hidden valley contrasts well with the thin surface layer outside the valley. It is, however, questionable whether the depth estimated from Eq. (3) is applicable to a soil deposit consisting of multiple layers and/or with a hidden valley.

Thus, an inverse analysis is performed so that misfits between observed and theoretical H/V spectra are minimized. It is assumed that the microtremor H/V ratios reflect those of the fundamental Rayleigh waves, and that the soil deposit is horizontally
stratified. The shear wave velocities of all layers are predetermined from the results and discussions relating to Fig. 3, and only their thicknesses are sought.

Fig. 8 shows the two-dimensional shear wave velocity structures thus determined for Lines A-A' and B-B'. Broken lines in Fig. 4 are the HV spectra of the fundamental Rayleigh waves for the inverted shear wave profiles. Good agreements in spectral shape and peak period suggest that the inverted structures could be reasonably reliable. It seems that the appearance of the second peaks at B2 and B3 is due probably to the presence of the stiff layer with $V_s$ over 250 m/s that overlies the softer layer with $V_s$ of about 180 m/s.

The results shown in Fig. 8 are consistent with those in Fig. 7, claiming the presence of the hidden valley. Probably, the valley formed by stream erosion during ice ages, when sea levels were much lower than today's, has been buried by stream deposition in the Recent epoch. Since neither boring log nor geologic map confirming this estimate is available, boring sites are placed at two sites, labelled S1 and S2 in Fig. 8, below which the hidden valley is suggested.

The results of these borings are superimposed in Fig. 8. The rock occurs at depths of 25 m at S1 and 43 m at S2, which appears to confirm the existence of the hidden valley. The confirmation of the hidden valley indicates that the proposed method may be an economical and yet reliable means of estimating two-dimensional shear wave velocity structure.

CONCLUSIONS

A simple method for determining two- or three-dimensional shear wave velocity profiles has been presented in which frequency-wave number (P-k) spectral analyses of microtremor array records are combined with microtremor horizontal-to-vertical (H/V) spectral ratio techniques. To demonstrate the effectiveness of the proposed method, microtremor measurements are conducted in Kushiro city using an array of sensors and with one station. The shear structures within the city are estimated, based on the microtremor dispersive characteristics and H/V spectra, assuming that they reflect those of Rayleigh waves. On the basis of the results and discussions, the following conclusions may be made:

1. The proposed method has outlined a three-dimensional soil structure down to the bedrock, which is consistent with the available geologic information.
2. The proposed method has detected the presence of a hidden valley, which has been confirmed by the subsequent borings.
3. The inversion of microtremor H/V spectra for soil profiling is promising where the impedance ratio of the deposit is reasonably high.
4. The proposed method using microtremors could be an economical means of estimating two- or three-dimensional shear structures.

ACKNOWLEDGMENT

The authors are grateful to Mr. Inata, A., former graduate student of Tokyo Institute of Technology, for his contributions to this study.

REFERENCES

Liquefaction assessment of mine tailings dams

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ABSTRACT: Seismic piezocene penetration test (SPT) soundings were conducted in the tailings beach of an active, upstream-constructed gold mine tailings dam in Nevada. The SPCPT is a versatile tool since it provides continuous and economical profiling of stratigraphy, strength, and penetration pore water pressure using the conventional measurements of a piezocene, while also giving downhole determinations of in situ shear wave velocity and low-strain shear modulus at the same location. Profiles of cone tip resistance and shear wave velocity were used to evaluate the potential for seismic liquefaction. Earthquake-induced maximum cyclic shear stresses were obtained using computer program QUAD-IM. It is concluded that flow liquefaction is unlikely for either the operating basis earthquake (OBE) or the maximum credible earthquake (MCE). Some degree of cyclic softening and possibly large deformations would be expected for the MCE.

1 INTRODUCTION

Four seismic piezocene penetration tests (SPT) were conducted in the tailings beach of an active, upstream-constructed gold mine tailings dam in Nevada. The objective of the in situ testing was to evaluate the potential for seismic liquefaction and estimate shear strengths for use in slope stability analysis. The results of dynamic site response analysis and estimates of shear wave velocity and cone tip resistance are presented in this paper as part of the liquefaction evaluation procedure.

Access to the soft tailings beach was made possible by the construction of a 3 m-thick sandhill ramp extending 45 m into the impoundment. The soundings were conducted at distances of 15 and 45 m from the crest toward the impoundment. The tailings storage facility is constructed as a fully lined ring impoundment with two distribution and two non-distribution embankments. Distribution embankments are constructed in upstream mode using compacted alluvial borrow from a nearby source. The tailings are discharged onto the beach using peripheral spigotting with frequent cycling of deposition to promote beach drying and desiccation. The tailings pond is confined by the two non-distribution embankments, which are raised in centerline mode using compacted waste rock. Deposited tailings have an average dry unit weight of 14.9 kN/m³, water content of 25%, and 90 to 95% of low plasticity fines.

The facility is constructed on a 30-mil very low density polyethylene (VLDPE) liner. Seepage from the pond, as well as from tailings deposition is collected by a system of finger drains located at the tailings/liner interface, and delivered to a seepage collection ditch which extends along the entire toe berm perimeter.

2 TEST RESULTS

A total of four SPT soundings were completed at the site. Typical SPCPT results, including the profiles of cone tip resistance ($q_c$), sleeve friction ($f_s$), penetration pore water pressure behind the tip ($u_t$), and shear wave velocity ($V_s$) are shown in Figure 1.

A truck-mounted cone rig was used to profile the mine tailings continuously from the surface to a depth approximately 3 m above the liner/foundation interface. The tests were conducted according to ASTM D3441-86 Standard using an electronic type 2 seismic piezocene with the porous element located behind the tip.

Details of the seismic piezocene methodology are provided by Robertson et al. (1986). An instrumented hammer at the surface was used as a seismic source.
and the triggered shear waves were detected by an accelerometer mounted internally in the cone. Using an oscilloscope to record the wave forms at the surface, the propagation time of shear waves between the source and the receiver through the soil was recorded. Travel times and distances were then used to calculate $V_s$ based on the pseudo time interval method.

The profiles of $V_s$ versus depth were used to assess the low-strain shear modulus ($G_{\text{max}}$) which was used in permanent deformation and dynamic site response analyses. The shear modulus was calculated as follows:

$$G_{\text{max}} = \frac{\gamma V_s^2}{g}$$  \hspace{1cm} (1)

where $\gamma$ = total unit weight and $g$ = gravity acceleration (9.81 m/s$^2$). Recent soil classification and interpretation methods for PCPT data were used to analyze the results (e.g., Robertson and Campanella, 1983; Jamiołkowski et al., 1985; Robertson, 1991).

3 PORE PRESSURE CONDITIONS

During piezcone penetration, pore pressure dissipation tests were performed at various depths by recording pore pressure decay versus time, as shown in Figure 2. Recent methods for evaluating the in situ horizontal hydraulic conductivity ($k_h$) from the interpreted time required for 50 percent dissipation ($t_{50}$) were used (Leroueil and Jamiołkowski, 1991). The calculated values of $k_h$ ranged from $10^{-6}$ to $10^{-4}$ cm/s. The interpretation of $k_h$ at a depth of 5.9 m is shown in Figure 2. The calculated values are consistent with published data on mine tailings (e.g., Mittal and Morgenstern, 1975; Volpe, 1979; Vick, 1983).

Pore water pressures in inherently stratified tailings beaches are almost never hydrostatic below the phreatic surface. Often times, the water is perched on top of low permeability silty layers and pore press-
Figure 3: Estimated pore pressure profile

pressures decrease to zero below the perched zone. When an underdrain system is present, the resulting high downward seepage gradients tend to decrease the pore pressures substantially. The pressure heads often approach zero at the bottom of the tailings deposit. Consequently, a parabolic pore water pressure distribution is believed to be more representative of the conditions beneath tailings beaches. Most open stand piezometers at this site indicate zero pore pressures at the tailings/liner interface, and are therefore consistent with this assumption. Moreover, the available CPT readings suggest the same trend when pore pressure dissipation tests are considered. Similar patterns of pore water pressure distribution beneath tailings beaches have been reported in the literature (e.g., Stauffer and Obermeyer, 1988).

The pore pressure profile adopted in the analysis reflects the influence of underdrainage at the tailings/liner interface, as shown by a solid line in Figure 3. The pore pressures were assumed to be even higher than hydrostatic in the topmost 10 m to model the unlikely case of intensified active deposition over a prolonged period of time.

4 EARTHQUAKE DESIGN PARAMETERS

The earthquake hazard at the site was evaluated using both deterministic and probabilistic approaches to assess the seismic exposure and provide earthquake design parameters. The site is located in a seismically active region of north central Nevada. The earthquake catalog maintained by the National Oceanographic and Atmospheric Administration (NOAA) National Geophysical Data Center at Boulder, Colorado, was searched for all earthquakes of magnitude 3 and greater within 200 km of the site. The results of that search revealed that 579 earthquakes occurred within approximately 200 km of the site.

The earthquake catalog was converted into a more useful form by determining the cumulative number of events equal to or greater than magnitude 3.9 that happen on an annualized basis. By focusing on the more reliable data compiled from 1962 to 1977, it was estimated that one earthquake of magnitude 4.5 would be expected to occur somewhere within 200 km of the site every year, whereas one earthquake of magnitude 6.25 would be expected to occur every 100 years.

For the operating basin earthquake (OBE), ground motion was estimated at the site with the aid of a newly developed attenuation relationship for extensional tectonic regimes (Spudich et al., 1997). The mean peak horizontal acceleration (PHA) values at the site, assuming soil site conditions, were predicted for all earthquakes in the NOAA catalog within 30 km of the site and for the largest three events within 200 km. In addition, probabilistic acceleration values developed in the fall of 1996 for the United States by the US Geological Survey (Frankel et al., 1996) were considered in the analysis. Maps for three exceedance probabilities (10, 5, and 2 percent) in a period of 50 years were developed for soil site conditions (NEHRP site class B/C).

Based on the analysis of earthquake sources and the relatively short design life of the facility, a PHA of 0.15 g was selected as the design horizontal acceleration. This level of ground motion would be generated at a soil site by a magnitude 6.5 earthquake at a distance of 17 km, which was recommended as the OBE. If the probabilistic values from the NEHRP 1996 study are used to extrapolate to a PHA of 0.15 g, an average return period of 362 years or an exceedance probability of 12.9% in a 50-year exposure period would be obtained. It is believed that this level of probability is appropriate for a stability analysis based on the OBE. For the OBE criteria as defined by I COLD (1989), a dam should be designed to remain fully functional or be easily repaired.

Deformation analyses require earthquake acceleration time histories as input. For the OBE deformation analyses, the Convict Creek record of the 25 May 1980 Mammoth Lakes earthquake was selected. This earthquake had a magnitude ($M_c$) of 6.7 and PHA of 0.18 g, and occurred 6.1 km from the Convict Creek station (Steckins et al., 1992). The Mammoth Lakes earthquake occurred on the east side of
the Sierra Nevada Mountains in an area of volcanic activity. The horizontal (south) and vertical components of the Convict Creek record from Seckar et al. (1992) were used. The State of California Strong Motion Instrumentation Program maintains the Convict Creek accelerograph. It is located in a small, one-story building and has more than 200 m of alluvium. It is believed that it is appropriate to represent free-field ground motion on alluvium or soil sites.

The maximum credible earthquake (MCE) for the site was estimated to be a magnitude $M = 7.3$ event at a distance of 17.5 km. This event would generate a PGA of 0.56 g, which represents the mean plus one standard deviation obtained using the attenuation relationship of Spadich et al. (1997).

For the MCE deformation analyses, a synthetic acceleration versus time record based on a hypothetical earthquake originating in the Wasatch fault zone in Utah was selected. The PHA perpendicular to the fault is 0.365 g, which compares favorably with the PHA of 0.36 g estimated for the $M = 7.3$ event.

5 LIQUEFACTION EVALUATION

In assessing the stability of mine tailings dams, it is important to make a distinction between flow liquefaction and cyclic softening. Whereas the former is associated with catastrophic failures (e.g., Fort Peck, Alterra, Stava) and is generally uncommon, the latter occurs due to cyclic undrained loading created by earthquakes, and can lead to permanent deformations of sufficient magnitude to be of concern in engineering design.

According to Robertson (1994), flow liquefaction can be triggered by either monotonic (e.g., rise in groundwater level or rapid undrained loading) or cyclic (e.g., earthquake) loading, and applies to strain-softening soils only. Furthermore, the triggering of flow liquefaction requires in situ shear stresses greater than the steady state (residual) shear strength.

Cyclic softening, on the other hand, applies to both strain-softening and strain-hardening soils. Robertson (1994) further divided cyclic softening into cyclic liquefaction and cyclic mobility. Examples of cyclic softening have been common during major earthquakes (e.g., 1964 Nicaragua and 1995 Kobe) in the form of sand boils, lateral spreads, ground surface cracks, etc. A detailed account of liquefaction features is given by Ishihara (1993).

If a tailings dam is composed entirely of strain-softening soil and the in situ gravitational shear stresses are greater than the steady state strength, a flow liquefaction can occur if the soil is triggered to strain soften. However, if a tailing dam is composed of both strain-softening and strain-hardening soils, and the strain-softening soil is triggered to strain soften, a collapse and slide will occur only if, after stress redistribution due to strain softening, the strain-hardening soil cannot support the gravitational shear stress (Robertson and Fear, 1996). A flow slide will occur only if a kinematically admissible mechanism can develop.

According to Robertson et al. (1992), both the cone tip resistance and shear wave velocity can be used to assess the liquefaction potential. Existing correlations based on the normalized cone tip resistance are suitable for clean sands, and their use for sands with appreciable fines content is uncertain. However, the fines content appears to have little or no effect on the correlations based on the normalized shear wave velocity which can be calculated as follows:

$$V_s = V_c \left( \frac{P_k}{\sigma_{o,e}} \right)^{0.25}$$

where $V_c$ = normalized shear wave velocity in m/s, $V_s$ = measured shear wave velocity in m/s, $P_k$ = atmospheric pressure (100 kPa), and $\sigma_{o,e}$ = effective vertical overburden stress in the same units as $P_k$. According to Robertson et al. (1992), the demarcation line between contractive and dilative behavior and consequently the boundary for possible flow liquefaction based on $V_s$ ranges between 140–160 m/s. Woeller et al. (1996) recommend that $V_s$ = 160 m/s be used to evaluate the potential for flow liquefaction of mine tailings sand. This boundary, plotted in Figure 4, indicates that a 1.6 m-thick, contractive and potentially liquefiable soil layer exists at a depth of 1.5 m; however, most of the tailings deposit appears to be dilative.

Evaluation of the cyclic softening potential was conducted by comparing the earthquake-induced cyclic shear stresses ($\tau_{cyc}$) obtained from the site response analysis with CPT-based estimates of cyclic shear strength. Numerical estimates of $\tau_{cyc}$ were obtained using QUADAM (Hudson et al., 1994) for two different acceleration versus time histories:

- OBE represented by the 25 May 1980 Mammoth Lakes earthquake input motion with the PHA of 0.18 g.
- MCE represented by the Wasatch fault synthetic input motion with the PHA of 0.36 g.

The site response analysis provided peak cyclic shear stresses which occur at specified depths only once throughout the time history. The average cyclic shear
Figure 4: Evaluation of flow liquefaction potential based on shear wave velocity

Figure 5: Evaluation of cyclic softening potential based on cone tip resistance

stresses which are used in liquefaction analysis were calculated as 65% of the corresponding peak values (Seed and Idriss, 1971).

An estimate of the shear strengths available to resist the driving shear stresses imposed by a design earthquake was obtained according to Stark and Oliver (1995). The cone tip resistance was corrected for the effects of pore water pressure behind the tip and effective overburden stress to obtain the normalized cone tip resistance (qct). The relationship proposed by Stark and Olhoe (1995) for sandy silt with more than 35 percent fines was then used to estimate the cyclic shear stress ratio (qct/\(\sigma_0\)) corresponding to the normalized cone tip resistance. This shear stress ratio was then corrected for the effects of earthquake magnitude and effective normal stress. Finally, the resulting cyclic shear strength was calculated and superimposed on the profile of earthquake-generated \(\tau_{ve}\) in the analyzed section, as shown in Figure 5.

As can be seen, the strength inferred from CPT is higher than the shear stress induced by the OBE, indicating a low susceptibility to cyclic softening. All four CPT soundings were evaluated separately to confirm this outcome. For simplicity, the results presented in Figure 5 are based on the average CPT response. Regarding the MCE, cyclic softening would most likely occur because the earthquake-imposed cyclic shear stresses exceed the shear strengths inferred from the CPT results.

6 CONCLUSIONS

SPCPT was used effectively to profile the tailings beach of the upstream-construction gold mine tailings dam in Nevada. In addition to the continuous and economical profiling of stratigraphy, strength, and penetration pore water pressure using the conventional measurements of a piezcone (qct, \(f_s\), and \(n\)), the SPCPT also enabled downhole determinations of the in situ shear wave velocity (\(V_s\)) and low-strain shear modulus (\(G_{max}\)) at the same location. The shear strength properties were used for site response analysis, which provided estimates of earthquake-induced cyclic shear stresses. The measured \(V_s\) and \(q_t\) were then used to estimate liquefaction resistance of the tailings beach.

It is concluded that the likelihood of flow liquefaction being triggered by the OBE during the exposure period is small and that only very limited cyclic softening would occur. The embankment would remain functional during the OBE and any damage would be easily repairable. If the MCE were to occur during the exposure period, relatively large deformations or stage failure would be probable, but a flow liq-
uefaction failure where large amounts of tailings are released from the impoundment would be unlikely.

After the exposure period, the tailings would be expected to desaturate. The effects of the OBE would then be negligible and the effects of the MCE would be minor and easily repairable.

REFERENCES


Geophysical testing – Electrical techniques
The establishment and monitoring of expansive soil field sites

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ABSTRACT: A long term study of expansive soils in the Newcastle-Hunter Valley region of New South Wales, Australia, has involved the establishment and monitoring of twenty field sites. One detailed site, consisting of two ground covers and extensive instrumentation, was established in 1993 to provide a comprehensive picture of expansive soil behaviour. A further nineteen sites covering a range of soil conditions with varying geological and geographic characteristics have been established to provide general data on a regional basis. Instrumentation includes surface and sub-surface movement pegs, neutron probe, time domain reflectometry, gypsum blocks, thermocouples and an automatic weather station. Ground movements and moisture profiles to depths up to 3m are recorded on a monthly basis with monitoring to continue until at least 1998. This paper discusses the site selection, establishment, instrumentation and soil classification.

1 INTRODUCTION

As part of a long term project on the behaviour of expansive or reactive soils, a detailed field study is being conducted with the primary objectives of providing:

- Data on design parameters for foundations on reactive soils (e.g. active depth, suction change and seasonal heave).
- Detailed information on field behaviour to compare with models being developed in a separate strand of the same project (Fityus et al, 1996).

Twenty field sites have been established in the Newcastle region on the east coast of Australia, approximately 150km north of Sydney. The sites include one detailed site with extensive instrumentation and nineteen regional sites with limited instrumentation.

This paper presents a background on the nature of reactive soils in the Hunter Valley region, outlines the site selection criteria, the establishment, instrumentation and monitoring of the sites, and presents the results of soil classification testing.

2 BACKGROUND

The impetus for reactive soil research in the Hunter Region is derived from a number of factors, including:

- Changing residential construction practice.
- The 1989 Newcastle earthquake.

- Co-existence of reactive soil and mine subsidence ground movements.
- Absence of data to assess the recommendations of AS2870, the Australian standard for lightly loaded foundations, for the Hunter Valley region.
- Predicted regional population growth.

Past residential construction in the Newcastle area has been typified by flexible timber with strip/pad footing construction, primarily due to the presence of shallow mine workings. Brick veneer and slab-on-ground residential structures now dominate construction. This trend towards masonry construction, combined with the large population growth predicted for the Hunter Region, justifies a close examination of reactive soils in the region and an assessment of their potential liability (Smith and Allman, 1995).

In Australia, damage to residential and low-rise buildings through expansive soil movements is widespread. Design of foundations for lightly loaded structures is performed in accordance with AS 2870 (1996). This standard places particular emphasis on design for reactive soil sites, an acknowledgment that it is the potential for reactive soil movements that is the critical element in design.

The standard bases foundation design upon an assigned site class. It is important to note that the principle characteristics of a site with reference to expansive soil are the soil properties and a range of environmental characteristics, which include such factors as groundwater conditions, climate and vegetation. The site classification that is specified by AS 2870 is intended to account for both of these broad
site characteristics. Sites may be classified as slightly, moderately, highly or extremely reactive. The site class may be assigned by any of the following three methods:

- Soil profile identification and classification from established data.
- Computation of predicted surface movement. Three methods are specified in the standard to account for different regional practice in Australia. The first two methods are commonly used where laterally extensive uniform soil profiles exist such as in Sydney and Melbourne. In the Newcastle region the third method listed is generally used. This is primarily due to the fact that the surface geology of the Newcastle region is relatively complex, comprising Permian Age Coal Measures, Triassic Age sedimentary rock and Quaternary cohesive and granular alluvial deposits. The coal measure sequences contain numerous subhorizontally interbedded units of conglomerate, sandstone, shale, siltstone, coal and volcanic tuff (claystone). Individual beds exhibit considerable variation in thickness and depositional patterns have resulted in a sedimentary sequence characterized by rapid lateral and often abrupt vertical changes in rock and derived soil type (Pfitzner and Delsmy, 1995). This geological variability has precluded site classification on the basis of site performance or soil profile identification.

The widespread use of the computation method of classification in the Newcastle region makes it important that the assumptions inherent in the calculation are sound. The important variables that are required to calculate potential surface movement are active depth, surface suction change, shrink-swell potential (clay reactivity) and depth of cracking. The field study will provide data on these four variables.

Additionally, ground covers were placed at the detailed field site to simulate house foundations and allow round profiles to be measured. Models of unsaturated flow that are being developed as part of this research project will be compared with this field data.

3 SITE SELECTION

The principal objectives of the field study are to:

- Provide information on the nature and distribution of reactive soils in the region.
- Record and model reactive soil ground movement and moisture variation.
- Acquire regionally specific data to assess the recommendations of AS 2870.
- Compare in situ movements with those predicted on the basis of laboratory indices.
- Evaluate local site classification methods.

To satisfy these objectives the philosophy used in planning the field study was to obtain comprehensive data from one detailed site that was typical of regional conditions and then to set up a number of smaller sites covering a range of regional conditions. In this way the dual objectives of obtaining high quality, detailed data for validating models and assessing regional variation of soils could be satisfied. The detailed site was selected at Maryland and nineteen regional sites were selected covering an area of about 3000 km² as shown in Figure 1.

Details of the sites are provided in Table 1. The major site was established in 1993 in the suburb of Maryland which is the major current and projected residential growth area for the City of Newcastle. The minor sites were distributed around the study area. Sites were specifically targeted to:

- Be representative of and distributed throughout the regional geological sequence (Table 1).
- Cover a range of residual soil profiles weathered from conglomerate, sandstone, shale, coal and tuffaceous rock types of variable depth.
- Include alluvial soil deposits.
- Provide a broad geographical coverage (Figure 1).
- Include a range of terrain, vegetation and site development conditions.
- Cover areas of future residential development as well as areas to be subjected to mine subsidences.

Sites were selected after consultation with local councils, statutory authorities, mining companies and private landowners. The major issues addressed at the planning stage were the need to ensure site access for at least five years and protection of instrumentation in an urban setting.

![Figure 1. Regional map showing site locations.](image-url)
Table 1. Site details.

<table>
<thead>
<tr>
<th>Site Name</th>
<th>Depth in Rock (ft)</th>
<th>Geological Formation</th>
<th>Rock Type</th>
<th>Soil Type</th>
<th>Range of Soil Moisture (g/cm³)</th>
<th>Range of Clay Content (%)</th>
<th>Max. Geom. Movement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highpoint</td>
<td>2.5</td>
<td>Teenage Clay Shale</td>
<td>Slate</td>
<td>CBG Clay</td>
<td>2.3 - 7.6</td>
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<td>Tuffaceous</td>
<td>CBG Clay</td>
<td>3.0 - 6.4</td>
<td>49 - 49</td>
<td>28.1</td>
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<td>Tuffaceous</td>
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<td>2.2 - 3.5</td>
<td>32 - 40</td>
<td>42.1</td>
</tr>
<tr>
<td>South Waller</td>
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<td>Newberry Tuffite</td>
<td>Tuffaceous</td>
<td>CBG Clay</td>
<td>2.5 - 8.3</td>
<td>46 - 55</td>
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<td>CBG Gravely Clay</td>
<td>2.7 - 5.4</td>
<td>16 - 65</td>
<td>18.7</td>
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<td>5.7 - 6.0</td>
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<td>Tuffaceous</td>
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<td>2.6 - 6.4</td>
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<td>Alluvium</td>
<td>CBG Silty Clay</td>
<td>3.4 - 6.8</td>
<td>31 - 65</td>
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<td>Conglomerate</td>
<td>CBG Silty Clay</td>
<td>2.4 - 6.8</td>
<td>40 - 50</td>
<td>18.9</td>
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<td>CBG Silty Clay</td>
<td>1.3 - 6.0</td>
<td>26 - 33</td>
<td>16.3</td>
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<td>CBG Silty Clay</td>
<td>1.0 - 6.8</td>
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<td>12.3</td>
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<tr>
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<td>1.0 - 6.2</td>
<td>31 - 55</td>
<td>13.7</td>
</tr>
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</table>

4 ESTABLISHMENT

The detailed site at Maryland was established in 1993. The site is extensively instrumented and includes a bored concrete slab and a flexible ground cover, both 10m by 10m in area. A total of 182 survey points and 22 neutron probe access tubes as well as the instrumentation shown in Table 2 are installed at the site. Further details of this site and the instrumentation can be found in Allman et al. (1994).

Regional sites 1 to 17 were installed during June to September, 1994 and sites 18 and 19 in September, 1995 with each site taking approximately one day to install. Drilling was undertaken by small trailer and bobcat mounted rigs equipped with 100mm continuous flight augers. Detailed geotechnical logs of soil (unified soil classification system) and rock profiles have been prepared. Standard penetration testing, undisturbed US0 and disturbed sampling was carried out. Neutron probe access tube holes (50mm diameter) were drilled by hand auger or continuous flight auger and sampled for gravimetric moisture determination. Sites occupy an area of about 2m by 2m.

5 INSTRUMENTATION

The instrumentation that has been used in the project is listed in Table 2. There is a degree of redundancy in readings for soil moisture and suction to allow data to be calibrated properly.

Plans showing Instrumentation layouts for a typical regional site and the flexible ground cover at the detailed site are given in Figures 2 and 3, respectively. Instrumentation at the regional sites comprises:
- Two surface movement probes with galvanised steel rods grooved to a depth of 150mm.
- Sub-surface movement probes installed to depths of 0.5, 1.0, 1.5, 2.0 and 3.0m. Details of the movement probes are shown in Figure 4.
- A deep survey datum grouted into bedrock or deep non-reactive layers where rock is deeper than 4.5m. (see Figure 4).
- An aluminium neutron probe access tube installed to depths up to 3m to allow soil volumetric moisture content determination with a CFN 503 hydrometer.
- Standpipe piezometers in alluvial soil areas.
• Vandal deterrent 90mm diameter galvanised steel pipes concreted to 150mm depth and secured by a tightened threaded cap. This method has proved to be very robust with no damage to instrumentation reported.

6 MONITORING

Sites are monitored on a monthly basis or when specific weather conditions warrant. This takes two people three days overall and for each site involves survey relative to the bedrock/deep datum using a Wild NA3000 electronic level capable of 0.01mm resolution. Ground movement results are reported to a precision of 0.1mm relative to levels at installation. Soil moisture content is monitored by neutron probe at specific depths. Drill cuttings were taken during installation to allow comparison with gravimetric moisture content.

Soil suction determinations on recovered samples are periodically undertaken with a laboratory transistor psychrometer capable of total suction measurements in the range of 100kPa to 10MPa (ϕF 3.0 to ϕF 5.0).

7 SOIL PROPERTIES

Soil index testing has been carried out on disturbed samples and undisturbed thin walled tube (50mm) samples. The classification tests that have been performed are shown in Table 3.

The swelling strain index test involves the measurement of axial shrinkage strain on unconfined 50 mm diameter clay cores by calliper on drying from field moisture to oven dry and measurement of confined oedometer swell from field moisture to saturation under a 25kPa loading. To compensate for the one dimensional nature of oedometer test, swell strains are factored by 0.5. Shrinkage and swell testing on natural and synthetic clay cores indicates that using a factor of 0.5, the swell strain index is independent of initial sample moisture content (Fityes, 1996).

Additional testing being carried out includes X-ray diffraction, heat of wetting, and chemical analysis. A summary of results is presented in Table 3 and Figures 4 to 8 show correlations between unweighted soil index properties.

Figure 4 shows a Casagrande plot which indicates a wide range of clay soil types, from low to high plasticity inorganic clays. The samples plotting below the A-line comprise topsoils and weathered rock. A good correlation between liquid limit and linear shrinkage is presented in Figure 5 which is consistent with results from 320 tests throughout N.S.W. (Tuduci and Nguyen, 1984).

<table>
<thead>
<tr>
<th>Table 3. Soil index properties.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Index</td>
</tr>
<tr>
<td>Swell strain index (%)</td>
</tr>
<tr>
<td>Plasticity limit (%)</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
</tr>
<tr>
<td>Plasticity index (%)</td>
</tr>
<tr>
<td>Linear shrinkage (%)</td>
</tr>
<tr>
<td>Cation exchange (Mg/100g)</td>
</tr>
<tr>
<td>Clay content (% &lt; 2µm)</td>
</tr>
</tbody>
</table>
Figure 4. Plasticity index versus liquid limit.

Figure 7. Swelling strain index versus linear shrinkage.

Figure 5. Linear shrinkage versus liquid limit.

Figure 8. Swelling strain index versus cation exchange capacity.

Figure 6. Swelling strain index versus clay content.

Figure 9. Swelling strain index versus CEC x clay content.

Figure 6 shows a general increase in soil reactivity with increasing clay content, however the broad spread of results highlights the importance of other factors such as soil structure and clay fraction mineralogy in determining the degree of reactivity. Figures 7 and 8 show poor correlation between disturbed soil indices (linear shrinkage and cation exchange capacity) and soil reactivity (swelling strain index) measured on intact clay cores. Plasticity index and liquid limit show similarly poor correlations. This concurs with the results of the Sydney Swelling Soils Study (Coffey Partners, 1985). These results highlight the uncertainties involved in characterising clay reactivity of the basis of index testing carried out on disturbed and fractioned soil samples.

The influence of both physical (clay fraction content) and mineralogical (CEC) factors in assessing soil reactivity is shown in Figure 9. Whilst the correlation is poor, it shows an improvement over correlation based solely on physical or mineralogical parameters.
8 CONCLUSIONS

A network of twenty sites was established between 1993 and 1995 in the Newcastle and Hunter region for the monitoring of reactive soil movements and moisture changes. This paper details the field instrumentation and soil testing methods involved in characterising site reactivity from sites with diverse geology. Profiles of ground movement and soil moisture change with time and analysis in terms of environmental factors will be presented in a future paper. Results will provide a benchmark to assess the different methods of characterising site reactivity.

The study to date has highlighted:

- The presence of a broad range of regional reactive soil ground movements. As shown in Table 1, maximum amplitude of ground movement at the sites has ranged from 7mm to 46mm. It is worth noting that no very dry conditions have been encountered yet.
- The influence of environmental factors such as the presence of trees and drainage. At some sites only a fraction of the characterised reactivity has been realised due to minimal change in soil moisture conditions, where at other sites, ground movements in excess of those characterised on the basis of soil properties have been recorded.
- A wide range of soil index properties and recorded ground movement which is consistent with the regional geological diversity.
- Small scale site specific variability in both soil reactivity and ground movement.
- Characterisation of soil reactivity is best achieved by direct measurement of swell or shrinkage strain on undisturbed core samples rather than correlation with indirect tests carried out on part samples where soil structure has been destroyed.
- Variability in soil reactivity is not readily discernible on a visual-tactile basis.

The field instrumentation and monitoring methods have provided a comprehensive data base including detailed ground movement and moisture content profiles from twenty sites. This data will provide the basis for ongoing research into characterisation of site reactivity as well as unsaturated moisture flow.

ACKNOWLEDGMENTS

This research has been supported by the Mine Subsidence Board of New South Wales, the Australian Research Council and Robert Carr and Associates Pty. Ltd. The assistance of Newcastle City Council, Lake Macquarie City Council, the Hunter Water Corporation and Landcom is noted.

REFERENCES


Monitoring water infiltration under an earth-fill levee with geophysical techniques

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ABSTRACT: A variety of geophysical techniques (DC resistivity, spectral analysis of surface waves, ground penetrating radar and seismic refraction/reflection) were used to investigate changes in soil properties and degree of saturation induced by periodic irrigation under an earth-fill levee and a control section near the Rio Grande. Joint interpretations of these surveys are consistent with expansion of resistivity and shear stiffness with water saturation and concentration of salts in the capillary fringe zone.

1 INTRODUCTION

We conducted a series of geophysical surveys along an earth-fill levee located about 40 m west of the Rio Grande River to investigate changes in soil properties induced by periodic irrigation. Although individual geophysical surveys can be used to monitor these changes, we felt that a combination of measurement techniques might permit independent evaluation of the integrity of an earth-fill levee before or after a flooding event.

Geophysical measurements were taken along a 30 m long section of the levee during the summer of 1996. The top of the levee is 3 m above the river level. A 1 m wide irrigation ditch runs along the west side of the levee. The ditch is filled with water one day every two weeks. Water in the ditch is about 1 m deep, with the maximum fill level about 1.5 m below the levee crest. The integrity of the irrigation ditch appears to be strongly compromised by the burrowing activities of gophers. Control measurements were taken along a 30 m long line located on the river bank 20 m east of the levee and fluctuating water levels due to irrigation. Our first set of measurements were taken during dry conditions (7-12 days after last irrigation). A second set of "wet" measurements were taken about 12 hours after irrigation, allowing time for the irrigation water to infiltrate into the levee. All this time the bottom of the ditch was muddy, but did not contain standing water.

In the following sections we discuss how each geophysical survey was conducted, the sensitivity of each survey to changes expected during the irrigation cycle and the results of our measurements. Finally, we integrate results to determine how the surveys could best be used together in a final interpretation.

2 GEOPHYSICAL SURVEYS

Four geophysical surveys were conducted along the crest of the levee and a control line during "wet" and "dry" conditions. The surveys were designed to concentrate on the structure of the upper 4 to 8 m of the levee and riverbank.

The spectral analysis of surface waves (SASW) technique was used to estimate shear wave velocity, and hence soil stiffness. SASW surveys were taken 12 hours after irrigation (wet) and 10 days after irrigation (dry).

Any SASW survey requires three steps, field measurement, construction of a dispersion curve, and the inversion of the dispersion curve. Field measurements were carried out using a sledge hammer source and geophone spacings of 2, 4, 8 and 16 m. A four-channel dynamic signal analyzer was used to collect the time-domain records and perform the spectral analysis. Signals were averaged five times in the frequency domain to improve the signal to noise.
ratio. The common receiver midpoint configuration was used to enhance the quality of the data as discussed in Nazarian and Desai (1993). The final dispersion curves were determined using the software developed by Nazarian and Desai (1993). The inversion program developed by Yuan and Nazarian (1993) was used to determine the representative shear wave velocity profile at the site.

The SASW method is sensitive to pore suction pressure, compaction, cementation and to a lesser extent to the soil gradation (fine- vs coarse-grained). Soil along the riverbank was generally fine-grained and extremely unconsolidated. Measurements on top of the levee were also made in a fine grained surficial layer. The interior composition of the levee is unknown, at this time, although we hope to core both the levee and riverbank during the fall of 1997. The levee undergoes compaction by periodic vehicular traffic (about 1 vehicle/day in summer).

DC resistivity measurements were taken using a Wenner array (electrodes uniformly spaced in a line) with electrode spacings of 1, 2, 3, 4, 5, 7, 10, 15 and 20 m. For the Wenner array the apparent resistivity, \( \rho_a \), is related to the electrode spacing, \( a \), change in voltage, \( \Delta V \), and the current input to the ground, \( I \), through,

\[
\rho_a = 2 \times \frac{a \times \Delta V}{I}. \tag{1}
\]

At each electrode spacing ten repeated measurements of \( \Delta V \) were taken and averaged to obtain the apparent resistivity. Resistivity surveys were taken 12 hours after irrigation (wet) and 12 days after irrigation (dry). Observed apparent resistivity was then modeled using a 1-D layered earth structure and the kernel calculation of Keller and Frehlich (1966).

The resistivity technique is most sensitive to water salinity/water saturation when water salinity is high, and becomes more sensitive to grain size at low salinities. Although river water and irrigation water in this region have low salinities (typically 400 to 600 ppm), the riverbank itself is a site where water briefly ponds during the summer rainy season. Evaporation of this ponded and capillary fringe water leads to the formation of a salty soil crust along the riverbank. Clay also forms a major component of the soils found in the region.

Seismic refraction surveys were conducted using a 24 channel recording unit and a sledge hammer/metal plate source. Geophones had a spacing of 1 m. Each geophone location served as a source location. Ten to fifteen hammer blows were collected and stacked at each source location to enhance signal quality. The data sampling interval was 0.5 m/s and 1 sec of data were recorded at each source location. Surveys were conducted 12 hours after irrigation (wet) and 10 days after irrigation (dry). Data collected for the "dry" levee survey were corrupted after field collection and are not shown here. In this study we focused on the simple interpretation of the direct wave and first refracted arrival, using the slope and intercept time technique to determine velocity and layer thickness. Several significant reflections were also seen in the data.

The seismic refraction technique is sensitive to changes in compressional stiffness associated with changes in the degree of saturation/surface tension. It gives an independent estimate of acoustic velocities and layer thicknesses that can be compared to results of the SASW technique. Shear wave velocities (measured with SASW) increase between 0 and 15-20% saturation and then decrease at higher saturation values. P-wave velocities (measured with the refraction technique) on the other hand, increase most strongly at higher saturation values.

Ground penetrating radar (GPR) surveys were conducted using 50 and 100 MHz antennae and source-receiver spacing of 1 m. Measurements were taken every 0.2 m and 128 traces were stacked at each location. A velocity analysis (common-endpoint, 30 m in length) was conducted along each line in order to determine time/depth conversion factors. Surveys were taken across the levee at 12 hours, 36 hours and 7 days following irrigation. A survey along the river control line was taken 7 days after irrigation.

GPR is sensitive to the dielectric constant, providing a measure of water content, with minor contributions from salinity and grain size.

3 RESULTS

Results of the geophysical surveys are shown in Figures 1 (river bank measurements) and 2 (levee measurements). Note that the measurements are relative to the elevation of the river bank for Figure 1, and to the top of the levee (approximately 2 m above the river bank) for Figure 2.
Shear wave velocities during the "wet" period decreased to 10 to 16% between 2 and 8 m depth below the levee (Figure 2). Along the river bank line, a slight decrease in velocity (4%) was observed between wet and dry conditions (Figure 1), similar to the decrease seen below 8 m at the levee site. It should be noted that although "standard" SASW interpretations suggest a velocity decrease spread out between 2 and 8 m depth, the observed data are also consistent with a velocity model that would place a larger decrease in velocity between 2 and 4 m depth, with little change in velocity from "dry" conditions below 4 m depth. The velocity increase at 8 m seen on both the river and levee surveys could be related to an increase in cementation or compactive effort.

Seismic refraction studies recorded surface waves transmitted through the upper soil layer with velocities in agreement with the SASW measurements. Surface waves recorded at the levee site showed more dispersion than those recorded at the river site. The dominant 60 to 80 Hz energy in the refraction surveys has wavelengths corresponding to the upper constant shear wave velocity interval of the river site, and to the shear wave velocity change at 2 m depth at the levee site.

Refraction measurements show a slightly higher average velocity in the surficial layer along the levee (336 m/sec versus 311 to 350 m/sec for the river line) (Figure 2). The first prominent refraction along the levee had an average velocity of 1480 m/sec and the river lines had average velocities of 1540 to 1550 m/sec (Figure 2). This refraction is most likely the water table. At the river this layer is about 1.6 m from the surface in both "wet" and "dry" conditions, whereas at the levee the layer occurs at 4.3 m during "wet" conditions.

Along the river line a prominent arrival appears to be related to reflections off the river bank (located 3 to 5 m east of the river line). A second prominent reflection at about 12 m depth could be related to a clay rich layer or an increase in compaction analogous to the shear wave velocity increase observed at about 8 m depth in the SASW studies. On the levee line, a prominent arrival is associated with reflection off the sides of the levee itself. A second reflector at 10-12 m depth may be the same reflector seen along the river line, as velocities of the reflectors appear similar.

No change in resistivity values was observed along the river bank (Figure 1). The low resistivity of the first river layer is salty soil. The second layer appears to have an unusually high resistivity (100 ohm-m) for a saturated soil. We believe the upper layer resistivity was underestimated due to the electrodes being buried too deeply (about 0.5 m) relative to the electrode spacings (1 and 2 m). This would lead to an over-estimation of the resistivity of the second layer, as well as an underestimation of the depth to the top of the second layer.

Resistivity modeling along the levee gives a resistive first layer we interpret as dry, unconsolidated material. The second layer is a partially saturated soil whose resistivity drops by 60% after irrigation due to desorption of salt during water influx. Our 1-D modeling software required a surface layer thicknesses of at least one meter (as reflected in the model shown in Figure 2), however the shape of "wet" profile suggests we would need to extend the low resistivity layer nearly to surface to obtain an optimum match to the profile. We interpret the shallowing of the second/third layer interface between "dry" and "wet" conditions to be due to the rise in the water level with irrigation.

Results of our radar surveys are shown in qualitative form in Figures 1 and 2 beside the resistivity profiles. Along the river we observe reflectors at 0.5, 0.7 and 1 m depth. The maximum depth of penetration on this line (below which we lose signal coherency) is 1.5 m. In comparison to the resistivity profile, the reflectors appear to be related to significant saturation in the capillary fringe within the unsaturated soil. The maximum depth of penetration is related to attenuation of the radar signal due to the low near-surface resistivities.

Along the levee line reflections were seen at 1.2 and 1.75 m depth at wet conditions. During "dry" conditions, even only 24 hours after the "wet" survey, the 1.75 m reflection had disappeared. The 1.2 m reflection was visible during "dry" conditions, but grew progressively fainter with time. The rapid, one-day reduction in reflection amplitude at 1.2 to 1.75 m appears to be associated with significant changes in moisture content associated with the drainage of water from the levee following irrigation.

**4 CONCLUSIONS**

All geophysical methods used in our study showed
Figure 1: Variations in geophysical parameters with depth at the river location.

Figure 2. Variations in geophysical parameters with depth at the levee site.

changes in soil properties along the levee between "wet" and "dry" conditions, while soil properties of the river control line stayed constant. There is no direct correspondence of interfaces seen with the different geophysical surveys, since each survey responded to quite different property changes associated with saturation changes. The depth of the water table was imaged with the refraction and resistivity survey. Salt remobilization was detected with the resistivity and radar. Variation in stiffness properties were measured by the SASW and refraction techniques. The changes we observed in this study also appear to be site specific (e.g. salt remobilization). With only a single set of geophysical observations we would have been unable to characterize the changes to the levee associated with irrigation at this site.
The stability and repeatability of the control line (river) observations suggest these geophysical
techniques have a high level of precision for
monitoring changes in soil saturation with irrigation.
More frequent observations are needed to capture the
transient nature of the system.

ACKNOWLEDGMENTS:

We wish to thank K. Miller and T. O'Donnell for
assistance with reflection data collection and
interpretation; D. Gibbon, H. Gurola and M. Martin
for use of GPR equipment and assistance with the
GPR surveys; and J. Tyburczy for use of resistivity
equipment. Finally, we thank the students taking a
graduate level geophysical field methods course
during the summer of 1996 for assistance during the
surveys.

REFERENCES

Keller, G.V., & F.C. Freschmnecht 1966. Electrical
methods in geophysical prospecting. Pergamon
Press.

Wave Method: Field Testing. Geotechnical
Engineering Journal, Vol 119, No. 7, pp. 1101-
11, ASCE.

Youn D., & S. Nazarian 1993. Automated Surface
Wave Method: Inversion Technique. Geotechnical
Engineering Journal, Vol 119, No. 7, pp. 1112-
1126, ASCE.
Dielectric constant and electrical conductivity of contaminated fine-grained soils

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ABSTRACT: In contaminated and non-contaminated sites it is necessary to obtain information about the process of contaminant movement in soil and groundwater by measurement of the characteristic variables. This is applicable for any physicochemical process which affects the mass transfer and diffusion. In this paper, two physicochemical parameters namely, dielectric constant and electrical conductivity of soil-water system are determined in an effort to use this procedure to indentify and characterize the fate of contamination. To demonstrate the effective use of both dielectric constant and electrical conductivity in the characterization of contaminated fine-grained soils, pore fluids were prepared at different ionic strengths, and were used as permeates for kaolinite and bentonite. Thus, both dielectric constant and electrical conductivity of the soils were measured by means of a capacitor over a wide range of frequencies and moisture content. It was observed that although each soil has its unique dielectric constant and electrical conductivity at a given moisture content, increases in ionic strength cause a decrease in the dielectric constant of soil-water system at very high frequencies (MHz), whereas the dielectric constant increases at low frequencies (kHz). Electrical conductivity of a soil-water system is independent of frequency. However, it is a function of ionic strength of the pore fluid. This method/procedure can be used in estimating the level of contamination as well as the direction of contaminant movement in the subsurface without the use of extensive laboratory testing.

INTRODUCTION

Identification and/or characterization of contaminated fine-grained soils are necessary steps in the development of effective remediation techniques. However, existing methods and/or procedures are far from satisfactory because they require extensive soil-pore fluid sampling in the laboratory and field. Pore-fluid sampling has inherent difficulties. For example, contamination may occur during the sampling and testing. For a method to be satisfactory, it should be non-destructive, reproducible, cost-effective and very precise. Furthermore, the arrival times of contaminants within specific locations in soil-based barriers need to be assessed through monitoring for risk assessment. Thus, there is the need to develop a satisfactory method to identify and characterize contaminated fine-grained soils. It is the objective of this paper to determine the effectiveness of dielectric constant and electrical conductivity to identify and characterize presence of contaminants in soil-water systems. In this study, dielectric constant and electrical conductivity of kaolinite and bentonite were determined as functions of ionic strength, moisture content, and frequency in order to analyze water-soil mineral interaction, as well as polarization of the diffuse double layer.

Dielectric Constant of Materials

Dielectric constant of a system or a material is a measure of its polarizability upon application of an external electrical field: the higher the polarization, the higher the dielectric constant. Dielectric constant consists of real and imaginary parts. The real part reflects the polarizability of the material, whereas the imaginary part reflects ohmic and polarization losses. Thus:

\[ \varepsilon = \varepsilon' + i\varepsilon'' \]  

where \( \varepsilon' \) is the real part of dielectric constant, \( \varepsilon'' \) is the imaginary part, and \( i = (-1)^{1/2} \).
Many empirical and semi-theoretical mixing formulations (Fricke, 1952; Okanski, 1959; Sachs and Spiegler, 1964; and Thevensyagam, 1995) are available to estimate dielectric behavior of soil-fluid systems. All of the formulations are mainly modifications of the original Maxwell’s equation which is stated as:

\[ \varepsilon = \frac{\varepsilon_1 \varepsilon_r^2 + \varepsilon_2}{\varepsilon_1 + \frac{\varepsilon_2}{\varepsilon_1}} + \frac{\varepsilon_2}{\varepsilon_1} \]

(2)

where \(v_1\) and \(v_2\) are volumes of liquid and solid, respectively; and \(\varepsilon_1\) and \(\varepsilon_2\) are the dielectric constants of solid and liquid.

Unfortunately, these formulations fail to predict both electrical conductivity and the dielectric constant of soils because of the complex interaction between soil minerals, water molecules and ions.

Electrical Properties of Soil-Pore Fluid Systems

The electrical conductivity of a material is the inverse of the electrical resistance (S m\(^{-1}\)), as shown in Equation 3.

\[ \sigma = \frac{L}{RA} \]

(3)

where \(R\) is resistance (Ohm), \(L\) is length (m), and \(A\) is cross-sectional area (m\(^2\)).

Generally, the variables that affect the electrical properties of soils are texture, structure, soluble salts, moisture content, temperature, density, and frequency at which measurements are conducted. The soluble salt content is the most important since ions increase the electrical conductivity of the system. Although there are several semi-theoretical and empirical formulations to determine the electrical conductivity of soil-pore fluid systems, none of them is satisfactory (Kaya, 1990, and Kalinski, 1992). It should be mentioned that all the presented formulations are derived from:

\[ \sigma^* = \sum v_i \sigma_i \]

(4)

where \(\sigma^*\) is conductivity of the surface and \(v_i\) is the volume fraction of substance and \(n\) is a constant (1 or -1).

The main limitation of formulations to predict the electrical conductivity of soil-pore fluid systems is the surface conductivity due to polarization of diffuse double layer under externally applied electrical field. Polarization of diffuse double layer varies with the type of soil mineral and type of ions present in the system. Although this issue deserves lengthy discussion, it will not be discussed further due to space limitations.

MATERIALS AND METHODS

The soils used in this study and their physicochemical properties are presented in Table 1. Although the detail procedure for measuring the dielectric constant and electrical conductivity of soils is given by Kaya and Fang (1997), it can be summarized as following: The soils were saturated with pure fluids at different ionic concentrations, then the samples were placed in a U-shaped plexiglass cell. The cell (2 cm x 4 cm) was covered by silver plates to eliminate impurities. The separation distance between conducting areas was 2 cm. All the measurements were conducted within a minute or so to eliminate the impurities resulting from overheating by the applied current.

RESULTS

Figures 1 and 2 present the dielectric constant and electrical conductivity of kaolinite and bentonite at various ionic concentrations as a function of porosity at 13 MHz. In the figures, values of the dielectric constant of soils are plotted with solid lines, whereas electrical conductivities are plotted with dashed lines. It can be seen that as ionic concentrations in the pore fluid increases, the dielectric constant of soils decreases at a given porosity. On the other hand, electrical conductivity

<table>
<thead>
<tr>
<th>Table 1. Physicochemical properties of soils</th>
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<tr>
<td>Kaolinite</td>
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<tr>
<td>Specific Gravity</td>
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<tr>
<td>Liquid Limit</td>
</tr>
<tr>
<td>Plastic Index</td>
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<tr>
<td>Main Cations: *</td>
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<tr>
<td>CEC (meq/100g)</td>
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<tr>
<td>pH</td>
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<tr>
<td>Surface Area * (m(^2))</td>
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* a: Na method; b: HCl method
of the soils increases as the ionic concentration in the pore fluid increases, as expected. It should be noted that when ionic concentration is low, i.e. the pore fluid DDI H2O (Distilled-Deionized water) and 0.001 M NaCl in DDI, the electrical conductivity of the soil-water system increases with a decrease in the moisture content. This is due to contributions from the surface conductivity of the clay particles influenced by the polarization of the diffused double layer. Rhodes et al. (1976), reported a similar phenomenon.

If Figures 1 and 2 are studied it is seen that each soil has unique dielectric constant and electrical conductivity. However, it is difficult to compare the dielectric constant and electrical conductivity of kaolinite and bentonite at a given porosity because of bentonite's high affinity toward water and difficulty to obtain bentonite samples with low porosity. It should be noted that the dielectric constant of soils can be used to identify the soil type in a given soil-water system since different clay minerals will hold different amount of bounded water (Kaya and Fang, 1997). However, comparisons across various soil types may not always be accurate due to mixture of clay minerals. The same conclusions are also valid for electrical conductivity.

Figure 3 shows dielectric constant and electrical conductivity of kaolinite with NaCl and CaCl2. It is seen that for the same molar concentration, the dielectric constant of CaCl2 is lower than that of NaCl, whereas the electrical conductivity is higher. This can be explained with ionic strength, which can be determined by Equation 5:

$$I = \frac{1}{2} \sum_{i=1}^{n} z_i^2$$  \hspace{1cm} (5)

where $n$ is the total concentration of ion (M), $z$ is the valence of ion and $i = 1, 2, 3, ... n$.

When the ionic strength of 0.01 M NaCl is 0.01 M and that of 0.005 M CaCl2 is 0.015 M (it is assumed in the calculations that molal = molar). For all practical purposes, the ionic strengths are very close, thus it is reasonable to expect similar dielectric constants and electrical conductivities. However, type of the ionic concentration maybe differentiated from relaxation time as discussed in the following paragraphs. Further discussion can be found Kaya and Fang (1997).

![Figure 1. Dielectric constant (solid lines) and electrical conductivity (dashed lines) of kaolinite as a function of porosity at 13 MHz.](image1)

![Figure 2. Dielectric constant (solid lines) and electrical conductivity (dashed lines) of bentonite as a function of porosity at 13 MHz.](image2)

Figures 4 and 5 show dielectric constants and electrical conductivities of kaolinite and bentonite, at 100% moisture content (n=0.72) and 600% moisture content (n=0.94) respectively, as a function of frequency. From the figures, electrical conductivity of soils remains almost constant over a wide frequency range, although there is a considerable increase in electrical conductivity of soil-pore fluid with increase in ionic concentration. As can be seen from Figures 4 and 5, as ionic concentration in the pore fluid increases the dielectric constant of soil-pore fluid system...
decreases at very high frequencies (MHz frequency range) whereas it increases at low frequency ranges (kHz range). This can be explained by hydration of water molecules and relaxation time. As ions attracted to water molecules, they cannot orient themselves in the direction of externally applied electrical field at high frequencies whereas they will do so at low frequencies. Furthermore, the ions move toward anode and cathode at very low frequencies according to the Debye theory. The Debye theory predicts the relaxation time (τ),
\[ \tau = \frac{4\pi \eta \alpha^3}{kT} \]
where \( \alpha \) is the radius of molar structure (m), \( \eta \) is the coefficient of internal friction, \( k \) is Boltzmann’s constant and \( T \) is the absolute temperature (K).

As seen from Eq. 6, the relaxation time (2/\( \tau f \rightarrow 1 \)) is inversely proportional to frequency. At high frequencies, hydrated ions will not contribute to dielectric constant of the system since they cannot orient themselves in the direction of externally applied electrical field due to lack of time. However, at low frequencies, not only ions orient themselves in the direction of externally applied electrical field but also move toward anode and cathode which makes the medium conductive, ultimately give rise to the dielectric constant of the system. Additional information on the interaction of water and ions, and their effects on dielectric constant of water is provided by Kaya and Fang (1997).

As mentioned above, there are no available formulations that accurately predict the dielectric constant of a soil-water system because the existing formulations assume that the system consists of two phases: soil and water. Furthermore, the dielectric constants of soil and water are treated as real, not complex, because these components are assumed to be insulating (i.e., \( e'' = 0 \)). An obvious limitation of these formulations is that they do not take into account electrochemical interactions among the components (i.e., interaction between soil particles and water molecules) and the effect of the surface charge increase due to increase in the ionic concentration in the system. In this study, the effect of charge accumulation on dielectric constant of soil water system is established. Therefore, to model dielectric constant of soil-water mixture the basic components should be redefined. Instead of mixing the dielectric constant of dry soil and water, mixing dielectric constant of wetted soil and water may result better predictions. Despite apart from the discussion of formulation of the dielectric constant and electrical conductivity of soils both dielectric constant and electrical conductivity can be successfully used to predict contamination in subsurfaces and trace the contamination path.
PRACTICAL APPLICATIONS OF RESULTS

The presented methodology and results can be used in the identification and characterization of contaminated fine-grained soils. The advantage of this methodology, once the system is installed, is that spatial and temporal variations in the physicochemical state of fine-grained soils can be determined readily in a non-destructive manner. Consequently, this reduces the cost of data collection and the potential for contamination during sampling and testing. For example, such a system can be easily adapted to check for contamination from a landfill, storage tanks and chemical sites. Both the dielectric constant and electrical conductivity of pore fluids up stream and down stream can be measured, and the contamination source can be identified without performing elaborate testing procedures. Another potential application of this technique is in the assessment of growth of contaminant concentrations in the aqueous phase within specific locations in relatively intact barriers.

CONCLUSIONS

The following conclusions can be drawn from this work:

(1) Dielectric constant and electrical conductivity of contaminated fine-grained soils are considerably different from those of non-contaminated soils.

(2) As ionic concentration in the pore fluid increases the dielectric constant of the soil-pore fluid decreases at MHz frequency range, whereas it increases at kHz frequency range.

(3) As ionic concentration in pore fluid increases the electrical conductivity of soil-pore fluid increases, however, it remains constant with frequency for all practical purposes.

(4) Both dielectric constant and electrical conductivity of a soil-pore fluid system can be used to determine spatial and temporal variations in physico-chemical state of pore fluid in a non-destructive and rapid manner.

REFERENCES


"Application of Resistivity Cone Penetration Testing for Qualitative Delineation of Creosote Contamination in Saturated Soils."


A trial of five geophysical techniques to identify small scale karst

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ABSTRACT: The occurrence of some sinkholes, typically 2m in diameter and 2m deep, during the expansion of a large infrastructural development in mid-western Ireland led to an intensive investigation into the causes and consequences of the karst features. It was initially thought that the site could be characterised by means of conventional techniques. It was found that the use of these techniques was slow and expensive. A geophysics trial was then carried out to test the effectiveness of these methods. The techniques used were seismic refraction, electrical resistivity, electromagnetics, ground probing radar (GPR) and microgravity. The success of the geophysical techniques was limited, mainly due to the nature of the overburden and the small scale size of the karst features. The GPR system proved to be the most useful, although its range of penetration was low.

1 INTRODUCTION

Some sinkholes occurred on the site of a large development in the mid-western part of the Republic of Ireland in the early 1990’s. The development had been in place for some time and expansion and upgrading were in progress when the sinkholes occurred. A thorough understanding of the nature of the karst and a risk assessment of its impact on the development were considered vital to the future use of the development.

Initially a comprehensive desk study together with conventional field investigation techniques involving geological logging, trial pits and boreholes was used in attempt to characterise the site. These techniques proved slow and expensive and it was decided that a geophysics trial should be carried out with a view to using a particular geophysical technique to characterise the whole site area if it were found to be effective.

The paper will describe the background to the problem, the desk study, and the field investigations. The geophysics trial will be described in detail. The paper will describe the results of the trial and compare the findings of the geophysical techniques to the actual conditions as proven by pitting and boring. Finally some conclusions will be made as to the usefulness of geophysics in karst terrain.

2 KARST IN IRELAND

Kkarst features are well known in Ireland. Karstic features such as the Burren Plateau and the caves of County Clare are well known to most of the Irish population. These features occur in the Carboniferous Limestone bedrock of the southern and western parts of the country.

Karst features have proven to be a problem for some civil engineering works, for example in the development of a large industrial complex at Aughinash, Co. Limerick (Clarke et al. 1991), at a road construction site in Co. Cork (Beese and Creed, 1995) and for some building structures in Tralee, Co. Kerry (O‘Leary 1995). However, the low density of population and of large civil engineering works in this area has meant that the significance of karst features in civil engineering projects is not well understood.

Karstic action in Ireland is understood to have been interrupted by the most recent ice age, some 12,000 years BP, but is slowly re-emerging. It is mostly confined to the epikarst region, i.e. confined to the uppermost layer of the rock, typically 5m thick. The state of karstic processes in Ireland are illustrated in Figure 1. This figure also shows the locations mentioned above (Drew 1995).
4 INITIAL INVESTIGATION

The initial investigation comprised 3 No. percussion (shell & auger) boreholes in the overburden which included the installation of standpipes, 11 No. rotary percussive boreholes in the rock and a diver survey in reservoir.

The findings of this initial investigation were limited. The overburden was proven to be sandy in nature, no voids were encountered in the bedrock and it was proven that there was hydraulic continuity between the overburden and the bedrock.

5 DETAILED INVESTIGATION

5.1 Desk study

The detailed investigation commenced with a desk study. This comprised an examination of old and current Ordnance Survey maps, geological maps, aerial photographs, various site investigation records, the construction records for the development, general references to karst in the area and a walkover study of geological outcrops and features revealed in the aerial photograph study.

The principle findings of the desk study were that sinkholes were relatively common in the area and that the particular type of limestone found in the area was prone to karstic solution.

Some twenty sinkholes had been observed in the area within a few miles of the site. These included some occurrences on the site itself recorded during construction. The sinkholes were all in the form of truncated cones some 2m to 3m in diameter and 2m deep. A typical sinkhole is shown on Figure 2.

The particular limestone bedrock formation on the site is known as Westcorkian Limestone. It is generally fresh, thickly bedded and contains a low argillaceous content. It is known to be prone to karstic solution. Its two predominant joint sets are orientated approximately north – south (J1) and east – west (J2). In one area the pattern of sinkhole occurrence followed the J1 joint set.

5.2 Trial pitting

The trial pits confirmed the overburden material was in all cases of a sandy nature. In one case a void, some 0.65m in diameter, was encountered in the overburden.

5.3 Nature of karst features

By this time it was understood that the karst was of a
small scale. It was also recognised that the mechanism of formation of the karst features was that of subsidence sinkholes. Existing bedrock or overburden voids are flushed out by groundwater flows percolating down to a limestone water table, the voids being replenished by overburden material, thereby resulting in progressive erosion of the overlying soil and eventual appearance at the ground surface as a sinkhole.

The bedrock voids were initially caused by solution of the rock along discontinuities. The overburden voids are also due to decalcification of the glacial sediments. This phenomenon is known to occur at depths of up to 10m (Warren 1990).

5.4 Rotary percussive drilling and rotary coring

However, it was not clear what were the sizes of the bedrock voids and whether the voids were interconnected in a pattern that might allow a serious leakage from the reservoir.

It was therefore decided to undertake an investigation of the underlying bedrock using rotary percussive and rotary coring techniques. In all a total of sixty one rotary percussive boreholes and two rotary cored boreholes were drilled at four locations. Sinkholes had occurred at three of these locations. The fourth location was used as a control to examine an area where no sinkholes had occurred.

The findings of the investigation for one of these areas, Area B, are shown on figures 3 and 4. The boreholes proved the bedrock to be Waulsortian Limestone. The bedrock voids were typically 150mm to 200mm but up to 500mm deep. Some "soft" zones up to 2.5m deep were also encountered in the bedrock. These may be fully or partially infilled voids. The voids and "soft" zones were proven to be less than 4m in lateral extent. Voids of up to 1.7m in diameter were encountered in the overburden. The driling confirmed that voids in various boreholes are interconnected.

5.5 Diver's Survey

The bed of the reservoir was also inspected, in more detail, by divers. Thirteen sinkholes were discovered in the bed. The sinkholes were up to 4m in diameter and 1m deep.

At the time of writing a total of thirty eight sinkholes have now been recorded in the area.

6 GEOPHYSICAL TRIAL - DETAILS

It was proposed to develop and expand the facility and this would necessitate the investigation of a relatively large area. The use of conventional techniques for such an investigation, though likely to produce useful results, would be time consuming and expensive.

It was therefore decided to investigate the possible use of geophysical techniques to characterise the site from the point of view of possible karst problems. The objective of the survey was to attempt to locate karst features in an area where they had been proven by drilling, with a view to evaluating the techniques and possibly using the optimum one in other areas.

6.1 Previous experience in the use of geophysics

There has been much previous experience in the use of geophysics to attempt to identify voids in limestone bedrock. Geophysical techniques such as electromagnetic traversing, microgravity, GPR and cross hole tomography have been used with a varying degree of success (see McCann et al. 1987 and Eng. Group Working Party 1988 for example). It is also clear from the literature that no one geophysical technique alone can be used to resolve a problem at a particular site. The literature suggests that a void whose depth is less than twice its effective diameter can be successfully detected by geophysics. The detection of these cavities is limited by the depth and conductivity of the overburden, the presence of a
Figure 3. Plan of borehole investigation location B

Figure 4. Geological cross section - location B

water table and by local environmental noise. It was recognised that the nature of the voids and the overborden at the site may render geophysical techniques ineffective. However, it was decided to proceed with the trial because of the importance of the facility and the inefficiency of conventional techniques in site characterisation.

6.2 Techniques used in trial

The five geophysics techniques used are summarised on
Table 1. Summary of Geophysics Methods

<table>
<thead>
<tr>
<th>Method</th>
<th>Equipment</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Refraction</td>
<td>Geometrics “Nimbus” 12</td>
<td>Geophone 5m</td>
</tr>
<tr>
<td></td>
<td>channel seismograph</td>
<td></td>
</tr>
<tr>
<td>Electrical Resistivity</td>
<td>ABEM DC</td>
<td>Used “Weaver Soundings” and gradient array</td>
</tr>
<tr>
<td>Electromagnetics</td>
<td>Géoscan EM 31 meter</td>
<td>Uses VLF transmitter at Rugby, England</td>
</tr>
<tr>
<td></td>
<td></td>
<td>mounted on 3.7m boom</td>
</tr>
<tr>
<td>Ground Probing Radar (GPR), Microgravity</td>
<td>SIR-8</td>
<td>Pulse frequency 120MHz, 0.001 milligals, resolution</td>
</tr>
<tr>
<td></td>
<td>Lacroix and Ronsberg Model D103</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. The techniques and the equipment used were considered to be the state of art available at the time of the trial (late 1993). The area shown in figures 3 and 4 was chosen for the trial. Three sinkholes had previously occurred in the trial area. A thorough understanding of the ground conditions in the area was available from boreholes and trial pits and the conditions were considered typical for the site as a whole. The survey was undertaken on a 50m x 50m square grid.

6.3 Cross hole geophysics

A study was undertaken into the possible use of cross hole geophysics systems to locate the karst features. It was found that the very high frequency systems (such as the 50kHz “sonic coring” systems used in the integrity testing of plenums) would require very closely spaced boreholes. Low frequency systems, such as a 250 Hz air gun, would not be able to detect features smaller than 2m to 3m in size. It was therefore decided that a trial of these cross hole techniques was not worthwhile.

7 GEOPHYSICAL TRIAL - RESULTS

7.1 Seismic Refraction

This technique gave inaccurate predictions of rockhead trends and depth. This is because of the poor contrast between the overburden and fissured bedrock. No information was obtained on possible voids.

7.2 Electrical Resistivity

This technique also gave inaccurate predictions of rock level but gave a good indication of the position of the groundwater table. It suggested that a zone of east-west trending poor quality rock exists in the region of the sinkholes. This is the orientation of the J2 joint set.

7.3 Electromagnetics

This technique had an effective depth penetration of about 5m and failed to detect any unusual features.

7.4 Ground Probing Radar (GPR), Microgravity

As for the electromagnetic technique, the GPR signals were absorbed by the claysy overburden and the groundwater. The effective depth of penetration was again 5m. A number of patterns possibly associated with overburden voids were detected. A typical anomaly is shown on Figure 5. The investigation of these anomalies will be discussed in the next section.

7.5 Microgravity

This technique predicted accurately the trends in rockhead level. It failed to detect any voids.

8 INVESTIGATION OF GPR ANOMALIES

The five most significant anomalies revealed by the GPR survey were located and trial pits were excavated at each of the locations. The GPR survey suggested the anomalies were located between 2m and 3m depth. The trial pits revealed that in three of the five cases the anomaly was due to a boulder in the glacial deposits. In two cases, voids were detected at depths suggested by the GPR survey. The result of the trial pit excavation at one of these anomalies is shown on Figure 6.

CONCLUSIONS

The nature and depth of the overburden, the high position of the water table and the small scale of the karst features on this site meant that the geophysical techniques produced poor results. Some promise was shown however by the resistivity techniques and also by the GPR system which was capable of identifying cavities in the overburden. The maximum size of cavity was 0.8m at a depth of 1.8m (ratio 2.5:1).

It was concluded that for future developments on the site it would be cost effective to carry out a GPR survey
Image shows possible cavity at 2m depth.

Figure 5 Typical GPR anomaly in overburden

Ground level

0.9m

0.3m

Voids

1.0m

Water Table

1.2m

Overburden comprises loose to medium dense purple brown clayey silty sand with some gravel cobbles and boulders up to 400mm in diameter.

Figure 6 Trial pit at GPR anomaly

...to search for overburden voids and a resistivity survey to attempt to locate zones of poor quality rock. This would aid in the location, and possibly help reduce the number, of boreholes and trial pits. Consideration should be given to using GPR systems with a lower pulse frequency. Systems with pulse frequencies of 35 MHz to 70 MHz would have greater penetrating power (but also lower resolution).

REFERENCES

Beose, A.P. & M.J. Creed 1995. A database for...
RI-Cone penetrometers experience in naturally and artificially deposited sand

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ABSTRACT: Missey tunnel site and KIDD-II (collectively called CANLEX sites) have been used by Canadian researchers extensively to delineate the liquefaction mechanism. Authors carried out RI-Cone testing at these sites. Various in-situ testing were carried out at these sites. Large diameter sampler (Laval sampler) was used at the KIDD-II site and frozen samplings were carried at these sites to obtain high quality samples. Nuclear borehole logging was also done. Some of the tests results are presented in this paper. It has been shown that the precision achieved through RI-Cones is better than any other method used. Data for the artificially deposited sand from Japan is also presented to show environments.

1. INTRODUCTION

Natural water content (w, %) and the bulk density (y) of the soil are two fundamental soil properties that define the behavior of soil. These properties are generally measured in the laboratory on the samples obtained from the field. Soil sampling present their own problems such as disturbance during sampling, transportation, handling, storage etc. and these effects have been well documented by several researchers.

In clay deposits, good quality samples can be obtained using large diameter samples such as Laval sampler or the Shearwater sampler. However, for the small projects the price can be prohibitive. Similarly, sand sampling below water table is practically impossible. Ground freezing is an option and in recent years much progress has been made to make this method cost effective but the cost remains an inhibiting factor in their use.

Indirect geophysical methods are available to measure the properties such as water content of the soil or the bulk density but they are limited in a sense that all these methods require the preparation of boreholes. Generally, calipers are lowered down in the borehole, hence some kind of correction is required either for the type of casing used, the thickness of the casing and/or the presence of fluid in the borehole. Some of these parameters have been discussed by Shrivastava (1994).

Radio-Isotope Cone Penetrometers (RI-Cones) have been developed to measure soil parameters mentioned above, furthermore, these penetrometers can also measure cone bearing (q), sleeve friction (t) and excess pore pressure generation (u) simultaneously, providing most basic subsurface information on which many preliminary judgments can be made with confidence. The working mechanism of the these cone penetrometers have been described in details by Shrivastava (1994) and their effectiveness under various soil conditions have been reported by Shibata et al., (1993) and Mimura et al., (1995).

In this paper authors describe their experience with RI-Cones at the Missey Tunnel and KIDD-II, Fraser River Delta sites in Vancouver, Canada. Authors were able to carry out this work in cooperation with Prof. Peter Robertson, Univ. of Alberta, Canada and ConTeac Investigation Ltd., Vancouver, Canada. These sites are also known as CANLEX (for Canadian Liquefaction Experiment) sites. These two sites are naturally deposited sites. Results are also presented for reclaimed sand deposit site from Fujishmi Otsukima site in Japan to show the effectiveness of these RI-Cones in different depositional environment.

2. DESCRIPTION OF TEST SITES

Geographical locations of the testing sites are shown in Fig. 1. Both, the Missey Tunnel Site and KIDD-II Site, these sites have been chosen for extensive studies by Canadian researchers to delineate the liquefaction mechanism. Various in-situ tests as well as different sampling methods have been used at these sites. RI-Cones have also been utilized at these sites. The location of RI-Cone Tests in relation to other tests are shown in Fig. 2 and Fig. 3. Geology of these sites have been described below.
2.1 George Massey Tunnel Site, Vancouver, Canada

This site is on the south side of the main channel of the Fraser River. Top of this site is covered by sandfill which covers the deltaic deposit. The target zone at this site lies between 8 m and 13 m. This unit consists of fine sand with rare interbeds of medium and fine grained sand to silt with woody organic laminae and silty clay. High angle planar cross-bedding is common and fine interbeds commonly dip as much as 30°. Meter scale fining and coarsening upward sequences are reported by Monahan et al. (1995). 

In Fig. 4, the CPTU component of the RI-Cones have been presented. This site is characterized by very low cone bearing. The sand deposit below 17 m to 21 m consists of medium sand with some granules and pebbles, and in part it is organically meter scale coarsening in upward direction (Monahan et al., 1995). The bulk density (p) and the water content (w, %) profiles as measured by RI-Cones at the Massey Tunnel site is also shown in Fig. 4.

2.2 KIDD-II Site, Vancouver, Canada

The KIDD-II site is at the northern margin of the Fraser River Delta. Beneath a thin gravel fill, three units are recognized in the deltaic sediments above Pleistocene deposits. At this site the CANLEX target zone between 12 m and 17 m. A brief description of the target zone is given below.

The target zone lies within the unit B, as recognized by Monahan et al., (1995). From a depth of 3.6 m to 12.5 m consists of medium to coarse grained sands with granules, pebbles and silt clasts, cross bedding seems to be common. From 12.5 m to the base of the observation hole the sand is massive and fines upwards from medium to fine. The CANLEX target zone is located in the finest grained profiles. The 14C dating on a fragile twig, found at the depth of 20.2 m, yields a date of 4430 ± 50 years. This is overlain by laminated silt and very fine sand with organic laminae. It has a gradational lower contact and the thickness varies between 2 m and 3 m (Monahan et al., 1995). The RI-CPTU results are presented in Fig. 5 and the bulk density (p) and the natural water content (w, %) as measured by the RI-Cones are also shown in Fig. 5.

2.3 Higashi-Ogishima Site, Kawasaki, Japan

Higashi-Ogishima site is located about 30 km south
of Tokyo. For geographical details see Minnura and Shrivastava (1997). It is a port city. During the last two decades this site was reclaimed by pouring sand from the adjacent localities. As Japan experiences many earthquakes, soil failure due to liquefaction triggered by the cyclic loading due to earthquake is always a concern to geotechnical engineers. Also, it is important for the effective operation of the ports that the port facilities do not fail during severe earthquake. This was made abundantly clear during the Great Hanshin-Awaji Earthquake (also known as Kobe earthquake) of 1995.

Extensive investigation was carried out at this site. Various in-situ instruments such as seismic cone, pressuremeter testing etc., were carried out to delineate the in-situ soil conditions. RI-Cones were also employed at this site to obtain soil strength properties along with the in-situ natural water content \( w_0 \) and the wet density \( \rho_w \) \( \text{(m}^3\text{)} \) of the soil in-situ. Frozen sampling was also carried out at this site to compare the results.

The CPTU and other RI-Cones components have been described in details by Minnura and Shrivastava (1997), we will restrict ourselves to the description of void ratio \( e \) as obtained by RI-Cone, as the data from CANLEX project is still not in public domain only the void ratio \( e \) obtained by means of nuclear logging is available.

3. FROZEN SAMPLING

Taking a sand sample has always involved considerable debate as to the use of appropriate sampling methods. In a regular soil work triple tube sampler can be used to obtain sand sample. Bishop sampler or modified Bishop sampler can be used for very loose sand but in order for capillary forces to work on the sample the quantity of the sample is limited. Recently much effort has been put in improving the sampling by ground freezing method. Frozen samples were taken at all the sites described above.

To determine the wet density of the soil from the frozen sample, care must be taken. Water content of the frozen sample was determined for the melted state by using following simple expression:

\[
    w_m = \frac{w_f}{0.917}
\]

where \( w_m \) is the water content of the melted state and \( w_f \) is the water content of the frozen state. Thereafter, the wet density of the soil was determined using the expression given below:

\[
    \rho_w = \frac{1 + w_m}{100} \frac{1}{\left[ 1 + \frac{w_m}{\sigma_f} \right]} \rho_f
\]

Table 1: Comparison of Natural and Artificial Sand Deposit in Canada and Japan

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Massey Tunnel Site ( a )</th>
<th>KIDD-II Site</th>
<th>Higashin Ogishima</th>
</tr>
</thead>
<tbody>
<tr>
<td>App. Age (years)</td>
<td>250</td>
<td>4000</td>
<td>30</td>
</tr>
<tr>
<td>Ave. depth of GWT (m)</td>
<td>1.5</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>γ (KN/m²) above GWT</td>
<td>18.5</td>
<td>18.5</td>
<td>18.1</td>
</tr>
<tr>
<td>below GWT</td>
<td>19.5</td>
<td>19.5</td>
<td>19.1</td>
</tr>
<tr>
<td>Mineralogy</td>
<td>70% Quartz, 15% Feldspar, 5% Mica, 5% Kaolinite, 5% Chloride &amp; Saccrite</td>
<td>70% Quartz, 15% Feldspar, 5% Mica, 5% Kaolinite, 5% Chloride &amp; Saccrite</td>
<td>&gt;99% Quartz</td>
</tr>
<tr>
<td>Grain Size (Ca)</td>
<td>2.14</td>
<td>2.5</td>
<td>2.43</td>
</tr>
<tr>
<td>( D_{10} )</td>
<td>(0.30, 0.14)</td>
<td>(0.35, 0.14)</td>
<td>(0.35, 0.14)</td>
</tr>
<tr>
<td>Ave. FC (%) from SPT</td>
<td>3</td>
<td>6.8</td>
<td>0.71</td>
</tr>
<tr>
<td>( c_{opt} )</td>
<td>1.056</td>
<td>1.061</td>
<td>1.0344</td>
</tr>
<tr>
<td>( C_{opt} )</td>
<td>0.67</td>
<td>0.73</td>
<td>0.6415</td>
</tr>
<tr>
<td>( G_s )</td>
<td>1.48</td>
<td>1.72</td>
<td>2.18</td>
</tr>
<tr>
<td>( K_s )</td>
<td>0.5</td>
<td>0.5</td>
<td>0.4 - 0.5</td>
</tr>
</tbody>
</table>

\( a \) data from Wride and Robertson (1997)
in the above equation (2) \( \rho_w \) is the wet density; \( \rho_r \) is the density of the water; \( G_s \) is the specific gravity of the soil and \( Sr \) is the saturation of the soil in percent.

The water content measured in the laboratory on the frozen sample may not be the actual in-situ water content as the effect of the freezing and thawing cycle is not well defined and the authors believe that the rate of freezing and thawing as well as to take care of the 9% difference in water content (by volume) that exists between the original water content of the soil and when freezing takes place may also affect the final results. However, in the case of RI-Cones what we measure is the actual in-situ water content with the minimum of the interference of other processes.

4. DISCUSSION ON TEST RESULTS

Figure 6 shows the void ratio (e) as obtained by different methods (frozen sampling, Laval Sampler, and nuclear borehole logging) including RI-Cones. In the case of KIDD-H site, Laval sampler was not used. For the H. Ogishiha site only frozen sampling was done. Some of the basalt features of these sands are shown in Table I.

In Fig. 6a, the bold line shows the void ratio profile as obtained by RI-Cones. The thin continuous line is the void ratio obtained using nuclear logging. The frozen sample void ratio is given by square box and the open circle is the void ratio obtained from Laval sampler. For most of the recorded depth at the Mussey Tunnel site, the borehole nuclear method gives higher results, especially in the CANLEX target zone. Frozen sampling was only done in CANLEX target zone. For most of the recorded depth the frozen sample derived void ratio is lower than the RI-Cone derived void ratio, though some consistency can be seen below a depth of 11 m.

Fig. 6b shows the void ratio profiles for the KIDD-II site. For all recorded depths the nuclear method under predicts the void ratio for all depths at this site, though the void ratio obtained from frozen samples shows some consistency. No Laval sampler was at this site to obtain large diameter samples.

Fig. 6c shows the void ratio profile at the H. Ogishiha site. At this site only frozen sampling was carried out along with RI-Cones testing. The results obtained from frozen sample show good agreement with the void ratio obtained from RI-Cone data. At all these sites described above, the saturation (Sr) of soil was assumed to be 100% in calculating void ratio from RI-Cone data.

These discrepancies in various tests results are not very surprising to the authors. For all kind of borehole logging some kind of correction is required be they be the roughness of the borehole, variation in filter case thickness among many others. These correction or compensation may be the reason for the nuclear data underpredicting the void ratio at the KIDD-II site (Wride, 1997).

Similarly, the differences in the void ratio profile obtained from frozen sample can be due to the differences in the compensation of the water when thawing the sample and the preservation of soil structure due to the freezing and the thawing cycle.

From the above discussion it is clear that RI-Cones provides better overall subsurface information compared to other methods available at relatively cheaper price. Indeed no method is completely free of shortcomings so does RI-Cones where in the information of halides ions in subsurface condition is required. Nonetheless, one can deploy the RI-Cones do obtain the density profiles and that can be used as the basis for the reconstruction of the samples in the laboratory to the in-situ condition.
5. CONCLUSIONS

In this paper the effectiveness of RI-Cones have been shown for both natural (CANLEX sites, Canada) and man made deposits (H. Oghihiha site, Japan). The prohibitively expensive cost of cone sampling and the uncertainties associated with subsequent experimental process puts this method beyond the reach of regular projects. Similarly many uncertainties are associated with the borehole nuclear logging (KIDD-II site in particular). Indeed the nuclear logging does give a continuous profile like the RI-Cones, however, the corrections required for nuclear logging is not required. So authors do feel that the RI-Cones gives a slight edge over other methods and hope that the RI-Cones shall become a routine tool for geotechnical in-situ survey work.

6. ACKNOWLEDGMENTS

Authors would like to acknowledge their heartfelt gratitude to the staff of Soil & Rock Engineering Co (AKS’s previous affiliation), especially, Mr. M. Ohake, Mr. M. Nobuyama and Mr. K. Ashachi; President, Vice President and staff engineer respectively for providing the facility to carry out the tests in Canada. Heartfelt thanks are also due to the members of Geotech. Labs., PHR1, for providing the conducive atmosphere to write this paper. Finally, this work could not have been accomplished without the efforts of Prof. Peter Robertson and Mr. Gerry Cye of Univ. of Alberta; Mr. D. Woolner, Mr. I. Weenees and other members of Coastal Investigations, Vancouver, Canada. Dr. Catherine Wride provided the geophysical logs. We gratefully acknowledge their efforts to make this project a success.

7. REFERENCES


Characterization of collapsible soils with combined geophysical and penetration testing

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ABSTRACT: Loess is characterized by an open structure of fine volcanic sand and silt particles with connecting clay bridges and buttresses at contacts. At low moisture content, the formation presents high stiffness and shear strength. When the moisture content increases, the soil structure undergoes a sudden volume collapse. This experimental study of Argentinean loess includes laboratory tests (index properties, shear wave velocity, permittivity and conductivity) and field tests (CPT, SPT and down-hole seismic). Micro-level analytical models of electrical forces, suction and cementation are developed to gain insight into the observed behavior. It is shown that geophysical methods present significant advantages that are complementary to standard field measurements for the characterization of loess deposits.

1 INTRODUCTION

Loess is a windblown soil formation. The Argentinean loess is the largest deposit of its kind in the southern hemisphere. It is composed of minerals of volcanic origin and its thickness varies between 25 m to 60 m. It has low mass density and open fabric made of fine sand and silt particles connected by clay bridges (mostly montmorillonite and illite). In its undisturbed form, the Argentinean loess is a metastable-collapsible soil which undergoes sudden volume change when subjected to either wetting or loading.

The characterization of loess with low energy mechanical and electromagnetic parameters is a promising alternative on the basis of physical arguments. The methodology is equally promising from the applied engineering perspective: these parameters can be easily measured in situ with available technology. For example, transducers are readily included in penetration testing (electrical and seismic cone). In addition, non-invasive techniques can be used such as ground penetrating radar, electrical resistivity techniques, and seismic methods (reflection, refraction, surface waves, vertical seismic profiling). The interpretation of these boundary measurements can be enhanced with tomographic inversion to obtain the spatial distribution of measured parameters.

This paper presents the results of an extensive laboratory and field study of Argentinean loess, including shear wave velocity, complex permittivity and conductivity measurements on specimens at different moisture content, down-hole seismic test, standard penetration and static cone penetration testing. The effect of mean confining stresses and moisture content on collapse is addressed.

2 BACKGROUND - PRELIMINARY ANALYSES

2.1 Argentinean Loess

The most relevant properties of Argentinean loess and the differences with other similar deposits around the world are described in Terzaghi (1957), Regniatto (1971), Regnaatto and Pireu (1973), Moll and Roca (1991). A brief summary follows.

The mineralogy of Argentinean loess includes volcanic glass 50%, quartz 20-30%, plagioclase 10%, and clay minerals (illite and montmorillonite). Insoluble calcium carbonate nodules and microcrystals are found within the soil mass (often less than 8% - includes some MgCO₃); they formed during successive dry-wet cycles due to capillary ascension of bicarbonate followed by crystallization. Gypsum is frequently encountered in varying quantities.

Calcium and sodium are the most common adsorbed ions; sodium is the most abundant cation in the pore fluid (saturation fluid was extracted from a loess-water slurry and ions were determined using...
2.2 The Role of Moisture Content

Water plays a preponderant role in the formation and posterior behavior of loess. As the moisture content decreases, fine particles displace towards the menisci, the ionic concentration in the pore fluid increases, the thickness of double layers shrinks, and van der Waals attraction prevails over double layer repulsion. This evolution of interparticle electrical forces is captured in Figure 2. These results were computed with soil parameters applicable to clays in loess and clay particle geometry.

As the interparticle force-balance becomes attraction dominated, clay particles flocculate forming the clay bridges and buttresses at contacts sketched in Figure 1. If the water content reduces even further, hydrated cations in the double layer dehydrate and ionically link two contiguous clay particles. In the meantime, the concentration of salts reaches saturation and salts precipitate as crystals that strengthen the soil structure.

Suction strengthens loess as well. Suction is more effective among platy clay particles in bridges and buttresses than at the menisci between coarser spherical particles. This is clearly shown in Figure 3 (Note that the equivalent effective stress due to suction in spherical particles reaches a plateau and does not increase further).

The combined effect of these processes confers loess high cohesive strength which leads to surprising vertical cuts and the ability to withstand moderate loads.

Upon wetting, most of the processes that contribute to strengthening the soil mass are reversed:

- Soluble salt hydrates and softens
- The ionic concentration in the fluid continues decreasing with the increase in water content.
2.3 Cementation: Strength and Stiffness

The strength of cemented materials depends on the degree of cementation and the level of confinement. The higher the cementation and the lower the confinement, the stronger the role of cohesion on the peak strength. On the contrary, at sufficiently high confinement, the peak strength of particulate materials is friction dominated. The strength in the critical state is characterized by c=0 and $\phi$ independent of the degree of initial cementation.

Cementation has a significant effect on the stiffness of particulate materials, even for low degree of cementation. A modified Hertzian contact model was used to estimate the change in stiffness of the medium as a function of the cement content by weight of dry soil [Fernandez and Santamarina, 1998]. Results presented in Figure 4 show the small-strain stiffness of the soil mass $E_{\text{soil}}$ (normalized with respect to the shear modulus of the material of the particles, $G_{\text{soil}}$) vs. the effective confining pressure $\sigma'_{\text{a}}$ (also normalized with respect to $G_{\text{soil}}$). These results show that even a very low cement content can have a very significant effect on the small-strain stiffness of the soil mass.

- The lower the ionic concentration the thicker the double layers that form around particles (this is aggravated by the low local confinement clay particles experience in clay bridges and buttresses). The shear stiffness and strength of the clayey formations decrease as the thickness of the hydration layers increases. Repulsion forces may become dominant and clay particles disperse.
- Suction gradually vanishes as saturation increases. Eventually, the structure weakens and collapses even before full saturation is reached. Very low external loads are required to trigger the final collapse; in fact, self weight alone may suffice.

It is worth noting that a dry loess does not collapse when permeated with a non-polar fluid. This highlights the importance of clay and salt hydration in the metastable behavior of loess.

It follows from this discussion that strength, stiffness, and the extent of collapse will be affected by the initial void ratio and moisture content of the soil. Other relevant parameters include: fabric, the chemical composition of the saturating liquid, the amount of soluble salts, the amount of non-soluble cementing agents, the depth of burial or the level of external loads.

Figure 3. The effect of suction in granular media. Dotted lines correspond to spherical particles of radius R (equivalent compressive stress is shown). Continuous lines show the suction between parallel play particles with different specific surface. No cavitation limit is presumed.

Figure 4. A small amount of cement has a very important effect on the small-strain stiffness of particulate materials (numbers indicate the cement content relative to the weight of dry soil).
2.4 Seismic Waves

Shear wave velocity \( V_s \) is related to the small-strain shear modulus \( G_{max} \) and the mass density of the medium \( \rho \) as:

\[
V_s = \frac{G_{max}}{\sqrt{\rho}}
\]  

(1)

In an un cemented particulate medium, \( G_{max} \) depends on the state of effective stresses \( \sigma' \), as can be readily demonstrated with simple micromechanical models such as Hertzian contact in coarse particles or Coulombian forces in fine particles (see for example Cascante and Santamarina, 1996). Cementation at particle contacts reduces the sensitivity of the soil mass to the applied stresses, as shown above (Figure 4).

It follows from this discussion that the soil parameters that affect their wave velocity are: void ratio \( e \), effective confining pressure \( \sigma' \), degree of cementation, grain size distribution, soil structure, and average coordination number. Empirical relationships for un cemented soils are power relations of the form:

\[
G_{max} = A \cdot e^{a} \cdot (\sigma' + c)^n
\]

(2)

where \( A \) and \( a \) are constants and \( e \) is a function of the void ratio (Table 1 - see Ishihara, 1993)

When \( G_{max} \) predicted in Equation 2 is introduced in Equation 1, a power relation between velocity and stress is predicted:

\[
V_s = \kappa \cdot \sigma'^{b}
\]

(3)

The sensitivity of shear wave velocity and penetration testing to the state of stress in soils allows for crude correlations between \( V_s \) and the number of blows \( N \) or the tip resistance \( q_t \). The expressions are of the form:

\[
V_s = k \cdot N^\delta
\]

(4)

\[
V_s = \nu \cdot q_t^\gamma
\]

(5)

where \( k \), \( \delta \), \( \nu \), and \( \gamma \) are constants. The values of the exponents \( \delta \) and \( \gamma \) documented in the literature vary between 0.34 and 0.59 (Sylos and Koester, 1988; Oltman, 1988; Mayne and Bix, 1993). The main advantage of these correlations is to associate geophysical parameters to the extensive engineering design experience with penetration testing. However, these relations must be used with caution: the physical mechanisms involved in penetration testing (large strain process) and in wave propagation (small strain phenomenon) are very different. This is particularly relevant to cemented materials.

2.5 Electromagnetic Properties

The characterization of soils with electromagnetic techniques has significant advantages in the case of loess. There are three main parameters. The magnetic permeability \( \mu \) is assumed very low due to low ferromagnetism. (Note: there are iron salts in Argentinean loess - this confers loess the characteristic reddish-brown color). The permittivity \( \varepsilon \) of the medium is a measure of the polarizability of the medium. If it is determined in the high MHz region, the real permittivity correlates with the amount of free water in the soil. Finally, the conductivity of the soil is a measure of ion availability and ionic mobility; the amount of soluble salts, moisture content and fabric affect the electrical conductivity.

3 EXPERIMENTAL STUDY

The testing program involved a site south of the city of Cordoba. Both laboratory and field tests were conducted. The soil profile encountered at the site is characteristic of this formation:

- 0m to ~7m: low density silty clay - loess;
- 7m to ~11m: dense silty clay with cemented inclusions;
- 11m to ~13m: clean coarse sand and gravel; water table;
- 13.0 to end: stiff dense silty clay.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>A</th>
<th>( e )</th>
<th>( \alpha )</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiff clays, Low PI</td>
<td>3270</td>
<td>1.5&lt;( e &lt;0.6 )</td>
<td>Hardin &amp; Black</td>
<td></td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>445</td>
<td>( 2.5&lt;\gamma &lt;1.5 )</td>
<td>Marcuson &amp; Whals</td>
<td></td>
</tr>
<tr>
<td>Comp. Clays</td>
<td>90</td>
<td>( \gamma &gt;0 )</td>
<td>Kokusho et al.</td>
<td></td>
</tr>
</tbody>
</table>
The values of Atterberg limits within the first 11 m are fairly constant (LL = 25, PI = 20 and PL = 5). The battery of laboratory tests included index properties, shear wave velocity under isotropic confinement, conductivity and permeability measurements. Field tests involved down-hole seismic, SPT and CPT. These tests were repeated at the same site in three different locations.

3.1 Shear Wave Velocity - Isotropic Cell

Isotropic tests were performed in a triaxial cell modified with bender elements. Specimens were trimmed from an undisturbed block sample recovered at a depth of 1.5 m. Two specimens were tested. One was air-dry (w = 3%). The other specimen was saturated while confined at 30 kPa (w = 25%). Backpressure was not used, yet full saturation is not required to reduce soil suction to very low values.

Confinement was increased in stages. For each load increment, the deformation of the specimen was monitored with a vertical LVDT, and measurements of wave velocity were repeated until constant values were obtained. Figure 5 shows the results.

The velocity is much lower for the saturated specimen than for the air-dry specimen, at the same confinement, and more sensitive to changes in confinement (particularly at higher confining pressure).

3.2 Shear Wave Velocity from Down-Hole Test

Signals were produced at the surface by striking a buried concrete block with a seismic hammer. The velocity for each stratum is obtained from the average slope of the travel time vs. depth plot at each depth. This approach reduces the amplification of high frequency noise in the data onto the inverted values of velocity (see Ballard 1976; Woods, 1978). Figure 5 shows the measured wave velocities as a function of the estimated mean confining stress \( \sigma_c \) on the same plot with laboratory data.

3.3 Changes in Wave Velocity During Collapse

Small-strain \( V_s \) and \( V_p \) velocities were measured in a loess specimen during water infiltration to gain further insight into the behavior of loess during collapse. The test started with the specimen at its natural moisture content; the applied isotropic load was 50 kPa. Results are shown in Figure 6.

Collapse takes place within few minutes. The shear wave velocity decreases from \( V_s = 256 \) m/s to 152 m/s. The longitudinal wave velocity decreases from \( V_p = 405 \) m/s to \( V_p = 260 \) m/s; this indicates that the material is not saturated even after collapse (\( V_p = 1550 \) m/s for water). The sample continues deforming after collapse, yet, wave velocities remain constant.

Figure 5. Shear wave velocity. (+) Laboratory measurements on undisturbed loess subjected to isotropic confinement; air-dry and saturated. (□) Down-hole seismic test in the field.

Figure 6. Loess collapse due to water infiltration. Undisturbed loess specimen subjected to constant confining pressure \( \sigma_c = 50 \) kPa.
3.4 Penetration Resistance

SPT and CPT data are plotted in Figure 7

Figure 7. Penetration resistance vs. depth (in terms of mean confinement)

 Kunw 0.2 w% + 0.002 (w%)²

Figure 8. Variation of conductivity with moisture content (undisturbed specimens).

3.5 Conductivity

The electrical conductivity of undisturbed soil samples at different moisture content was determined with the two-electrode cell configuration. Figure 8 shows the results. The in situ electrical resistivity at different depths can be measured from the surface with the four electrode configuration or during penetration testing with electrodes mounted on the cone.

3.6 Permittivity

Undisturbed samples were trimmed into cylindrical specimens and moistened by capillarity by placing them on a porous stone in contact with deionized water for different periods of time. Moist specimens were kept in hermetic containers for homogenization. The complex permittivity was determined with a coaxial termination probe operating between 20 MHz and 1.3 GHz. After testing, the moisture content was determined. The real permittivity at 1 GHz is plotted vs. moisture content in Figure 9. The high frequency complex permittivity can be measured in situ with ground penetrating radar GPR, time domain reflectometry TDR, and during penetration testing with probes similar to the one used in this study.

4 ANALYSIS AND DISCUSSION

4.1 Wave Velocity

The least squares fitting of the velocity-stress power relation to the data gathered in the laboratory renders the following fitting coefficients:

- Lab air-dry \( v_1 = 370 \sigma_1^{0.56} \)
- Lab saturated \( v_2 = 60 \sigma_2^{0.24} \)

where \( v \) is in m/s and \( \sigma \) in kPa. On the other hand, the fitting of the power relation to down-hole measurements gives the following velocity-overburden relation:
Down-hole $V_h = 160 \alpha^{0.19}$

For comparison, the expressions in Table 1 are evaluated for a void ratio $\alpha = 0.96$.

Hordl & Black $V_h = 65 \alpha^{0.15}$

Marcuson & Wahls $V_h = 1 \alpha^{0.25}$

Kokusho et al. $V_h = 34 \alpha^{0.3}$

Clearly, loess at natural moisture content has a significantly higher stiffness than standard clayey soils, within the stress-range of these tests.

The exponent in velocity-stress relations reflects the combined effect of the geometry of contacts (e.g., spherical Hertzian $\alpha=1/6$, conical $\alpha=1/4$), changes in fabric (as density increases, the coordination number increases, and the exponent increases), and other physical processes at contacts (e.g., the yielding of spherical contacts leads to an exponent $\alpha=1/4$; deformation at contact governed by electrical interparticle forces manifests in high exponents, usually $\alpha=2.0$). The low exponent for the air-dry specimen tested in the laboratory indicates that contact deformation and fabric changes are very limited. These results are in agreement with analytical predictions presented in Figure 4 for cemented particulate materials.

The higher exponent observed in the saturated specimen is consistent with values obtained for typical uncedentated silts. Careful observation of the experimental results obtained for the saturated specimen (Figure 5 - w%<25 Lab) shows an upwards trend for the higher confinement, similar to the trends predicted in Figure 4. This suggests a change in behavior away from cementation controlled, towards a behavior controlled by contact deformation and fabric changes.

The estimation of effective interparticle forces is quite complex in soil systems with broad particle and pore size distribution. For example, clay particles in bridges and buttresses do not feel the direct impact of changes in external confinement. On the other hand, they directly experience suction and electrical forces which can be very high at low moisture (much higher than the effect of changes in external forces). Hence, changes in confinement and moisture content have different local effects within the soils mass.

Velocity trends obtained in the laboratory and in the field are superimposed in Figure 5. This graph accentuates the role of confining pressure and moisture content on the small-strain stiffness of loess. It is apparent that clay/silt bridges and buttresses creep with increased moisture, and the coarser particles effectively feel the overburden. This analysis also explains the weaker strength of loess with increased moisture content. It is important to realize that the creep of supporting bridges does not necessarily lead to the structural collapse of the soil skeleton: only virtual forces are required to prevent the buckling of grain-chains.

### 4.2 Wave Velocity and Penetration Resistance

The shear wave velocity obtained with down-hole testing was correlated with the penetration resistance corresponding to the same stratum. The fitting parameters are

- SPT $V_s = 180 N^{0.23}$
- CPT $V_s = 370 q^{0.18}$

where $q_s$ is in kN/m$^2$ and $V_s$ in m/s. These exponents are smaller in loess than in clays (0.35 to 0.7). The variability in these correlations discourages the use of small-strain parameters to estimate large-strain parameters, and vice-versa.

### 4.3 Electrical properties

The conductivity of loess is a function of the concentration and the mobility of ions. These depend on the availability of soluble salts and moisture content (soil structure and specific surface are also relevant - Rinaldi, 1994).

On the other hand, the high frequency polarizability of the medium assessed by the permittivity at 1 GHz is a measure of the free water content independent of soil characteristics and soluble salts (part of the water in the soil will be adsorbed and not free to rotate and align to the applied external field, hence it will not contribute to the measured polarization - Santamarina and Fam, 1997).

It follows from these observations that both permittivity $\kappa'$ and conductivity provide complementary information relevant to the characterization of loess.

### 5 CONCLUSIONS

Gravity forces prevail among sand and silt grains in loess. However, surface related electrical forces and suction prevail in clay particles. This distinction causes local segregation, and the formation of clay bridges and buttresses. The resulting material resembles a gap-graded, dual porosity medium, with unique and intricate behavior.

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The cementation provided by precipitated salts, clay bridges and buttresses vanishes as the moisture content increases: suction forces disappear, precipitated salt crystals solubilize, the electrolyte concentration decreases and clay particles form thick double layers. The strength and stiffness of clay bridges and buttresses decrease, and the metastable soil structure provided by the coarse particles collapses. External forces increase the magnitude of the collapse.

Small-strain wave propagation velocity and large-strain strength depend on the applied confinement and moisture content. The lower the moisture content, the less important the effect of confinement becomes. The reverse is also true.

The high frequency permittivity and the electrical conductivity of loess are robust indicators of moisture content, ionic concentration and ionic mobility.

ACKNOWLEDGMENTS

Support for this research was provided by national funding agencies CONICET (Argentina) and NSF (USA). D. Fratta made thoughtful comments and suggestions.

ELECTRONIC FILES & COMMUNICATION

Mathcad files used to compute Figures 2, 3, and 4 can be obtained from the authors. These files include assumptions, derivations, and numerical study. The authors can be electronically reached at: vtrinardi@grving.efn.uncor.edu, carlos@ce.gatech.edu, eredolfi@ccm.uncor.edu.

REFERENCES


Collecting real-time soil moisture profiles in the vadose zone

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ABSTRACT: The savings realized by using cone penetrometer (CPT) technology to supplement site characterization work are well documented. Adding soil moisture profiling to CPT testing can further enhance the interpretation of geologic and hydrogeologic conditions at a site. The performance characteristics of three soil moisture probes (SMPs) used with direct-push technology were tested at a site in South Carolina. In the unsaturated zone, all three probes predicted volumetric soil moisture within ±5 moisture units of the laboratory values more than 65% of the time. When the results were outside this range, the difference was not large enough to lead to erroneous interpretations of the actual soil moisture trends. Results of testing in a variety of hydrogeologic conditions indicate that when SMPs are used with proper site calibration and controls, each probe can predict soil moisture values within the unsaturated zone that closely match laboratory results, in real time.

1 INTRODUCTION

Soil moisture measurements are currently made on samples retrieved from a bore hole or test pit by using specific ASTM (American Society for Testing and Materials) methods in a geotechnical laboratory. The entire process can take days or even weeks. In addition, the soil moisture content measured in the laboratory sometimes underestimates the actual moisture content of the soil because of drying of the sample during handling. Additional sampling is often required after the initial results are received.

The purpose of this project was to evaluate in the field three SMPs developed for deployment with a CPT. The SMPs operate by using the soil’s dielectric properties to predict the moisture content. A time domain reflectometry probe developed by Sandia National Laboratories and two frequency domain probes developed by Applied Research Associates (ARA) and Coneval were used in the study. All three SMPs were easy to set up and install, and each provided real-time data that were useful for immediate assessment of the subsurface moisture conditions in the vadose zone. Each SMP was to provide soil moisture data within ±5 moisture units of the actual soil moisture, as defined by using conventional ASTM standardized sample collection and analysis methods (ASTM 1964; Wilson 1990).

Three separate locations were selected at the U.S. Department of Energy’s Savannah River Site in South Carolina to provide geologic and hydrogeologic variability between the evaluations of the SMPs. To provide control, continuous soil samples were collected adjacent to each test site with conventional drilling techniques and then sent to a laboratory for geotechnical analysis. The SMPs were used in conjunction with conventional CPT technology that provided tip stress and sleeve friction parameters for further characterization of the subsurface conditions. Multiple soil moisture profiles were obtained around the borehole at each control location.

2 FIELD PROGRAM

Three core holes (SBD1, SBD2, and SBD3) were drilled at two different sites (TXA Area and M Area; Figure 1) with a hollow-stem auger. Soil samples were collected at each location with a 7.6-cm (3-in.) Shelby tube. The samples were handled, stored, and analyzed according to the ASTM procedures defined in Argonne’s Test Plan (Argonne 1996a). Law Environmental’s soil engineering laboratory analyzed the samples for grain size and volumetric soil moisture content.

Data were collected with each of the three SMPs at minimum of two locations by pushing each sensor along the circumference of a circle 2.6-3.9 m (8.5-12 ft) in diameter around SBD1, SBD2, or SBD3.
Figure 1. Location of study area.

Each SMP was coupled to a 4.4-cm (1.75-in.) electronic CPT that permitted the simultaneous acquisition of soil moisture and geotechnical data (tip stress, sleeve friction, and pore pressure) for each push. The ARA and Converel SMPs recorded percent volumetric soil moisture. For best results, each SMP developer (ARA, Sandia, and Converel) recommends site-specific calibration. Argonne further recommends that all CPT sensors always be used with site-specific...
controls. However, no site-specific information was available for the test site prior to the field evaluation. Therefore, each sensor was calibrated before deployment on the basis of standard laboratory readings (bucket tests) for sand or previous tests performed with the sensor.

Each of the three sensors performed similarly in evaluating the volumetric moisture profile at each of the three core locations. Because of limited space, the SMP data for only one of the three probes are presented here. The data presented are reasonably representative of all of the probes tested.

3 DISCUSSION

To verify the sensors' capabilities to characterize geology, three continuous cores were collected from soil borings at two sites (TNX Area and M Area) to provide detailed geologic and soil moisture data. After volumetric soil moisture measurements were completed, the Shelby tube was cut open, and the core material was characterized. The sediments at the TNX Area and the M Area consist of interbedded fine sands, silts, and clays, with clayey sands and sandy clays dominating the vadose zone (Figure 2). The sediments at the TNX Area were saturated at approximately 10.8-11.2 m (33-34 ft) below ground surface (BGS). Saturated sediments were not encountered at the M Area because of the greater depth of the water table. The lateral variability in the near-surface sediments was more prominent at TNX Area, whereas the M Area showed little lateral variability.

Soil moisture probe data were collected by pushing each sensor to a depth of approximately 16.4 m (55 ft) BGS around soil borings SB01 and SB02. Because the water table at TNX began at 11.0 m (35.5 ft) BGS, the probes' effectiveness in both saturated and unsaturated conditions could be evaluated. At the M Area, SMP data were collected only in the vadose zone.

Figure 3 shows the ratios of sand or silt to clay (as determined by the laboratory), resistivity plots for the CPT push of the ARA sensor, and plots of the ARA sensor's volumetric moisture data with the ±5 moisture unit error range (based on the laboratory volumetric moisture results) for SB01, SB02, and SB03. The CPT resistivity measurements were taken to provide supplementary stratigraphic information (see Argonne 1996b for more details).

In the vadose zone, variability in soil moisture content is controlled primarily by geology. Moisture content is generally greater in soils containing a higher percentage of clay and or organic material. The volumetric moisture content in these clays and organic-rich soils can be > 33%, a value similar to that for saturated soils (Wierenga 1995). Conversely, the soil moisture content in sandier soils is generally much lower, often <10%. Below the water table, the distinction between the various types of geologic materials becomes uncertain.

4 RESULTS AND COMPARISON OF THE THREE SOIL MOISTURE PROBES

The site installation, setup, and operation of each sensor proved to be quick and simple. All sensor data were collected by using the ASTM standard push rate
Figure 3. Ratios of sand or silt to clay, resistivity data, and volumetric moisture data for soil boring locations SB01 (top), SB02 (middle), and SB03 (bottom). Resistivity data are shown for two pushes. Volumetric moisture data are also shown for two pushes, and the acceptable range (±5 moisture units from laboratory values) is indicated by the heavy lines.
of 2 cm/s. The data were displayed in numeric form on the screen during the push. Each sensor was operated with software provided by the developer; a simple program was written in the field to merge each file with the CPT tip stress and sleeve friction data files. Plots of the final logs were produced within minutes of each completed push.

The variability in volumetric soil moisture with respect to geology might be due in part to surface elevation differences and thickening or thinning of units over small spatial intervals at the site. In addition, the laboratory volumetric moisture and grain size analyses were performed on discrete sections (15.2-25.4 cm [6-10 in.]) collected from soil cores, whereas CPT sensor readings were collected continuously along the entire borehole.

To evaluate the ability of the SMP to reasonably predict volumetric moisture content within the soil profile, each of the SMP profiles was plotted in conjunction with an error line of ±5 volumetric moisture units. This margin of error was based on the laboratory soil moisture data.

Evaluation of the ARA sensor showed that its volumetric moisture predictions generally agreed with laboratory data, given the geologic differences between each sensor location and the control location, as well as the geologic variability at each location (Figure 3). In the vadose zone at the TXN Area, the SMP data fell well within the range of ±5 volumetric moisture units established from the laboratory data. In the saturated zone, discrepancies between the values predicted by the sensors and the laboratory data were more common, with approximately 20-30% of the predicted data outside the acceptable range of ±5 moisture units. Both sets of data follow the same trends, predicting higher soil moisture values in the clayey material and lower values in the sandy units in the vadose zone. The data for the percent volumetric moisture collected at SB03 in the M Area (Figure 3) fell almost entirely within the acceptable range (±5 moisture units).

Evaluation of the Sandia sensor showed that its predicted percent volumetric moisture values generally agreed with the laboratory values for the vadose zone, given the potential variability in geology at the TXN Area. However, in the saturated zone, values greater than 100% volumetric moisture were predicted, falling outside the acceptable range 75% of the time. In the upper 9.8 m (30 ft) at the M Area, the Sandia SMP predicted a volumetric moisture content slightly lower than the laboratory data 20% of the time, but the values matched almost perfectly at 9.8-16.4 m (30-50 ft) BGS.

Evaluation of the Conevel sensor showed that, within the vadose zone, this sensor predicted a sediment moisture content lower than the laboratory data 8-37% of the time. For the saturated zone at the TXN Area, moisture contents predicted were higher than the laboratory data. The Conevel SMP data fell outside the range of ±5 moisture units 45-72% of the time and were in excess of the laboratory data by as much as 30-40%. In comparison to the laboratory data at M Area, the Conevel SMP predicted a slightly lower volumetric moisture content 20-30% of the time and a higher moisture content only 5-7% of the time in the upper 13.8 m (42 ft). Below this level, the probe predicted volumetric moisture contents below the range of ±5 moisture units.

5 CONCLUSIONS

Each SMP profile followed the actual laboratory data trends at all three locations. The geologic units with higher clay contents had higher percent volumetric soil moisture values, whereas geologic units in the vadose zone with higher sand contents had predictably lower volumetric soil moisture values. Beneath the water table, clay units might not be distinguishable with the limited data provided by these tools. However, used in combination with geologic data and the standard CPT measurements, the SMP can make a substantial contribution to the geologic/hydrogeologic framework required for characterization of any site. Additional studies are needed to evaluate further the performance of SMPs in predicting soil moisture in a variety of hydrogeologic conditions.

ACKNOWLEDGMENTS

This work was supported by the U.S. Department of Energy, Assistant Secretary for Environmental Management, under contract W-31-109-Eng-38.

REFERENCES


590
Handbook of vadose zone characterization and monitoring: 730. Ann Arbor: Lewis Publisher.

Development of a CPT probe to determine volumetric soil moisture content

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ABSTRACT: Presented in this paper are the theory of operation of the ARA SMS and details of the sensor design approach. Also presented are results from a field evaluation of the SMS. The field evaluation program was conducted at a United States DOE site where both soil moisture and resistivity were measured at three different sites. The soil moisture sensor can be used in conjunction with the Cone Penetration Test or deployed as a permanent sensor with the CPT. Determining moisture content with a cone penetrometer deployed SMS is rapid and minimally invasive. The mobility and push capacity of the CPT truck makes this sensor especially useful in lithologies with deep water-table aquifers and where ordinary techniques used to determine soil moisture would be unsuitable.

1 INTRODUCTION

Measurement of soil moisture is a key parameter used to evaluate the degree of contamination present within vadose zone soils. It is necessary to determine the potential for contaminant migration and possible environmental remedial options (i.e. bio-remediation). Soil moisture has been shown to correlate closely with pneumatic-permeability and the diffusion coefficient of vadose zone soils (Ref.1, 2, and 3). As a geotechnical parameter, soil moisture content is used to evaluate the performance of geotechnical structures such as pavements, foundations, earth dams, and retaining walls. Laboratory testing of soils demonstrates that an increase in soil moisture content can degrade the ability of a foundation to support structures. Increases in the water content of a pavement base course can lead to early failure of pavements systems. For some soils, such as the expansive soils found in the southern United States, increases in moisture content can lead to volume increases resulting in cracked floor slabs and differential building settlement.

Currently, soil moisture measurements are made using laboratory techniques on samples retrieved from a bore hole or test pit. While measurements can be made in the field, in general, these measurements are made after the field investigation and on a limited number of samples. Hence, questions arise as to how representative laboratory samples are of the actual field conditions. It is also costly and time consuming to obtain a soil moisture profile using traditional drilling and laboratory techniques.

Recognizing the need for a soil moisture, Ohio University funded Applied Research Associates, Inc. (ARA) to develop a higher frequency soil moisture probe. This effort resulted in a soil dielectric CPT module that incorporates a custom designed circuit board located downhole in the CPT probe. The board generates a 100 MHz oscillatory signal between two rings to measure the dielectric of the soil between the rings. Downhole electronics provides a simple DC output voltage which is proportional to the dielectric constant.

2 SMS OPERATION PRINCIPAL

The fundamental principal applied to the operation of the soil moisture probe is that dry soil, having a dielectric constant of roughly 3 to 6, will approach the dielectric constant of water (80) as the moisture content increases. A commonly used relationship between soil moisture and the dielectric constant was proposed by Topp, et al. (Topp, 190, 1985):

$$q = -5.3 + 2.92K_e - 5.5 \times 10^5 K_e^2 + 4.3 \times 10^5 K_e^3$$

(1)
where: \( q \) = volumetric moisture content, \( \% \\
\( K_e \) = dielectric constant

This equation is known as the Universal (Topp's) equation and has been applied to all soil types.

Conventional TDR measures the dielectric at very high frequencies (several hundred Megahertz) using specialized equipment and has been demonstrated to minimize the effect of salt content and soil type. This technique has been confirmed by Topp and Davis (Topp, 1980, 1985), Baker and Goddich (1987), and Zegelin (1990). ARA initially examined this method using TDR as a CPT technique to measure the soil dielectric. However, TDR can only be used over relatively short cable distances (generally less than 100 ft) as the TDR wave attenuates rapidly with increasing cable length. In soils with high electrical conductivity, the TDR signal is greatly attenuated, making interpretation of TDR data difficult. Another limitation of TDR is that the equipment used for measurement is expensive and requires a second computer in a CPT truck.

ARA chose to use a Resonant Frequency Modulation (RFM) approach to determine \( K_e \) and ultimately the soil moisture. This approach consists of installing a custom PC board in a CPT probe which is then interfaced with standard CPT equipment, eliminating the need for specialized measurement equipment. A second advantage of this approach is that cable distances are unlimited as all conditioning and processing of the signal occurs downhole, eliminating the effect of cable length induced signal attenuation.

As 100 MHz signals are difficult to transmit without using a coaxial cable, signal conditioning is conducted downhole which reduces hardware and cabling problems. The RFM signal is put through a downhole counter which outputs a TTL clock signal in the range of 20 KHz. The TTL clock frequency is then converted to an analog signal. This conversion was made in order to condition the signal into a 0-4 volt output, which is compatible with a standard CPT electronic cable and data acquisition system.

The ARA soil moisture sensor (SMS) module, shown in Figure 1, is installed directly behind the CPT probe. The combined probe measures the conventional CPT tip, sleeve, and pore pressure in addition to the simultaneous acquisition of electrical resistivity and dielectric constant. From this data a wide range of geotechnical properties can be calculated. For resistivity measurements, the array operates at a frequency of either 40 Hz or 400 Hz to avoid soil polarization effects. The SMS module measures two electrical parameters: 1) electrical soil resistivity, which is related to the conductivity of a soil and can be used to determine the degree of soil contamination; and 2) the soil dielectric constant, which is related to the soil moisture content and can be used in interpretation of ground penetrating radar surveys.

3 FIELD EVALUATION

An evaluation of three different soil moisture CPT probes was conducted by the DOE Argonne National Laboratory at the SRS (Argonne, 1997). The objective of the field evaluation program was to define the limits of the probes. Only the ARA SMS probe data is presented in this paper.

3.1 Savannah River Site Description

The Savannah River Site is located in Aiken, South Carolina, and is operated by the Westinghouse Savannah River Company. It has an environmental program for site characterization and remediation of all its inactive waste sites. During execution of the program, hundreds of permanent groundwater monitoring wells were installed, and geologic and stratigraphic information about subsurface conditions were gathered for various areas at SRS. The CPT soil moisture probe evaluations were conducted at the TNX and M Basin Area.

4 COLLECTION OF SMS AND SOIL SAMPLE DATA

Two sites at the TNX area and one at the M Basin Area were selected for testing of the SMS. These locations had enough distinctive geologic variability
to test the SMS under a variety of lithologic and moisture conditions. The sites were characterized by SRS personnel to determine the site geology and geotechnical properties and to also insure that the sites were uncontaminated. Characterization of the site consisted of collecting undisturbed Shelby tube soil samples which were field logged. The samples were carefully wrapped and transported to a geotechnical laboratory for determination of the soil wet and dry density, specific gravity and gravimetric soil moisture. The gravimetric water content and dry density were used to calculate the volumetric water content, $\theta$, as:

$$\theta = \frac{w \gamma_s}{\gamma_w}$$ (2)

where $w$ = gravimetric water content
$\gamma_s$ = dry density
$\gamma_w$ = specific gravity of water

The three core hole locations were used as the SMS data collection sites. Each SMS probe was pushed twice, a minimum of 25 probe diameters apart (about 3.5 ft) centered around each core hole. The general configuration for conducting the SMS test is shown in Figure 2. Where possible, the sensor readings were collected prior to drilling to eliminate the possible effects of the soil disturbance on the sensor reading. The first push of one of the SMSs (1a) was at any point along the circumference of an imaginary circle. The second push (1b) with the same SMS was directly opposite the first push. This sequence was done for each of the three soil moisture sensors. The tip and sleeve readings were collected in addition to data from each moisture sensor. Collection of the CPT data was done to help in data interpretation should the geologic conditions change substantially between the sampling locations.

The SMS data was analyzed and compared to laboratory measured volumetric soil moisture contents. A comparison for all three test locations is shown in Figure 3. Plotted on the figure are a laboratory and field measured soil moisture as a function of the SMS output voltage as well as fits to the laboratory and field data (Shinn, 1997). As can be seen, the laboratory calibration curve was fairly accurate for low values of volumetric soil moisture; however, for volumetric soil moisture values greater than 25%, the laboratory curve begins to underestimate the best fit to the field data. This is in part due to the previously described limitations of the laboratory test program.

Also plotted on Figure 3 are the ±5% bounds for the test data. The bulk of the data falls within these bounds, indicating for the sites tested that soil type is a secondary variable. It should be noted that the clay component of the SRS soils is dominated by kaolinite, which has a low CEC. More active soils may show a wider range of response that was evident at the SRS site. As the field derived curve was based on a higher quality data set, it was decided to use the best fit to this data as the SMS calibration curve.

The CPT and SMS data from the TNX 1 location is plotted in Figure 4 and 5. Figure 4 contains the CPT tip, sleeve, friction ratio, pore pressure, and soil classification data. Presented in Figure 5 is the measured electrical resistivity and calculated soil moisture content, dielectric constant, and dry
density. The soil classification is repeated for comparisons purposes.
The SMS determined dry densities are presented for the portion of the profile below the water table. Below the water table the volumetric soil moisture and porosity are equal and it is possible to estimate dry and wet densities. By assuming a value for the specific gravity of the soil grains and using Equations 3 and 4, the dry and wet density can be estimated as:

$$\gamma_d = (1 - n) \delta \gamma_w$$  \hspace{1cm} (3)

and

$$\gamma_w = \gamma_d + n \gamma_w$$  \hspace{1cm} (4)

where

- $\gamma_w$ = wet density
- $\gamma_d$ = dry density
- $n$ = measured porosity
- $\delta$ = specific gravity

As shown in Figure 5, the SMS probe provides an excellent estimate of the soil volumetric moisture content and dry density of the saturated zone. A further comparison of the laboratory and SMS determined dry densities are presented in Figure 6. The dielectric constants presented in Figure 5 were back calculated using the SMS determined soil moisture content and Topp's equation.

5 DISCUSSION

In general, the SMS probe estimates the measured soil moisture within ± 5% using a non-soil specific empirical fit to the data. For a specific soil type, laboratory testing has indicated that the uncertainty can be reduced to ± 2%. The module's ability to measure material dielectric in the laboratory has been well documented, and the CPT SMS module development has undergone both laboratory and field testing. The laboratory testing has demonstrated that changes in resistivity have only a minimal effect on the SMS output, in contrast to earlier attempts at lower frequencies which were unfavorably influenced by the soil resistivity. An additional benefit has been the ability to provide detailed profiles of the porosity and wet density below the groundwater table as shown in Figure 6.

This paper has demonstrated that the CPT can be used to measure profiles of the soil dielectric and soil moisture content of soils in situ. However, there are several variables which can affect the accuracy of the calculated soil moisture content. Variables which can influence dielectric constant measurements are summarized above and include:
- The sample heterogeneity, density and
chemical composition will affect the measurement. To minimize the effect of these variables the SMS should be calibrated to the in situ soil.

- Dissolved salt concentration in the pore water may change the slope of the calibration curve. This effect has been minimized for the ARA SMS probe.
- The temperature of the soil may affect this method. The dielectric constant of water varies linearly with temperature, decreasing with increasing temperature. Generally, the effect of temperature changes may be neglected when the fluctuation in temperature is less than ±5°C.
- The SMS cannot be used to measure the moisture content of frozen soils as there is no free water.
- The SMS exhibits spatial bias in that it is more sensitive to water contained in the material closest to the probe. The bulk of the response is due to the first 1 to 2 in. of material around the probe.
- Measurements closer than 20 cm to the surface may be affected by the air-soil interface.
- Dielectric probes may exhibit poor accuracy in clay soils. This effect has not been fully evaluated for the ARA SMS probe.
- Metals of either natural (e.g. ores, etc.) or man-made (e.g. barrels, etc.) origin may affect measurements if they are present in sufficient quantity and are within the volume of soil tested by the device.

The ARA SMS has been demonstrated to provide accurate estimates of the in situ soil moisture and density profile for one geology. It is expected that the probe will be accurate for all types of sands and less active clay types. However, the probe response in clays with high CEC (i.e. bentonites, etc.) has not been fully evaluated. With further field calibrations ARA believes that the SMS will be a valuable CPT tool for determining soil moisture for both the vadose and saturated zones and density of the saturated zone.

6 REFERENCES

Radio-isotope cone penetrometers and the assessment of foundation improvement

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ABSTRACT: Radio-isotope cone penetrometers (RI-Cones) were developed by the authors to measure basic soil properties in-situ. Their effectiveness under various soil conditions are now well established. This paper discusses the use of these RI-Cones in a land reclamation project in the northeast of Tokyo. An accurate density of the soil density is important in the successful completion of the project.

1. INTRODUCTION

Japan is a mountainous country where more than 70% of the landmass is covered by mountains. Population and various industries compete for the remaining 30% of the landmass. With the ever growing demand for space, geotechnical engineers are faced with worsening ground conditions on which to build the foundations, such as tunnelling in a very soft ground. Another option is to reclaim land. A lot of reclamation is done in Japan to alleviate the space shortage, Kansai International Airport, in the offing of Osaka, and the Haneda Airport in Tokyo are prime examples.

In any reclamation project, amount of material required is in millions of cubic meters and is placed by various methods. Various methods that can be used for such work is e.g., hydraulic placement, large barges for dumping of sand, or by the conveyor belt system are some of the options. Since the huge amount of material is required, a precise density measurement is required to calculate the volume of sand. Furthermore, once the sand is deposited, it is imperative that a check is made of the progress. Once again density profile can give a good indication of the progress.

To check the density of the soil in-situ, authors have developed RI-Cones. Their effectiveness under various soil conditions are now well established (Shibata et al., 1993, Minura et al., 1995). In this paper such a land reclamation work has been described and the role of the RI-Cones have been discussed.

2. ADVANTAGES OF PUSH TECHNOLOGY BASED MEASUREMENT

The advantages of push technology based measurements, of which cone penetration technology is one, have been listed by many researchers. Authors feel regulatory agencies in many countries are still not very enthusiastic about this technology and would like to emphasize upon it by mentioning some of them below related to this project and in general:

1. It provides continuous, subsurface data to aid site characterization;
2. Among all the known methods, CPT-based measurement disturbs the subsurface conditions, the least, as no drilling mud or fluid is required in its operation;
3. When compared with drilling and sampling operation, it is relatively cheaper;
4. Real time data is obtained;
5. Other sensors can be used with CPT; as authors have done with RI-Cones to facilitate the measurements of other soil parameters, and the possibility of others have been shown by Shrivastava and Minura (1996).

6. RI-Cones by no means replaces the sampling and analysis for the characterization of sites. For any big and important project sampling will remain essential, but RI-Cones can be a mean by which quick assessment of the site under consideration can be made during initial phases of site characterization.

7. RI-Cones have their own limitations like any other push technology based site characterization system; namely it can not pushed in a deposit made of boulders, cobbles and other such big particles and the NM-Cone data should be corrected for the presence of chloride ions if accurate measurement of natural water content \((w_c)\) is required in suspected marine or similar deposits.

3. SITE DESCRIPTION

A very large reclamation project has been undertaken
in the northern part of Japan. The purpose of the reclamation is to develop an alternate port facilities which can service the country north of Tokyo. Moreover, with the ever increasing trade, the port facilities in Tokyo and the vicinity shall reach their maximum capacity in very near future. The prospect of further enlargement of the present facility will lead to the already congested area to further congestion.

Since the natural harbor is not available, and the other supporting facilities are needed on the site for self sufficiency, the total area to be reclaimed in the finally tally shall be more than 500 hectares (ha). The sand for the reclamation work is to be obtained from the nearby sites, we shall call it mother sites and have been termed as Site A and Site B. Their geotechnical characteristics have been discussed below.

4. CONSTRUCTION SEQUENCE

4.1 General Characteristics

For the construction of port facilities under consideration, about 700 hectares is required of which a total area of more than 500 hectares is to be reclaimed. In the first phase, which is dealt in this paper, a total area of about 150 hectares will be reclaimed, which will house the power generation unit and the related facilities. Following characteristics stands out in this project:

1. at the reclamation site, the average depth of water is about 12 m;
2. at the reclamation site, the sea floor is generally made up of the alternating layers of sand and sand + gravel, though occasional presence of loam layers have also been detected, and
3. this reclamation project must be completed in short time.

Table 1 shows the utilization plan of land space when the project is operational in about 12 year time.

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Function</th>
<th>Area (Hectares)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Wharf facilities</td>
<td>213</td>
</tr>
<tr>
<td>2.</td>
<td>Harbour facilities</td>
<td>200</td>
</tr>
<tr>
<td>3.</td>
<td>Industry (Power generation and related facilities)</td>
<td>141</td>
</tr>
<tr>
<td>4.</td>
<td>Disposal site</td>
<td>23</td>
</tr>
<tr>
<td>5.</td>
<td>Green area</td>
<td>52</td>
</tr>
<tr>
<td>6.</td>
<td>Others</td>
<td>58</td>
</tr>
<tr>
<td>7.</td>
<td>Total area</td>
<td>695</td>
</tr>
<tr>
<td>8.</td>
<td>To be reclaimed</td>
<td>502</td>
</tr>
</tbody>
</table>

4.2 The Construction Sequence

In the first phase, caisson wall is constructed to enclosure the area to be reclaimed. Once the wall is constructed it is left to settle under the forces of its own weight. Once the caisson wall is established the pouring of sand by means of conveyor belt (at this site) from the shore side is affected. Briefly, following process is followed in determining the geotechnical properties of the various sites:

1. physical properties of the sand to be used from the Site A and Site B in the reclamation project is determined by the proper means and preferably in-situ. R1-Cone system is used to determine various geotechnical properties in-situ, which can give the $p_s$, $D_s$ and $D_p$ profiles continuously along with measuring regular CPTU parameters. Other method used at this site is modified Bishop’s Sampler and the SPT;

2. sample is procured (modified Bishop’s sampler) and various physical tests are run on the sample to characterize the sand in the laboratory;
3. sand is excavated and placed on the conveyor belt, at the present site, to be transported to the reclamation site and is poured. Some of the characteristics of the conveyor belt systems used in this project is shown in Table 2.

<table>
<thead>
<tr>
<th>Capacity, $Q$ (m$^3$/h)</th>
<th>Width (mm)</th>
<th>Speed (m/min$^1$)</th>
<th>Total Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC-1</td>
<td>5,500</td>
<td>1,600</td>
<td>195</td>
</tr>
<tr>
<td>MC-2</td>
<td>5,500</td>
<td>1,500</td>
<td>195</td>
</tr>
<tr>
<td>MC-3</td>
<td>8,000</td>
<td>2,000</td>
<td>1,650</td>
</tr>
<tr>
<td>LC</td>
<td>8,000</td>
<td>2,000</td>
<td>2,604.5</td>
</tr>
</tbody>
</table>

4. sand is poured in an arc sweeping manner so that the sand can spread as evenly as possible over the desired area.
5. once the desired depth is reached, the dry density of the deposited sand is measured by the R1-CPTU as well as determined in the laboratory on the samples obtained. Schematic diagram of various measurement points are shown Fig. 1.

5. GEOTECHNICAL PROPERTY OF THE SITE

5.1 Geotechnical properties of sand Site A and Site B

R1-Cones alongside SPT and modified Bishop’s sampler was used to obtain various geotechnical properties at these sites including reclamation sites. R1-Cones were used one time at each of these sites. Modified Bishop’s sampler is briefly described below and the data from various boreholes are presented in Table. 3 and Table. 4 for Site A and Site B respectively. R1-Cones data are shown in Fig. 2 (Site A) and Fig. 3 (Site B). Also plotted in the same diagram is the $N_{mp}$ values.
Hanzawa and Matsuda (1978) using a stationary piston and reducing the diameter to 50 mm. One of the main advantages of this modified sampler is that the end samples of rather low density can be obtained without the loss of loose sand. However, the disadvantage being the sample must be small enough to be retained by capillary force. Samples were obtained to determine the physical properties of soil as well as to carry out testing by remolding to the natural state. Some of the results are presented and discussed below.

5.2 Geotechnical properties of reclaimed site

As shown in Fig. 1, the measurement was carried out along two lines, i.e., both sides of the sand deposited along Line 1 and Line 2. Due to lack of space and also not all the data is in public domain, only data related to RI-Cones have been presented. Fig. 4 and Fig. 5 shows the data obtained by means of RI-Cones. Also plotted in the diagram is the sample dry density as determined in the laboratory obtained from the modified Bishop's sampler. Two things stand out immediately: one being the data density is almost negligible, i.e., very few data points; and the second being the scattering along the

### Table 3. Physical characteristics of some of the borehole at Site A.

<table>
<thead>
<tr>
<th>Borehole No.</th>
<th>P13</th>
<th>P14</th>
<th>P18</th>
<th>P24</th>
<th>P33</th>
</tr>
</thead>
<tbody>
<tr>
<td>Physical Properties</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>depth (m)</td>
<td>4.15m</td>
<td>6.15m</td>
<td>13.15m</td>
<td>18.15m</td>
<td>26.15m</td>
</tr>
<tr>
<td>D (cm)</td>
<td>2.099</td>
<td>2.724</td>
<td>2.682</td>
<td>2.729</td>
<td>2.691</td>
</tr>
<tr>
<td>w_s (%)</td>
<td>11.0</td>
<td>11.0</td>
<td>14.4</td>
<td>20.3</td>
<td>10.6</td>
</tr>
<tr>
<td>pebble (20mm-75mm) %</td>
<td>6</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>33</td>
</tr>
<tr>
<td>sand (2mm-20mm) %</td>
<td>79</td>
<td>92</td>
<td>46</td>
<td>33</td>
<td></td>
</tr>
<tr>
<td>silt (5mm-2mm) %</td>
<td>10</td>
<td>4</td>
<td>38</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td>clay (0.5mm) %</td>
<td>2</td>
<td>2</td>
<td>16</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Dm (g/cm³)</td>
<td>2.556</td>
<td>1.632</td>
<td>0.994</td>
<td>1.247</td>
<td>1.478</td>
</tr>
<tr>
<td>Dmax (g/cm³)</td>
<td>1.182</td>
<td>1.248</td>
<td>0.696</td>
<td>1.041</td>
<td>1.030</td>
</tr>
</tbody>
</table>

### Table 4. Physical characteristics of some of the borehole at Site B.

<table>
<thead>
<tr>
<th>Borehole No.</th>
<th>P6</th>
<th>P10</th>
<th>P15</th>
<th>P19</th>
<th>P27</th>
</tr>
</thead>
<tbody>
<tr>
<td>Physical Properties</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>depth (m)</td>
<td>5.15m</td>
<td>5.45m</td>
<td>9.45m</td>
<td>14.45m</td>
<td>18.45m</td>
</tr>
<tr>
<td>D (cm)</td>
<td>2.68</td>
<td>2.728</td>
<td>2.741</td>
<td>2.703</td>
<td>2.733</td>
</tr>
<tr>
<td>w_s (%)</td>
<td>49.4</td>
<td>49.4</td>
<td>49.4</td>
<td>49.4</td>
<td>49.4</td>
</tr>
<tr>
<td>pebble (20mm-75mm) %</td>
<td>4</td>
<td>5</td>
<td>19</td>
<td>30</td>
<td>50</td>
</tr>
<tr>
<td>sand (2mm-20mm) %</td>
<td>66</td>
<td>40</td>
<td>77</td>
<td>62</td>
<td>45</td>
</tr>
<tr>
<td>silt (5mm-2mm) %</td>
<td>35</td>
<td>4</td>
<td>19</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>clay (0.5mm) %</td>
<td>15</td>
<td>2</td>
<td>3</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Dm (g/cm³)</td>
<td>1.389</td>
<td>1.391</td>
<td>1.296</td>
<td>1.728</td>
<td>1.747</td>
</tr>
<tr>
<td>Dmax (g/cm³)</td>
<td>1.399</td>
<td>1.398</td>
<td>1.580</td>
<td>1.385</td>
<td>1.399</td>
</tr>
</tbody>
</table>
Figure 2. RI-Cones data at Site A.

Figure 3. RI-Cones data at Site B.
Figure 4. RI-Cones data along testing Line 1 at No. 1-4.

Figure 5. RI-Cones data along testing Line 2 at No. 2-7.
depth when and if the data at regular intervals from Bishop’s sampler is available for the depth under consideration.

6. DISCUSSION

For Site A and Site B, the source sites, N_{150} values have also been plotted. For Site A, a good match is seen, however, at Site B much scattering is noticed. Inside of scattering seen at Site B, there is almost a linear relation between q_c and N_{150}, which varies between 4 and 6 times of N_{150}. Such a relationship has also been proposed by several other researcher as well.

Comparative data was available for only one site, designated as No. 1-4 (Fig. 4). Shown are the profiles of q_c, u_s, density and water content. In the density profile plot, both the pre- and post-reclamation data are plotted. Pre-reclamation plots are from a depth of 5.5m until the refusal is met, just above 9 m. Even though the changes are very small, it can be picked by deploying RI-Cone. Also plotted is a point obtained by modified Bishop’s sampler. At site number No. 2-7 (Fig. 5), the data density from modified Bishop sampler is more than the previously mentioned site, however, one notices a scattering in data, especially, between 5m and 7m depth. Such discrepancy shall either lead to under- or overestimation in the quantity of the material required and hence changing the economics of the project.

7. CONCLUSIONS

RI-Cone have shown to measure some of the very fundamental soil properties very accurately in-situ. It has been shown that the compared to various sampling methods, whether frozen sampling, nuclear borehole logging (see accompanying paper, Minura and Shiravastava, 1998), modified Bishop’s sampler, RI-Cone results are consistent and accurate. Authors were not able to discuss the economic aspect of such a huge reclamation project, however, one can easily realize the saving in the cost of reclamation material if the dry density is measured accurately and vice versa.

8. ACKNOWLEDGMENTS

Authors would like to extend their thanks to the staff of Soil and Rock Engineering Co., Osaka, for carrying out test at this site; to Dr. Hanzawa, Director, Toa Research Center, Yokohama for providing many insights into the reclamation project; Dr. Ali Porbaha, Toa Research Center for many helpful discussion. Finally, to my colleagues at the PHRI for providing conducive atmosphere in writing this paper.

9. REFERENCES


Application of advanced borehole geophysics in site geological, geotechnical and environmental characterization

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ABSTRACT: Geological and environmental subsurface characterization programs usually consist of simple, uncalibrated borehole geophysical logging suites consisting of natural gamma and resistivity tools. Although basic lithologic and hydrogeologic data may be interpreted from these curves, it is difficult to determine the geologic conditions that control contaminant flow and transport or to evaluate basic rock properties such as density and porosity, that may impact geotechnical considerations. Advanced logging tools, such as magnetic resonance, digital sonic and spectral gamma, may be correlated to standard tools such as CPT and ground penetrating radar data. Correlating advanced geophysical data to standard tests in recent programs has demonstrated that 1) stratigraphic intervals with variable geohydrologic and geotechnical properties may be defined, 2) sediment properties, such as Poisson’s Ratio, may be directly measured and related to specific geological members, and 3) the direct association of certain contaminants to a specific stratigraphic interval, may be possible.

1 INTRODUCTION

It is often difficult to determine the shallow geological conditions associated with environmental and geotechnical studies. Deeper studies are generally more predictable either because the scale of investigation is larger (thick, deep zones such as in oil and gas exploration) or because the need for fine detail is not as great (basin studies, for example, usually do not require meter scale resolution). This paper presents two examples of the use of advanced geophysical logging techniques, usually found in deeper exploration, for enhancing shallow investigations. These examples are from the US Department of Energy Savannah River Site (SRS) in the southeastern US Atlantic Coastal Plain (Figure 1). Data from advanced borehole geophysical logs are correlated to 1) standard cone penetrometer soundings (CPT) and 2) ground penetrating radar (GPR) data. Geophysical parameters, both measured and calculated, may be assigned to stratigraphy inferred from the CPT and GPR data.

2 BACKGROUND GEOLOGY

The Atlantic Coastal Plain is a seaward-thickening wedge of unconsolidated and semi-consolidated sediments ranging from Late Cretaceous to Holocene in age. Sediments include intercalated sands, silts and clays with localized carbonate muds and occasional fossil shell mounds. Carbonates are generally sparse to the north and west and become more abundant seaward (Fallaw & Price, 1995). The Late Cretaceous to Holocene sediments are approximately 300 to 400 meters thick. Since the water table is very shallow, these sediments are generally saturated to within a few meters of the surface.

The present day surface geology of the study area includes formations that vary in age from Upper Tertiary to Quaternary. Pleistocene and Holocene rivers incised valleys through the older sediments, depositing bar sands interbedded with channel sands and clays. These sediments unconformably overlie older late Tertiary units which were exposed during periods of regression. The older units were deposited in various depositional settings ranging from fluvial to shallow marine (Fallaw & Price, 1995).

Near surface sediments that make up geologic formations in the area usually evaluated for geotechnical or environmental studies, include the “Upland Unit” which consists of poorly sorted sands and clays interbedded with conglomerates generally thought to have been deposited in a fluvial...
environment (Fallaw & Price, 1995). Underlying the "Upland" are the Tobacco Road Formation, the Dry Branch Formation, the Griffin's Landing and Utley carbonate members of the Clinchfield and Santee Formations, and the sands of the Santee Formation. The Tobacco Road Formation is composed of poorly sorted sands interbedded with clay, mostly kaolinite, lenses. The presence of marine fossils and glassy concretions indicate a shallow marine environment to transitional tidal flat to back-bay setting (Fallaw and Price, 1995). The Dry Branch Formation consists of sands, clays, calcareous siliciclastics and some carbonates from a marine unit, with the depositional environments range from back-bay and lagoonal muds to barrier islands. The Clinchfield and Santee Formations consist of interbedded sand and carbonates. The carbonates vary between limestone muds (micrite) and shell mounds. Because of the multiplicity of depositional settings, erosional influence and ongoing diagenesis in sediments in the near surface, the vertical and lateral variation of sediment character may be great. Therefore, a detailed subsurface characterization is often required.

3 DISCUSSION

Near surface geological investigations for engineering or environmental characterizations require a knowledge of the shallow subsurface and mapping of near surface strata that may involve paleochannels, bars or other depositional features. Shallow buried channels and pits, while obvious in outcrop, are seldom obvious from surface characterization. The presence of these shallow features, particularly in the alternating sands and clays of the southeastern U.S. coastal plain, is important for understanding hydrogeologic and geotechnical conditions associated with these lithologies. Shallow fractures and channels may act as preferential pathways for horizontal fluid flow or as conduits for vertical flow between aquifer systems. Variations in permeability and porosity, may affect fluid flow and contaminant transport. The lateral variation in clay content and type may affect contaminant dispersion. The presence of deeper processes such as carbonate dissolution or liquefaction, may affect differential soil strengths and conditions, and may be a geotechnical hazard.

A typical geotechnical characterization or hydrogeological boring geophysical log is seen in Figure 2. These logs usually consist of a natural gamma ray, and short and long normal resistivity data. Occasionally, spontaneous potential and single point resistivity curves are also obtained. This combination of subsurface data is generally interpretable for basic lithology (sands versus clays) and fluid content or level. Most often, these logs are not calibrated to international standards. Very little of the sediment character, geochemistry, or depositional environment, necessary for detailed characterization, is discernible.

Advanced geophysical log data are routinely obtained in hydrocarbon exploration. There are numerous measured and derived parameters that may be obtained from the art borehole geophysical logs. Typical suites of log derived geophysical data include:

- Spontaneous Potential, Caliper,
- Natural Gamma, Gamma-Gamma,
- Spectral Gamma (K, U, Th),
- Multi-Depth Resistivity/Induction.

![Image](image_url)

**Figure 1.** The study area lies within the unconsolidated sediments of the southeastern US Atlantic Coastal Plain. "EF", "UTBF" and "AF" are regional faults. The Savannah River Site comprises an area of approximately 740 square kilometers.
Density, Neutron and Sonic Porosity,
Magnetic Resonance Porosity and Permeability,
Bulk Density,
Photoelectric Value, and
Compressional and Shear Sonic.

From these measured and derived log data, it is possible to calculate other geophysical information such as:

Poisson’s Ratio,
Shear and Young’s Modulus,
Hydraulic Conductivity, and
Lithology.

With suites of data that contain lithologic, physical property and fluid information, it is possible to construct a boring log that provides a correlation standard for other geotechnical, environmental and geological characterization tools. For example, these data may be associated with standard cone penetrometer soundings or ground penetrating radar data to establish the lateral geological conditions and continuity across an area. Figure 3 is a comprehensive suite of data for a correlation geophysical boring. This log contains 18 data tracks. For this discussion, all curves are related to the gamma curve.

Table 1 compares some of the calculated, log derived data for the shallow horizons noted on Figure 3. The parameters in this table are representative of data that may be useful for environmental, geotechnical, or geological characterization. These data may be associated with stratigraphic units and tracked across an area and therefore are helpful for contaminant flow and transport and in the lateral evaluation of geotechnical data.

3.1 Comparison to Cone Penetrometer Data

Once the bare geophysical log correlation is made, it is then possible to correlate standard cone penetrometer (CPT) results to the geophysical log as seen in Figure 4. Cone penetrometer data is routinely used for geotechnical and environmental characterization. This figure compares a series of CPT’s obtained across an area adjacent to the geophysical correlation boring (GCB-1, Figure 3). The CPT’s are spaced along a line at approximate 30 meter intervals. The correlation boring data is approximately 20 meters from the line of CPT’s.
Figure 3. Summary log showing spectral gamma data, multi-depth induction resistivity, porosity, permeability, density and acoustic information. Core data is from detailed lithologic examination of wireline core. The interpretation of depositional environments, geologic formation and age are also shown. Depths are in feet (50 feet equals 15.24 meters). The horizontal bars are contaminant values in mg/l.
Figure 4. CPT cross-section correlated to geophysical log boring GCB-1 (Figure 1). "C", "R", and "D" are the spectral gamma ray, deep induction resistivity and density logs respectively. In areas where there is little change in character of the CPT data (indicating similar lithologies) it is possible to correlate through with values derived from the geophysical log. The geophysical log is approximately 20 meters from the line of CPT's.

Figure 5. Z-plot of deep induction resistivity and spectral gamma data versus calculated clay (CGR) content by depth. Gamma counts are in API units. Resistivity data is in ohm-meters. Mud values are in percent of total. The three boxed areas represent zones with relatively high resistivity values and relatively low gamma values suggesting clays. The arrows on the log curves indicate zones in the near surface where the low gamma high resistivity are greatest and may provide an interval that is radar reflective. Deeper zones for possible radar horizons were not considered because field experience suggested that 30 meters was the maximum depth of signal penetration in the area.
Figure 6. Correlation panel for geophysical data, derived core data, Friction Ratio, synthetic radiogram and field radar records. The correlation lines correspond to those on Figure 5. Depths on the log tracks are in feet (1 foot equals 0.3 meters).

Figure 7. Ground penetrating radar profile along the line of CPT's. The interpreted horizons are from the CPT correlation panel (Figure 3) and are tied to two of the CPT's located on the profile. Data are 25 MHz obtained in a CMP mode using 12 one meter offsets.
fairly consistent across the area. However, the "CZ" (hydrogeologic confining zone), does have a variable CPT signature and the geophysical log parameters shown, and calculated values from Table 1, may vary across the area. The amount of consistency or variation is based upon the interpreter's experience with minor variations in the data.

Once the geophysical data has been extrapolated across the CPT section, the additional contaminant data may also be extrapolated across the section. For example, Figure 3 contains a data track with horizontal bars corresponding to intervals with elevated tetrachloroethylene (PCE) and trichloroethylene (TCE). The measured permeability is also shown in this track. The PCE and TCE were detected by a head-space sampling of core data. The intervals with elevated contaminants, associated with zones of higher permeability, may be tied to other geophysical curves and extrapolated across the CPT section.

3.2 Comparison to Ground Penetrating Radar Data

Ground penetrating radar data are often used in site characterization for the direct detection of buried debris, the water table surface, geological relationships, and occasionally for direct contaminant detection. Data from advanced borehole geophysical logging may be used to provide subsurface parameters to calibrate this tool. Figure 5 is a z-plott of borehole data derived from Figure 3. A z-plott is a graphical presentation of two parameters compared against a third in depth.

Figure 5 compares deep resistivity values against the calculated gamma data from the spectral gamma log, plotted against log calculated percent of mud, or clay, in the boring. Clay zones, or sand/clay interfaces, are intervals of high dielectric change and are often radar reflective and therefore may be mapped. The values on the graph correspond to depth intervals on the adjacent log curve plot (represented by bars). The arrows on the log curve plot refer to enclosed values from the graph, with high resistivity values (low conductivity), and high clay content, that are likely to be reflection events in the GPR data. This tier, from the z-plott data to the gamma ray correlated to specific depth intervals, defines specific strata that may be radar reflective and identified. Since the permeability, porosity, hydraulic conductivity, clay types density, etc., may be associated with the gamma curve (or depth) then these parameters may be mapped using the radar stratigraphic horizons. Because the borehole geophysical data is tied to zones that are correlatable with the stratigraphic information provided by the GPR, it is possible to estimate lateral stratigraphy and changing geological conditions.

Figure 6 is a synthetic radargram, calculated from the CPT and resistivity data. This figure ties the borehole geophysics, CPT and radar data. Stratigraphic intervals from the geophysical log data corresponds with "breaks" on the CPT data that correlate with reflective horizons on the synthetic radargram. The synthetic radargram may be correlated with the actual field radar results. Based on this comparison, it becomes possible to track log derived parameters and calculated parameters (from Table 1) continuously across an area with more certainty.

Figure 7 is the actual radar profile across a portion of the CPT transect. It is obvious from this figure that there are lateral variations between two of the CPT's not immediately interpretable from Figure 3 or Figure 4 data. There are bar and sheet sands in the Upland Unit adjacent to channel fill materials, all of a similar composition (as based on the CPT data). The data from the borehole geophysics may be compared to this variation and estimates made of the effects of lateral continuity.

4 CONCLUSIONS

Because it is possible to transfer point location parameters (from a borehole with advanced geophysical data) to multiple points (CPT locations, through correlation) and then to continuous data (GPR), it becomes possible to evaluate the lateral subsurface variation of the parameters. Advanced logging tools, such as magnetic resonance permeability and porosity, digital sonic and spectral gamma, as well as standard tools such as density and neutron porosity, may be correlated to standard geotechnical tools such as CPT and environmental tools such as ground penetrating radar. Petroleum industry standard instruments such as the neutron and density porosity, and induction logging tools, greatly enhance geological and geotechnical understanding of the shallow subsurface. Stratigraphic intervals, defined from the boring geophysical data and correlated through other borings or CPT's, and further defined by GPR, may have specific physical (such as Poisson's Ratio), and or geochronal (such as U/Th ratio), attributes assigned.

Results from recent programs have demonstrated that 1) stratigraphic intervals with variable geohydrologic and geotechnical properties may be defined, 2) sediment properties, such as Poisson's Ratio, may be directly measured and related to specific geological numbers, 3) the direct association of certain contaminants to a specific stratigraphic interval, may be possible.
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**REFERENCE**

Geoenvironmental direct push technology
A novel technique for detection of contaminated groundwater in situ

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ABSTRACT: The authors have recently developed and patented a novel cone penetration technique (CPT) for detection of dissolved phase contamination in groundwater. The device is based upon traditional laboratory techniques of purge and trap stripping of volatiles from groundwater samples. The purging technique is applied downhole in a CPT probe to reduce the potential loss of volatiles during sample handling activities. Once the downhole sample is purged, the plume gas is transferred to a heated transfer line and analyzed using High-Speed Gas Chromatography (HSGC) techniques. The new vapor analyzers are rapid (detection in less than ten seconds) and have significantly improved detection limits over other vapor analyzers and CPT sensing approaches. A prototype of the CPT In Situ Purge (CISP) device has been preliminarily tested and the results are presented along with future improvements that are planned for the Phase II effort being funded by the United States Department of Energy (DOE).

1 INTRODUCTION

Over the past five years several new techniques for in situ measurement of chemical contamination of soils have been developed, however most of these are geared towards a field screening level of detection. Typical approaches consist of resistivity or conductivity devices, fluorescence systems, Raman systems, and several others. One aspect common to nearly all these systems is detection limits that range in the hundreds of parts per million (ppm). Although detection limits in the hundreds of ppm are useful for determining the extent of pure phase plumes, they are not able to address the much larger problem of delineation of dissolved phase plumes. For delineation of these plumes, detection limits in the parts per billion (ppb) range are needed. Typically, these detection limits have only been achieved using conventional sampling techniques with on-site analytical testing services if fast turn-around times are needed for guiding the investigation. Due to the high cost of setting up a mobile laboratory, most samples are typically sent-out for analysis and the results are not available for several days, hindering the flexibility of the site characterization plan.

A new approach that is based on applying laboratory analytical techniques and procedures directly in situ is being investigated under funding from the Department of Energy. This approach takes laboratory extraction techniques of thermal desorption and groundwater purging and combines them with a CPT approach to produce a gas stream containing volatile contaminants stripped from either soil or groundwater. A prototype system applying these techniques has been developed by the authors for the characterization of fuel compounds in situ. The project is now entering a second phase where the same tools will be adapted for chlorinated compounds.

The up-hole analysis of the vapor stream is performed using a High Speed GC (HSGC) approach. For the work performed during this development a laboratory-grade Varian GC was modified for high-speed application by Chromotec, Inc. This tool allowed speciation of fuel components (BTEx) within 10 seconds and enhanced quantification of the compounds using a 90-second analysis time. The system was tuned for detection limits of 10 to 50 ppb for BTEX compounds. Recently several other sophisticated gas
analyzers have become available that achieve ppb level detection limits with speciation and fast turn around. One such analyzer developed by DOE that will be evaluated with the downhole CPT tools is the Surface Acoustical Wave/Gas Chromatograph (SAW/GC) device from Electronic Sensor Technology. This device has detection limits of 10 to 100 ppb for many chlorinated compounds with an analysis time of approximately 10 seconds.

This paper presents the development and initial test results of the in situ purge device designed for tracking and delineating dissolved phase plumes. The major advancement over other CPT sensors for either fuel or chlorinated solvents is that this approach offers is that the detection limits have descended from the hundreds of ppm range into the tens to hundreds of ppb. This is nearly three orders of magnitude below the detection limit of traditional CPT sensors, and allows the numerous benefits of CPT approaches to be used for delineation of dissolved phase plumes. In addition, the probe can easily be modified into a monitoring point that operators can test periodically to track changes over a long period of time.

2 BACKGROUND ON PURGING TECHNOLOGY

The CPT In Situ Purge (CISP) device is based upon the purge and trap approach commonly used in EPA methods (EPA 1990) for the stripping of volatile organic compounds from groundwater samples. The technique is based upon bubbling an inert gas (nitrogen or hydrogen) through a groundwater sample at ambient temperature so that the volatile organic compounds are efficiently transferred to the vapor phase.

Henry’s Law governs the equilibration between the solute gas phase and the dissolved phase of a compound at low concentrations (Mackay and Peterson 1981, Mackay 1979). Henry’s law is commonly presented as:

\[ C_v = \frac{P}{H} \]  

(1)

where \( C_v \) is the concentration of the solute in water on a molar basis (mol/m³), \( P \) is the partial pressure of the compound in the gas phase (atm), and \( H \) is the compound’s Henry’s law constant (atm·m³/mol). As the purge gas, which initially contains no volatile organic compounds, is bubbled through the groundwater, volatile organic compounds in the groundwater are transferred to the purge gas as the system tends toward local partitioning equilibrium between the bubbles and the solution.

In ordinary laboratory practice, the vapor is then swept through an adsorbent column where it is temporarily adsorbed on to the surface of the column. Once purging is complete, the adsorbent column is heated and the trapped volatile organic compounds are flushed with an inert gas through a column to a gas chromatograph.

3 CISP PROBE DESIGN

To apply this technique for use with CPT equipment, a modified groundwater sampling device called the ConeSipper® is utilized. Groundwater is drawn through a filter mechanism into a purging chamber located down-hole in the probe. The sample is purged and the vapor containing the inert gas and volatile contaminant is transported through a heated transfer line to the up-hole analyzer. In the CPT/HSGC system, the focusing inlet on the vapor analyzer replaces the conventional adsorbent column. Performing the purge in situ decreases the potential amount of sample loss frequently encountered in sample retrieval/laboratory analysis methods. A schematic of the CPT In Situ Purge device is shown in Figure 1. The primary features of the CISP are the sampling system, the purging system, and the back-flushing system.

A sample enters the sample/purge chamber through the slotted sleeve, sintered screen, and solenoid valve under hydrostatic pressure. The solenoid valve closes when the proper volume has entered as determined by the conductivity electrode. The sample in the chamber is purged with nitrogen bubbled through the purge gas. The stripped organics are carried to the surface via the heated purge gas transfer line, where they are analyzed by HSGC.

3.1 Sampling system

The groundwater sample taken into the CISP probe passes through three essential hardware components: two sample filtering components, and a solenoid valve. The sample filtering components are the
3.2 Purge system

Two concerns were addressed in the design of the purge system for the CISP probe. First, purge efficiency was considered. Studies have shown that purge efficiency (recovery) increases with increasing ratio of headspace to sample volume in the purge chamber (Doskey 1996). A 2:1 ratio was the greatest that could be achieved while suppressing surface bubble formation and keeping the probe length reasonable for field operations. Another factor affecting purge efficiency is the ratio of the surface area at the interface of the purge gas with the sample versus the volume of purge gas. Minimizing purge gas bubble size maximizes this ratio, and thus, all other factors being equal, optimizes purge efficiency. The pore size of the purge gas frit was chosen to be 2 microns. This size minimized bubble size while not requiring excessive pressure to operate and maintain an essentially linear pressure versus flow rate relationship to ease control of the purge process.

The second concern addressed in design of the purging mechanism was field serviceability, or the minimization of potential downtime. The internal workings of the CISP probe were constructed of two pieces that join at the base of the sample chamber. This allows the purge gas frit and most sealing o-rings to become accessible for replacement by removing a single bolt holding the two halves together. The solenoid valve that controls flow in and out of the sample chamber is also accessible for service or replacement by removal of the same bolt. This design feature is essential for minimizing downtime in the field should components fail or become fouled.

3.3 Backflushing system

Once the analyst of interest has been purged from the groundwater sample and analyzed by the up-hole vapor analyzer, the CISP probe is backflushed. Backflushing is achieved by forcing the purged groundwater sample back out into the formation through the solenoid valve and sample filtering components. Because the analyst of interest has been purged from the sample, the sample becomes a solvent for cleaning the components along the sampling/backflushing flow path in preparation for the next sample. Cleanliness of the sample chamber and transfer line is verified after backflushing by analyzing purge gas blown through the empty sample chamber with the solenoid valve closed. The solenoid valve remains closed while the probe is advanced to the next depth at which analysis is desired.

4 CISP PROBE OPERATION

The probe is advanced to the desired depth by normal CPT procedures. With the probe at the
desired analysis depth, the operator switches a three-position toggle switch on the up-hole control panel to the fill position. In this position, power is supplied to the solenoid valve, opening it to allow water to enter the sample chamber under hydrostatic pressure (pressure in the chamber is atmospheric). A light emitting diode (LED) remains lit while the down-hole solenoid valve is open. A second LED indicates when the pre-set sample volume (5 ml) has entered the chamber and triggered the level sensor. The time to fill is measured and recorded. As the sample chamber fills, backflow of water into the purge gas inlet at the base of the chamber is prevented by the stainless steel frit, whose pore size (2 microns) is too small to allow flow-through under the hydrostatic pressure created in the chamber (50 mm H2O maximum).

As the sample chamber fills, the rate of inflow can be monitored up-hole on the gas flow meter, which is connected to the outlet of the heated transfer line. The gas flux out the up-hole end of transfer line equals the volumetric flow rate of water entering the sample chamber because the water is displacing gas in the chamber and sending it up the transfer line. The transfer line is open to the sample chamber at the top of the head space.

With the groundwater sample in the sample chamber, the three-position toggle switch is set to the neutral position. In this position, the solenoid valve remains closed. An up-hole toggle valve in the purge gas supply line is then opened, allowing purge gas (grade UHP nitrogen) to bubble through the water sample. Contaminants in the purge gas are trapped on the focusing inlet of the analytical instrument for the five-minute purge duration and then analyzed.

When the HGSC has finished sub-sampling the purge gas from the transfer line, the operator backflushes the probe by first closing the furthest upstream toggle valve in the transfer line, then pressurizing the purge gas line to 30 psi greater than hydrostatic pressure outside the probe, and finally switching the three-position toggle to the drain position. In the drain position, the down-hole solenoid valve is powered up and remains open regardless of the state of the level sensing circuitry. The probe is allowed to backflush for a period equal to at least two times the product of the fill time and the ratio of the external hydrostatic pressure to the excess internal pressure (30 psi). For example, if the chamber filled in 90 seconds at 20 psi external hydrostatic pressure, then the backflush time at 50 psi internal pressure (20 psi external plus 30 psi excess) would be two times 90 seconds, times the ratio of 20 psi to 30 psi, or 120 seconds. Because the water flushed back through the sample entry path has been purged of the contaminants of interest, this water becomes a solvent for cleaning contaminants from the probe’s sample flow path as it is discharged back into the formation.

At the end of the backflush period, the three-position toggle switch is set back to the neutral position, the purge gas supply is toggled off and set back to its purge mode pressure, and the transfer line toggle valve is opened, relieving the pressure in the lines and sample chamber. Following a backflush, the cleanliness of the purge chamber can be checked prior to the next analysis by analyzing purge gas blown through the empty sample chamber.

5 PRELIMINARY TEST RESULTS

The CISP module was tested during a preliminary demonstration of its and the thermal desorption probes at a site in Vermont. Due to debris buried at the site, a 1.75-inch diameter pilot hole was pushed to a depth of 12 feet. The water table was measured in a nearby monitoring well was at 7.3 feet below the ground surface. The probe is greater than 2 inches in diameter, so the groundwater that was sampled came directly from the formation pore space, and not from side-wall drainage into the pilot hole. The probe’s sample chamber successfully filled with groundwater at depths of 8, 9, and 10 feet below ground surface - demonstrating the probe’s ability to collect and discharge multiple samples during a single penetration.

Figure 2 presents results from the groundwater sample analyzed at a depth of 10 feet. The primary peak shown in the chromatogram at 7 seconds is Toluene, and the peak at 19 seconds is Xylene. A confirmation soil sample was collected from a neighboring penetration at the same depth and the analysis indicated 890 ppb of Toluene and 1680 ppb total Xylene. Benzene concentration was determined to be 94 ppb, indicating a fairly weathered fuel. To demonstrate that the peaks shown were due to contamination in the groundwater, and not residual in the system, the purged sample was discharged back into the ground and purge gas blown through the transfer line, and the empty sample chamber was analyzed. The second chromatogram in Figure 2 shows the result of this analysis (a system blank).
The water at this depth contained a fair amount of hydrocarbons. The blank shows a minute amount of carryover of the larger peaks, three orders of magnitude less than the groundwater sample that preceded it.

6 SUMMARY AND CONCLUSIONS

The initial success of the CPT in situ Purge device has prompted additional development and evaluation of the approach for additional compounds, including chlorinated solvents that plague so many federal facilities of DOD and DOE. The device has proven to be effective and able to test multiple depths during a single penetration with negligible cross contamination. The device worked very effectively in sandy soils, however improvements in the filtering system are needed to prevent clogging as the probe passes through silts and clays. This item is being addressed under the phase II effort. Finally, the combination of the CISP with improved vapor analyzers transitions the detection limits nearly three orders of magnitude. These improvements in both the downhole CPT techniques as well as the vapor analyzers allows these tools to be used for more than free phase delineation, but also for the dissolved phase plume delineation.

REFERENCES

Doskey, F. May 1996. Personal communication. Argonne National Laboratory, Argonne, IL.


Integrated opto-electronic chemical sensor for BTEX detection in cone penetration testing

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ABSTRACT: Laboratory evaluation of the performance of an integrated opto-electronic sensor, developed for the quantitative in-situ detection of benzene, toluene, ethylbenzene, and xylenes, was carried out in an aqueous / Ottawa sand matrix. The sensor used in the experiments was miniaturized for ultimate inclusion in a trailing module of a cone penetrometer, and a series of proof-of-concept experiments which used benzene, toluene, and xylenes as contaminating chemicals in both aqueous and saturated sand matrices were carried out in order to demonstrate the viability of the integrated optic sensor's operation in a soil environment.

1 INTRODUCTION

Inclusion of an integrated optic (IO) chemical sensor, capable of quantitatively measuring chemical concentration, within a cone penetrometer offers many advantages over the more traditional methods of groundwater sampling and analysis. Because the IO sensor operates in real-time and reversibly, contaminant concentration with depth can be mapped essentially continuously, during the cone push. In addition, there is no exposure of the groundwater to the atmosphere during sampling, and there is no need for transportation of the sample to a laboratory for analysis. While the integrated optic chemical sensor will not eliminate the use of monitoring wells, it can help optimize placement of these expensive wells and also reduce the number of wells required for adequate coverage of a site.

This article details the operation of the newly-developed integrated opto-electronic sensor used for the detection of benzene, toluene, ethylbenzene, and xylenes (BTEX chemicals). During this study, the sensor was miniaturized for inclusion within a trailing cone penetrometer module. While the sensor used in these experiments was developed for the detection of the BTEX chemicals, the design can be readily adapted to detect a wide variety of other chemicals as well, including chlorinated hydrocarbons and dissolved metals.

2 SENSOR OPERATION

The Hartman interferometer used in the testing program was originally developed at the Georgia Tech Research Institute (GTRI) for the detection of gaseous ammonia, NH₃ (Hartman et al., 1988; Hartman, 1990; Ross et al., 1991; Hartman et al., 1992). More recent versions of the interferometer are used for the measurement of pH (hydrogen ion concentration) and for the detection of the BTEX chemicals. Sponsorship for the integration of the sensor into a cone penetrometer was provided by the Army Research Office (ARO) in Raleigh, North Carolina. Phase I, which was a proof-of-concept stage was completed in June, 1997; Phase II is a joint venture between ARO and the US Army Corps of Engineers Waterways Experiment Station (WES), and includes the complete integration of the sensor into a trailing module of a cone penetrometer. Phase II is scheduled for completion in February, 1999.

The basic operation of the sensor relies on the use of a chemically-selective polymeric coating placed on an optically transparent waveguide to interact with the chemical of interest either through adsorption, absorption, or a chemical reaction (Figure 1). The measuring device includes the glass waveguide (12 mm by 25 mm by 2 mm), polymeric coating (0.14 µm thick), a small commercial laser light source (12.7 mm diameter by 16.5 mm length),
and a data processing electronics circuit board (127 mm long by 27 mm wide). The sensor, laser, and processing electronics are packaged within a stainless steel casing, which is 35 mm in diameter and 100 mm long.

When the specific chemical of interest interacts with the polymer, the effective index of refraction is altered, and the velocity of an optical beam which is passed through the waveguide changes, and becomes either slower or faster. The optical beam produces electric fields which decay into the cover film; when the speed of this beam is altered, the electric field from the optical beam is altered, which creates a different decay into the cover film. The changed optical beam from the chemically-reactive channel is then combined with an unlabeled reference channel light beam which produces an interference pattern between the two beams (Figure 2). This pattern can then be correlated to the concentration of the chemical being measured. The sensor can be configured to measure both gaseous and aqueous phases; however, the sensor used in this study was limited to aqueous measurements of the BTEX chemicals. Additionally, the chemically-reactive coating can be changed in order to detect different, or multiple chemicals or to optimize the detection of specific contaminants. The original IO system was a large bench-top configuration using lasers, mirrors, and a table array for setup. During this study, Photonic Sensor Systems and GTSI miniaturized the layout into a compact size for incorporation into a cone penetrometer module.

3 LABORATORY TESTING PROGRAM

A preliminary laboratory testing program was undertaken in order to demonstrate the viability of the BTEX sensor in a soil environment. Initial experiments were performed in aqueous solutions and subsequent experiments were performed with the sensor in a soil/water matrix. The soil used in all testing was Ottawa 20/30 sand which is a relatively unreactive sand soil composed of essentially pure silica (Lambe and Whitman, 1969; Mitchell, 1993). Ottawa sand was chosen as the soil matrix because it is a well-characterized soil and provides relatively little chemical interference during the testing.

3.1 BTEX chemicals

The contaminating chemicals used in the experimental study include benzene, ethylbenzene, toluene, and xylene (BTEX chemicals). The BTEX chemicals belong to the group of aromatic hydrocarbons and are constituents of fuel products. Aromatic hydrocarbons are composed of a benzene ring with additional functional groups attached in the ortho-, meta-, and para-positions. The chemicals are classified as light non-aqueous phase lipids (LNAPLs) because they are less dense than water and are not miscible with water; however, they are slightly soluble. Because they are the most soluble components of gasoline, the BTEX chemicals are commonly used as diagnostics of gasoline spills, either from leaking underground storage tanks or from surficial spills. Sampling of the BTEX chemicals is difficult due to their volatility; consequently, in-place measurements of BTEX concentrations, which do not expose the chemicals to atmospheric oxygen are desirable.

3.2 Experimental apparatus and test procedures

All tests were performed in a laboratory setting using a triaxial calibration chamber (14 cm diameter; 61 cm height) constructed out of BTEX resistant materials: Pyrex glass, stainless steel, and Viton o-
Data acquisition was performed using a Pentium notebook computer which was used to monitor 8 channels from the waveguide. Figure 4 shows a picture of the miniaturized sensor supported by an external clamp on a laboratory bench-top. The processing electronics are to the left of the clamp, and the sensor, flow cell, and inflow and outflow tubing are to the right of the clamp. Note also that a glass waveguide is shown at the right mid-center.

4 EXPERIMENTAL RESULTS

Seven experiments were performed with the sensor in aqueous and/or saturated sand matrices. The experiments performed were initial proof-of-concept tests done to verify the proper operation of the miniaturized sensor in a soil matrix. The initial tests were successful; however, malfunctioning of the prototype sensor operation prevented further experimentation. Consequently, only the results of the initial testing program are presented.

The results from the seven experiments are shown in Figure 5 through Figure 11 in both aqueous and soil environments. Figure 5 shows the sensor response to benzene (chemically resistant coating polybutylisobutyrimethacrylate (PBI-BMA)) in de-ionized water with benzene added after nine minutes of operation at a stable baseline. The addition of benzene produced an immediate response which measured increasing concentration up to a concentration of 250 ppm. After four minutes of operation at a constant concentration, de-ionized water was added to the test cell in order to dilute the concentration of benzene, and the sensor measured a decrease in concentration. Figure 6 shows the results of a test performed with the sensor operating.

Figure 5. Sensor Response to Benzene in De-ionized Water

Figure 4 shows a picture of the miniaturized sensor supported by an external clamp on a laboratory bench-top. The processing electronics are to the left of the clamp, and the sensor, flow cell, and inflow and outflow tubing are to the right of the clamp. Note also that a glass waveguide is shown at the right mid-center.
in a soil/aqueous environment. This figure shows the continuation of a previous test where the concentration in the test cell was initially 190 ppm. Additional benzene was added in order to raise the concentration in the test cell to 380 ppm.

One notable difference in the response of the sensor in a soil matrix as opposed to a pure aqueous environment is the rate at which the concentration changes. The soil matrix, as would be expected, slows the movement of the contaminant to the inflow tubing of the sensor, producing slower changes in concentration than are seen in pure aqueous solutions. This behavior is consistent throughout the tests performed.

Figure 7 and Figure 8 show the response of a different chemically reactive coating, Teflon-A, to toluene. The Teflon-A was used as the reactive coating in these experiments because the response was much more definitive than the response seen with the PBIBMA coating. In fact, the sensor response time was so rapid that the data acquisition electronics could not record data fast enough to monitor the concentration change of benzene, the smallest molecule. Consequently, these tests were performed using toluene and xylene which are larger molecules and have slower response time due to mass transfer effects into the coating. The toluene behavior recorded again shows rapid response and concentration change in aqueous solutions while the response in the soil/aqueous matrix demonstrates a significantly slower response (Figure 7 and Figure 8).

The final set of experiments were performed using...
Figure 10. Response to Xylene in Soil Matrix (9.5 ppm)

Figure 11. Response to Xylene in Soil Matrix (7 ppm)

the Teflon-A reactive coating in both aqueous and soil environments. Xylene, which is a larger molecule than toluene, shows a slower response time than the smaller molecules, even in aqueous solution. Figure 9 shows the sensor's response to xylene (concentration = 6 ppm) in an aqueous solution and Figure 10 and Figure 11 show the sensor's response in a soil matrix (concentration = 9.5 ppm and 7 ppm). The same patterns are evident in the sensor response to xylene in a soil matrix as were seen with benzene and toluene: the soil matrix slows the transport and diffusion of the chemical through the soil and into contact with the sensor.

5 DISCUSSION

Two serious complications with the prototype sensor hardware prevented further experimental testing during this phase: the waveguide mount was not manufactured to a sufficient tolerance and broke the glass waveguide when it was tightened into place. Fracturing of the waveguide occurred on three separate occasions, which required approximately two months for repair in each instance. Additionally, there was an incompatibility of communication protocols between the waveguide hardware and the data acquisition electronics which prevented the rapid data transfer needed for measurement of some chemicals because the sensor development was performed independently of the electronics development. Ongoing work in Phase II is underway to re-design the communication systems in order to obtain sensor-electronics compatibility. Additionally, the electronics processing hardware is being developed under the Environmental Systems Management, Analysis, and Reporting Network (E-SMART) in order to provide a standard for connection of environmental monitoring sensors used by the U.S. Department of Defense.

While a complete evaluation of the sensor behavior in a variety of soil environments was not possible due to the above reasons, enough data were gathered to demonstrate that the sensor can measure ITEX chemical concentrations in an Ottawa sand soil matrix. A range of concentrations were measured, from a high of 380 ppm benzene to a low of 6 ppm xylene, which demonstrates the sensor versatility in terms of detection limits. Additionally, the only limits on the measurement response time were imposed by the data acquisition software and hardware which is currently being upgraded in order to improve processing time. Another key factor in the sensor performance was the question of reversibility. In repeated experiments, the sensor demonstrated that increases as well as decreases in chemical concentration could be measured.

Experimental work with the next version of the IO sensor is currently being planned in order to verify its field applicability, including a study of sensor response to a mixture of chemicals, the effect of soil grain size on sensor measurements, sensor response at extremes of pH, sensor response in high ionic strength environments, the interaction of naturally occurring soil acids with the sensor's measurement, the influence of sampling procedure including sampling and filtering materials and pump configurations, and the influence of dynamic penetration on the sensor readings.

6 CONCLUSIONS

The integrated optic sensor was miniaturized during this study and has demonstrated viability for the detection of the ITEX chemicals in soil.
environments under laboratory conditions. While the robustness of the device must be improved in order to withstand the rigors of cone penetration testing, the RTX sensor is clearly a promising tool in terms of providing quantitative measurement of chemical concentrations in-situ. Of particular interest to other applications is the fact that the waveguide and electronics packaging would be the same for sister sensors because only the polymeric coating is changed for detecting other chemistries.

7 ACKNOWLEDGEMENTS

The work performed in this study was supported in part by the U.S. Army Research Office, an NSF Graduate Traineeship, and an NSF grant under the direction of Priscilla P. Nelson, Program Director for Geomechanical, Geotechnical, and Geo-environmental Systems. This support is gratefully acknowledged.

8 REFERENCES


Reliability of soil gas sampling and characterization techniques

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ABSTRACT: Vadose zone soil gas sampling is an important aspect of many environmental geotechnical site characterization and remediation programs. Typical measurements include total hydrocarbons, oxygen, and carbon dioxide. Measurements can be made using a variety of meters, procedures and samplers, however, variations in equipment and procedures used can result in significant differences in measured values. This paper describes results from research conducted on soil gas measurements at several contaminated test sites. Critical sampling procedures that were investigated include direct measurement versus sample collection with subsequent measurement, meter limitations, purge volume, and short circuiting. The results presented show that variations in these procedures can sometimes result in significantly misleading results. Recommendations are given for minimizing these potential error sources and increasing the reliability of soil gas measurements.

1 INTRODUCTION

Soil gas sampling has proven to be a useful tool in environmental geotechnical site characterization and remediation programs. At hydrocarbon contaminated sites, vadose zone soil gas surveys are used to delineate the spatial extent of contaminated soil and groundwater and to monitor the effectiveness of remediation efforts. In these cases, the soil gas survey typically involves making depth specific measurements of the concentration of volatile organic compounds (VOC), oxygen, and carbon dioxide present in the soil gas. VOC concentrations are used to identify locations of separate phase petroleum products (Ostendorf et al. 1995) whereas oxygen and carbon dioxide concentrations are used to assess the level of aerobic activity occurring in the subsurface (Ostendorf and Kampell 1991).

Soil gas sampling can be performed using a variety of methods with the whole air active technique being one of the most common. This technique involves the forced movement of soil gas from the sampling horizon to a collection or measurement device (ASTM 1994). Whole air active samplers can be designed to serve as permanent installations or to be used as rapid deployment probes. They are generally considered to be reliable especially when compared to indirect methods such as headspace analysis (e.g., measurement of soil gas in the headspace of jar samples). However, even when using the whole air active method there is a variety of equipment and procedural issues that may affect the reliability of measured results. This paper discusses the reliability of soil gas sampling based on measurements from whole air active soil gas samplers conducted over several years at four contaminated sites located in New England. Two were the location of service stations with hydrocarbon contamination from leaking underground storage tanks, one was the location of jet fuel contamination and the fourth was the location of a project investigating the effect of the highway delicer calcium magnesium acetate on groundwater quality. The main soil unit at the sites consists of sand with traces of gravel, silt, and clay. The depth to ground water varied among the sites ranging from 1.8 m to 12 m below ground surface.

2 SOIL GAS METERS

Three battery-powered portable meters were used to measure the total hydrocarbons, oxygen, and carbon dioxide. The total hydrocarbons concentration was measured using a Bacharach TLV hydrocarbon meter calibrated with hexane. This meter pulls gas
samples using a built in 2 liter/minute pump into a catalytic combustion chamber for destructive sampling of the gas flow. Output is read using one of three analog scales in units of volumetric parts per million (vppm) with a maximum value of 10,000 vppm. Oxygen concentrations were measured using a Bacharach Model 302 meter equipped with a potentiometric cell and a built in 2 liter/minute pump. The meter is calibrated using ambient air (assumed to be 20.9% oxygen). Carbon dioxide measurements were made using a Riken-Keiki meter that is equipped with a flexible flow pump. Percent carbon dioxide is measured through infrared absorption with a range of 0 to 100%. Calibration of the instrument is performed using a cylinder of 100% carbon dioxide.

3 SOIL GAS SAMPLERS

A variety of soil gas samplers consisting of both permanent installations and rapid deployment probes were used at the sites. The main type of permanent installations consisted of clusters of stainless steel tubes with a sintered stainless steel filter element (Figure 1a). All permanent installations were constructed using a drill rig equipped with hollow stem augers. Rapid deployment probes consisted of a variety of designs including flush filter probes, recessed filter probes, and retractable tip probes. An example of these types of probes is given in Figure 1b that shows a schematic of a flush filter probe. The probes were typically advanced using a drill rig and contained an internal tube that was used to transport soil gas samples to the surface. Details on the design of all the soil gas samplers used are given in DeGroot et al. (1996) and Ostendorf et al. (1995).

4 RESULTS

4.1 Direct Read versus Tedlar Bag

Soil gas measurements can be made using one of two techniques: direct read and Tedlar bag. The direct read method consists of connecting a portable meter directly to the sampler tubing and using the meter’s internal pump to draw the soil gas sample into the meter. The Tedlar bag protocol (Robbins et al. 1990) uses a separate vacuum pump, regulator and flow meter system that is connected to the sampler tubing as shown in Figure 2. This system is used to draw a soil gas sample to the surface at a controlled rate into an evacuated 3 liter nitrogen-cleaned Tedlar bag. Once filled with a sample, the

![Figure 1. Soil gas samplers: (a) permanent point, (b) flush filter rapid deployment probe (not to scale).](image)

![Figure 2. Vacuum pump assembly and Tedlar bag.](image)
failure and/or erratic readings under low flow conditions due to low soil air permeability or the sampler clogging with in situ moisture. This problem arises from the fact that the vacuum pumps built into the meters are relatively weak and are designed for open air, ambient conditions. The Tedlar bag sampling system can largely overcome this problem since the sample is first drawn into the Tedlar bag using a separate vacuum pump. Because of this, and the need for conducting serial dilution and gas chromatographic analysis at some sites, the sampling protocol used at the test sites was gradually changed over to the Tedlar bag system. However, this system requires more connections (Figure 2) than the direct read method which can leak ambient air especially when the vacuum pressure is high. It is therefore important that the vacuum gage and flow meter be used together to monitor and adjust the sample stream rate. The flow meter allows the flow rate to be decreased when the vacuum pressure becomes too high so as to minimize the potential for a short circuit to occur. It is also important that all connections be checked frequently for leaks. Unfortunately, the method is more time consuming and the Tedlar bags need to be cleaned between each reading, however, these problems are considered to be a minor tradeoff considering its advantages as noted above.

4.2 Meter Limitations

The catalytic combustion TVL meter described in Section 2 relies on sufficient oxygen for complete combustion and therefore meter accuracy. As a result, VOC readings in zones of high hydrocarbon flow oxygen can be unreliable. This effect is shown in Figure 3 for data from stainless steel permanent clusters. Measurements of total hydrocarbons were made on each Tedlar bag sample using both the TVL meter and an HNU portable gas chromatograph (GC). The data in Figure 3 suggest that the ratio of GC to meter hydrocarbon reading increase with decreasing oxygen when the oxygen concentration is below approximately 5%. These results clearly show that portable catalytic combustion meters must be calibrated when used in high hydrocarbon flow oxygen environments otherwise hydrocarbon concentrations can be significantly underestimated.

4.3 Purge Volume Prior to Reading

Recommendations in the literature vary as to how soil gas sample systems should be purged prior to taking readings. Christy and Spruill (1992) suggest that readings be taken on the gas stream immediately upon sampling whereas Burris et al. (1988) suggest that several purge volumes must first be drawn before taking a reading in order to get a more standard sample concentration. Figures 4 and 5 present results regarding the influence of purge volume on readings from clusters of the stainless steel permanent samplers. Figure 4 plots data from samples that involved taking a reading on the first 1.5 liter draws from the sampler followed by a reading on a second 1.5 liter sample taken immediately after the first. The results show slightly higher oxygen readings and lower carbon dioxide readings for the first sample and mixed results for the VOC data. However, the differences between the first and second sample readings are not significant and suggest that purging large volumes of soil gas prior to taking readings is unnecessary.

Further evidence that purging large volumes is unnecessary is shown in Figure 5a that plots readings for eight 1.5 liter Tedlar bag samples taken in rapid succession. Other than a slight change in readings from the first to second sample, the readings remain approximately constant with continued purging. This was not the case for another sampler as shown in Figure 5b. However, the effect shown here, of increasing oxygen and decreasing carbon dioxide and VOC, was determined to be due to a short circuit leaking ambient air into the samples. The short circuit was found to be along the outside wall of the sample riser tube from the ground surface to the sampler port. The change in values from the second to third reading was due to a four hour time delay between these readings allowing in situ conditions to reestablish itself – further evidence of the presence of the short circuit. All other readings were taken in rapid succession.
The data in Figures 4 and 5 indicate that only a small volume of soil gas needs to be purged prior to taking readings. In fact, purging large volumes can cause a number of problems including increasing the likelihood for a short circuit to occur, dilution by creating a large zone of influence, and overlapping zones of influence when making depth varying measurements. Good practice should involve some purging of system dead gas prior to sampling, especially if small samples are being collected (e.g., for gas chromatograph analysis). Another important issue, particularly when making temporal comparisons of soil gas data, is to be consistent in the procedure used for each sampling trip. With these factors in mind and the type of results shown in Figures 4 and 5, the standard practice adopted at UMass for permanent samplers is to purge the first 1.5 liter sample and use the second 1.5 liter sample for readings. The 1.5 liter sample is approximately the minimum amount of gas needed for taking a set of three portable meter readings, i.e., O₂, CO₂ and VOC. The flow rate during purging and sampling is adjusted depending on the vacuum pressure being developed, i.e., the flow rate is decreased if the vacuum pressure increases significantly. This is done since higher vacuum pressures increase the likelihood of a short circuit (developing as noted in Section 4.1. Typical flow rates range from 0.5 to 1.5 liters/minute. Between each set of readings the Tedlar bags are cleaned with nitrogen and the meters are purged with ambient air.

4.4 Short Circuit

The potential for a short circuit to develop is a problem for both permanent and rapid deployment samplers. Figure 5b showed clear evidence of a short circuit problem with permanent samplers. This type of problem can be minimized with careful installation of the samplers including the use of bentonite seals to isolate the sampling zone and concrete surface plugs to keep the samplers fixed in place. However, even with these precautions problems can arise. Bentonite plugs must remain
hydrated otherwise significant desiccation cracking can occur. Concrete surface plugs tend to break down especially in areas that experience freeze-thaw cycles. Excessive handling of the sampler during each sampling trip can exacerbate these problems. Using quick-connect type connections rather than threaded or push on connections allow for easy connection of the sampling riser pipe to the instrument or vacuum pump system with a minimal amount of handling.

Short circuiting can also be a problem for rapid deployment probes. Evidence of this is shown in Figure 6 that presents VOC and oxygen data taken using stainless steel permanent clusters and a retracted tip drive probe. This drive probe has a retractable tip that protects the filter element during driving which is subsequently retracted a few centimeters at each sampling depth to expose the filter element (see DeGroote et al. 1996 for more design details). The probe readings for VOC plotted in Figure 6a are very low near the surface and again at a depth of approximately 2m clearly showing the diluting effect of ambient air being drawn from the surface along the drill string. The oxygen data in Figure 6b for the retractable tip probe are very high compared to the permanent samplers. Furthermore, most of these readings are almost equal to ambient conditions which is highly unlikely in this area that is known, from monitoring well observations, to have a significant amount of free floating petroleum product at a depth of about 2.5 to 3 meters. It was determined that these short circuits were produced because the drill rods were pulled on an angle during retraction of the drill string at each sampling depth. This breaks the contact between the drill string and native soil allowing ambient air to be drawn down the annulus and drawn into the sampler. Similar problems can occur with short friction reducers located just above the sample port because this often creates an open annulus between the native soil and drill string.

Being careful to ensure that the drill string always remains vertical during driving and retraction can minimize short circuit problems with drive probes. In fact, because of this difficulty, the retractable tip probe is not recommended for soil gas sampling. Flush filter type probes such as that shown in Figure 1b are preferred since the probe does not need to be pulled back at each sampling depth thus decreasing the likelihood of a short circuit developing. The flush filter probes can periodically clog in fine grained soils but rarely do so in coarse grained soils like those encountered at the sites studied in this research. Friction reducers should be relatively long and located some distance away from the filter element or, if possible, eliminated completely.

5 SUMMARY AND RECOMMENDATIONS

Many of the systematic errors associated with soil gas sampling can be minimized if careful attention is given to the procedures used during sampling. Based on the site conditions and work described in this paper, recommendations for improving the reliability of soil gas measurements include:

1. The Tedlar bag protocol is preferred over direct read measurements provided a flow meter and vacuum gage is used to monitor the flow stream.
and all connections are checked frequently.

2. Combustion type hydrocarbon meters should be calibrated using gas chromatograph data for high hydrocarbon/oxygen samples.

3. Standard sampling protocol should involve some purging prior to sampling but purging large volumes prior to readings does not appear to be necessary.

4. Careful attention to sampler construction, installation, and handling should help to minimize dilution problems arising from short circuits. Flush filter probes are preferred over retractable filter tip probes especially in coarse grained soils.

ACKNOWLEDGEMENTS

The work described herein was conducted as part of projects sponsored by the Department of Defense (DOD), Federal Highway Administration (FHWA) and the Massachusetts Highway Department (MHD). The Authors would like to acknowledge the work of their colleague, Dr. David Ostendorf, who was the lead Principal Investigator on these projects. The authors thank all the graduate students in their department for their assistance in the projects. The opinions and findings contained herein are those of the authors and do not necessarily reflect that of DOD, FHWA, and MHD. This paper does not constitute a standard, specification, or regulation.

REFERENCES


Remedial design for DNAPL recovery at a former wood preserving plant, Pensacola, Florida

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ABSTRACT: Operation of a wood preserving facility for nearly eighty years at Pensacola, Fl contaminated near surface groundwater with dense non-aqueous phase liquid (DNAPL). The major sources of aquifer contamination were two unlined ponds and a partially backfilled drainage channel which were in direct communication with the groundwater. A remedial design involving DNAPL recovery and recycling was developed for the site. The design was accomplished in four phases: phase 1 included the review of existing site data and development of a conceptual site model; phase 2 a remedial design investigation was planned and implemented; phase 3 involved development of a conceptual remedial design and performing a demonstration pilot test; and phase 4 included evaluating the pilot test results and completing the final remedial design.

1. BACKGROUND

The American Creosote Works site, located in Pensacola, covers approximately 18 acres and is owned by the bankrupt American Creosote Company of Escambia, County, Florida. The area surrounding the site is mixed commercial and residential, with Pensacola Bay and Bayou Chico located approximately 600 yards to the south (Figure 1).

Wood-preserving operations were carried out at the site from 1902 until December 1981. The operations consisted of pressure treating wood poles with creosote. The site contaminants primarily consisted of polycyclic aromatic hydrocarbons (PAHs), pentachlorophenol, monocyclic aromatic hydrocarbons, and dioxins and dioxinofurans.

Two main ponds, a rail road (RR) impoundment and an evaporation area were located in the western portion of the site. Wastes in the ponds were stabilized and capped during an emergency response action in 1983. The former process facility and drip truck area are located northeast of the main pond as shown in Figure 1.

The site is underlain by a freshwater aquifer, approximately 200 feet in thickness, comprised of sand with interbedded layers and lenses of clay and sandy clay. Remediation investigation studies have detected the presence of creosote type dense non-aqueous phase liquid (DNAPL) in the aquifer, as well as associated dissolved phase groundwater contamination.

2. CONCEPTUAL SITE MODEL

A Conceptual Site Model (CSM) was developed

Figure 1. Site location
to illustrate the relationship between contaminants, retardation/transport media, potential receptors, and possibility for exposure. Historical site photographs and data from previous investigations were compiled for an up-to-date data base on the site. These data were reviewed for identifying the potential waste source areas, and defining subsurface soil conditions and routes for contaminant migration. The CSM is presented in Figure 2.

Development of the CSM identified data gaps in the DNAPL delineation. Filling these gaps was required prior to designing the DNAPL recovery system and therefore were the focus of the phase 2 remedial design investigation.

3 REMEDIAL DESIGN INVESTIGATION

The purpose of the remedial design investigation was to provide additional data to support the remedial design process. This included investigating potential source areas for DNAPL contamination, determining the horizontal and vertical extent of DNAPL in the site area, and physical characteristics and extractability of the DNAPL.

Investigation activities included; fluid level measurements; trenched suspected source areas of DNAPL contamination; cone penetrometer testing (CPT) with soil borings; DNAPL characterization; and well installation and extraction testing.  

3.1 Fluid level measurements

Ninety groundwater monitoring wells were monitored for the presence or absence of DNAPL.

Fluid levels were measured using an electronic water level indicator. The indicator activated at contact with the water surface and remained activated while lowering through the water column. Where DNAPL was present, the indicator ceased operating when encountering the top of the DNAPL column. DNAPL was observed in four wells located at the southwest corner of the site. DNAPL thickness ranged from approximately 3 to 42 ft in wells at depths of 40, 60, and 80 ft below land surface (bgs). The observed thickness was not a true reflection of the DNAPL distribution in the subsurface as determined later in the investigation. Observed DNAPL thickness is most likely an accumulation of DNAPL from well development and previous groundwater sampling activities.

3.2 Trenching and test pits

Six trenches and test pits were excavated to confirm the presence or absence of DNAPL source areas. Two trenches were excavated in the northern drip track area, and one trench and three test pits excavated at a former RR impoundment. Trenches were excavated to a depth of 3 to 6 ft bgs and from 50 to 150 ft in length. Groundwater was encountered from 2 to 4 ft bgs. Test pits were excavated from 2 to 4 ft bgs and approximately 6 to 8 ft across. Results of excavations in the drip track area showed significant amounts of residual DNAPL in the near surface soils with some DNAPL seeping into the excavation nearest to the former process area. Results for the trench and test pits at the RR impoundment showed no evidence of DNAPL. This area was therefore eliminated as a source area for DNAPL.

3.3 Cone penetrometer testing

CPT technology was selected because it represents a cost-effective alternative to a soil boriing program. Benefits of the technology include rapid collection of continuous soil property and electrical conductivity data, while producing minimal investigation derived waste.

Twenty-four CPT’s were conducted to determine subsurface stratigraphy, identify potential zones saturated with DNAPL, measure permeability, and to collect soil and fluid samples from the subsurface. The CPT probe included the standard tip and sleeve resistance sensors, a pore pressure sensor, and an electrical conductivity sensor to detect low conductivity DNAPL zones.
CPT holes ranged in depth from 22 to 96 feet bgs. Verification soil borings were drilled at localities selected from field interpretation of the CPT results. Observations from the soil borings were used to correlate stratigraphic profiles and to confirm DNAPL distribution with the CPT response measurements. These data were combined to determine DNAPL presence and definition of DNAPL pooling mechanisms.

Results of the investigation indicate two modes of DNAPL emplacement. At the base of a upper sand, a zone of pooled DNAPL up to 6 to 7 ft thick is present below the former ponds and along the former drainage channel. CPT electrical conductivity measurements within this discrete pool of DNAPL exhibited no detectable conductivity. The second mode of emplacement was in a lower sand from 30 to 75 ft bgs and below the former ponds. DNAPL in the lower sand is present in thin (1 to 6 in) black layers that alternate with water saturated light colored layers. CPT electrical conductivity measurements within this zone show a decrease in conductivity. Selected conductivity profiles for the upper sand are provided in Figure 3.

The conductivity profile for CPT-14 shows a value of approximately 10 milli amperes (mAn) from 2 to 10 ft (water saturated sand), drops to between 0 to 1 mAn at 11 to 14 ft (DNAPL zone) and increases to greater than 25 mAn in an underlying clay layer. A fluid sample collected from 11.5 to 12.5 ft in CPT-14 recovered 750 ml of DNAPL with a minor amount of water. The profile for CPT-23, located 35 ft east of CPT-14, shows a higher, 5 to 7 mAn, reading in the corresponding DNAPL layer. A fluid sample collected from 13 to 14 ft in CPT-14 recovered water with strong creosote odor.

Results of CPT and soil boring program indicate DNAPL is present in medium to coarse-grained, well sorted sand layers up to 6-in thick. DNAPL has primarily pooled at the base of the upper sand along a low permeable contact (30 ft bgs) with the lower sand. The DNAPL zone in the upper sand has been observed up to 6 ft thick. A lesser amount of DNAPL has worked its way into the lower sand to an approximate depth of 75 ft bgs.

Conductivity measurements were near zero in 2 to 4 ft thick layers of pooled DNAPL in the upper sand. Conductivity measurements for the lower sand, where DNAPL layers are much thinner, were slightly less than the conductivity of adjacent water saturated sand. This is attributed to the thin

![Figure 3. Selected conductivity profiles for the upper sand.](image)

3.4 DNAPL characterization

Samples of DNAPL were collected from monitoring wells, Hydrocone® samples, and pump discharge during an extraction test. The samples were field-tested to determine specific gravity, temperature, viscosity, and water content. Selected samples were sent to a laboratory for chemical analyses.

Field tests for specific gravity ranged from 1.06 to 1.08 onsite and 1.11 dowgradient and offsite. Temperature ranged from 17 to 25 degrees Celsius, viscosity 4.4 to 14.3 centi stokes, and water content ranged from 0 to 42 percent, depending on the DNAPL extraction rate during a test. The increase in specific gravity of offsite DNAPL is thought to be a reflection of weathering. Degradation or volatilization of the lighter fraction constituents would result in a more viscous material.

Chemical analyses included volatile organic compounds, extractable organic compounds, and BTU value. Results showed volatile organic concentrations in the several hundred parts per million (ppm) range and PAHs in the 1,000 to 55,000 parts per million range. The BTU value was 17,000, similar to that of fuel oil.

3.5 DNAPL/water extraction test

A short term extraction test was performed to determine the extractability of DNAPL. The well used for the test was completed to a depth of 76.5 ft bgs and equipped 20 ft of 0.020-in slot stainless
steel well screen from 56 to 76 ft bgs. At this location, a 21-ft zone of sand interlayered with DNAPL was observed from 55 to 76 ft bgs. The test initially consisted of pumping groundwater for 8 hrs at an average rate of 700 mL/min (0.17 gpm) and observing the DNAPL thickness changes in response to lowering the well water level. Results showed DNAPL thickness increased nearly 20 ft; 13 gal of DNAPL was pulled into the well. Subsequently, approximately 55 gal of DNAPL was extracted from the well during a combined water/DNAPL extraction test. The DNAPL extraction rate was varied to allow for observation of fluid level responses and monitoring of the percentage of free water in the DNAPL. The general trend was that an increase in water content accompanied an increase in the DNAPL extraction rate. During DNAPL extraction, controlling a low percent of free water in DNAPL minimizes channeling or short circuiting the DNAPL zone with groundwater. This is desirable to maintain preferential DNAPL flow into the well and prolong the recovery of DNAPL. Figure 4 shows the extraction rates for both water and DNAPL and the percent free water observed during the short-term test.

4 CONCEPTUAL REMEDIAL DESIGN AND PILOT TEST

A conceptual remedial design was developed and a pilot test performed for DNAPL recovery and recycling. The conceptual remedial design included: installing recovery wells in both the upper and lower sand units; equipping recovery wells with a pneumatic pumping system; a collection system for conveyance of extracted fluids; a system for treatment and discharge of effected groundwater; and temporary storage and recycling of recovered DNAPL.

A month-long pilot test was performed to demonstrate extended DNAPL extraction is feasible. At the end of the test, two wells, one each completed in the upper and lower sand, recovered approximately 7,400 L (2,000 gal) of DNAPL. DNAPL recovery rates ranged from 4 to 6 L/hour (1.0 to 1.6 gph) for the upper sand and 3 to 5 L/hour (0.8 to 1.3 gph) for the lower sand.

4.1 Recovery wells

Three extraction test wells were installed based on the interpretation of DNAPL distribution from the remedial design investigation. All wells were constructed of 4-in. diameter stainless or galvanized steel, with 5 to 25 ft of 0.020-in continuous slot wire-wrapped well screen. Well TEX-01 was completed in the lower sand at a depth of 76.5 ft bgs with 20 ft of well screen from 56 to 76.5 ft bgs. Well TEX-02 was completed at the base of the upper sand with a 5-ft screen from 27 to 32 ft bgs, and well TEX-03 was completed in both the upper and lower sands with 25 ft of screen from 26.5 to 51.5 ft.

Due to a high water and low DNAPL recovery, well TEX-03 test was terminated within 6 days. Apparently, a combination of screen length, high percentage of water-saturated sand, and low percentage of DNAPL-saturated sand resulted in excessive water and extremely low DNAPL recovery.

4.2 DNAPL/water extraction

The pneumatic dual pumping system for extracting DNAPL and water during the test included an upper water level depression pump and a lower DNAPL extraction pump. The upper pump was placed below the top of the water level to maintain a water level drawdown in the well. This provided a decrease in hydraulic head resulting in preferential DNAPL movement or upconing into the well. The lower pump was placed at the bottom of the well to extract the accumulating DNAPL. Optimum performance was achieved by operating both pumps simultaneously while maintaining a fairly constant DNAPL thickness and DNAPL/water extraction ratio for the well. Figure 5 shows the general extraction well setup.

The upper pump was a controllerless, bottom filling, air displacement pump with an approximate hydraulic flow rate of 12.3 L/min (3.25 gpm). The
to 7 ft drawdown below static water level was maintained in the recovery wells to promote DNAPL movement into the well.

The lower pump was a bottom filling bladder pump equipped with a control box to regulate the DNAPL extraction rate. The lower pump was capable of pumping up to 150 mL/min (0.04 gpm). Regulating the extraction rate for DNAPL maintained a fairly constant column of DNAPL in the well. Figures 6 and 7 display the third level data for wells TEx-01 and TEx-02.

4.3 Extraction performance

Both wells demonstrated sustained DNAPL recovery for the month-long test. Upper sand DNAPL extraction was approximately twice the rate as the lower sand. This apparently is a function of screen length and a greater pooling of DNAPL at the test permeate contact between the
upper and lower sand. Well TEX-02, completed with a 5-ft screen set at the base of the upper sand, is open to a 4 to 6 ft zone of sand with DNAPL. The lower sand well, TEX-01, was completed with a 20-ft screen where approximately 30 percent of the screened interval consists of sand with DNAPL. The water recovered from TEX-01 was approximately three times that of TEX-02.

Average daily recovery rates for lower sand well TEX-02 were 83 L/day (22 gpd) DNAPL and 608 L/day (164 gpd) water. The DNAPL to water ratio was 1.7. Cumulative fluid recovery was approximately 2,700 L (730 gal) DNAPL and 19,500 L (5,270 gal) water. DNAPL and groundwater recovery data for TEX-01 is displayed in Figure 8.

Average daily recovery rates for upper sand well TEX-02 were 148 L/day (40 gpd) DNAPL and 23 L/day (6.2 gpd) water. The DNAPL to water ratio was 5:1. Cumulative fluid recovery was approximately 4,700 L (1270 gal) DNAPL and 731 L (198 gal) water. DNAPL and groundwater recovery data for TEX-02 is displayed in Figure 9.

5 FINAL REMEDIAL DESIGN

Results of the pilot test were evaluated and a final remedial design for DNAPL recovery and recycling was developed. The final design includes 6 recovery wells to be completed in the upper sand and 5 wells to be completed in the lower sand. Recovered DNAPL will be sent to a fuel blending facility for recycling. Recovered groundwater will be treated onsite and infiltrated into the subsurface.
Grouting on retraction of cone penetrometer

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ABSTRACT: Research was undertaken on the selection and use of grouts and grouting equipment in sealing geotechnical investigation holes created during penetrometer-based site investigations. Portland cement-based grouts with or without environmentally compatible retarders were found to be the most useful materials. A progressive cavity pump with a controller and a direct motor drive pumping through a 9.5-mm grout injection tube performed satisfactorily. Using the proper materials and equipment it is possible to consistently produce a secure, durable seal under a test site.

1 INTRODUCTION

Cone penetrometers equipped with chemical sensors are becoming a preferred method of investigating subsurface distribution of contaminants in soil (Robitaille 1994). Many of the contaminants present in the soil at a site are liquid phases that are either non-aqueous liquid phases or contaminants dissolved in the groundwater. A drill hole or a push hole created in the investigation of the site can potentially allow the movement of contaminants deeper into the soil along the path of the hole. Cone penetrometers equipped with sensors have the advantage of collecting data in near real-time and there is no need to leave an open hole when the rod and sensors are withdrawn. If the penetrometer rod is equipped with a grout injection system with ports at the end of the rod, the penetrometer can seal the hole when the rod with its sensors is withdrawn to the surface. The approach allows the hole to be sealed and does not allow any significant amount of contamination to migrate at the site. If properly executed with suitable material, the penetrometer hole is never an open conduit for movement of contamination. The use of through-the-rod penetrometer grouting becomes significant when it is noted that the location and tracking of contaminants on a waste site may require dozens of closely-spaced holes.

The purpose of this paper is to summarize the results of testing and evaluation of materials and methods that have been found useful in through-the-rod penetrometer grouting. Developing standard materials and procedures can assure more rapid acceptance of the penetrometer by regulators and officials involved in approval of site investigation programs.

2 GROUT SELECTION

The grouts used with the penetrometer must:
1. Have a viscosity that is sufficiently low to allow it to be pumped through approximately 100 m of tubing that may be as small as 9.5-mm diam.
2. Maintain a low viscosity for several hours while the site investigation is underway.
3. Seal the hole so that the overall permeability of the penetrated soil layers are not increased.
4. Maintain the seal even when the grout is subjected to drying.
5. Leach no components or reaction products into the soil that will significantly alter the water chemistry or confuse subsequent groundwater monitoring efforts.

The intensive analytical screening efforts that are used on groundwater samples where contamination is suspected almost eliminates the use of chemical grouts, that are based on cationic polymers. The very sensitive techniques employed in the detection and quantification of organic compounds in groundwater may be easily confused by the solvents and unreacted monomers that are present in organic-based grouts, such as those based on polyurethane or urea-formaldehyde. Additionally some of the organic grouts have significant drying shrinkage that may adversely affect their ability to produce a durable seal.
with surrounding soil under varying moisture conditions.

Particulate grouts or cement-based grouts have the greatest potential for producing a good sealing material due to their dimensional stability and the general lack of objectionable components. A program was undertaken to evaluate each of the candidate grout types. Data was developed on the viscosity at standard water-to-cement ratios, consistency of time-to-initial set (using retarders as needed), permanence, and rate of strength gain (Bean et al. 1995).

Viscosity was measured using the flow cone test described in ASTM C 939 (ASTM 1988b). Experience with a prototype injection system demonstrated that all grouts with flow times below 13 sec could be used successfully. Consistency of time-to-setting was measured using ASTM C 191 (ASTM 1988c). Experience with field operations showed that grouts that could be consistently retarded (no stiffening) for two hours could be considered acceptable. Permanence was judged from a literature search on the solubility and durability of the reaction products from each grout. Presence of soluble components produced a low ranking. The rate of strength gain was determined from the determination of unconfined compressive strengths at different times after placement using the procedure outlined in ASTM C 39 (ASTM 1988a) and from data in the literature. Obtaining strength equal to that of a typical soil (approx. 1.2 MPa) within two days was judged acceptable.

The results of the evaluation of grout properties are presented in Table 1. The portland cement-based grout was considered the best overall material especially when used with an environmentally compatible retarder such as sucrose. Pozzolana-based grouts were also considered a useful alternative, but set slowly. Portland cement-based grouts were carried into field testing (Lee, Malone and Robitaille 1997).

### Table 1. Rating of grout types of potential use in penetrometer grouting.

<table>
<thead>
<tr>
<th>Grout Type</th>
<th>Viscosity</th>
<th>Consistency of Time of Setting</th>
<th>Permanence</th>
<th>Rate of Strength Gain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gypsum</td>
<td>Acceptable</td>
<td>Variable</td>
<td>Low</td>
<td>Acceptable</td>
</tr>
<tr>
<td>Portland cement</td>
<td>Acceptable</td>
<td>Consistent</td>
<td>Acceptable</td>
<td>Acceptable</td>
</tr>
<tr>
<td>Pozzolana</td>
<td>Acceptable</td>
<td>Consistent</td>
<td>Acceptable</td>
<td>Low</td>
</tr>
</tbody>
</table>

3 GROUT INJECTION SYSTEM

The grout injection system is currently employed uses a small progressive cavity pump, with a directdrive, high torque electric motor (Fig. 1). The injection tube consists of 100 m of fluorocarbon-lined nylon tubing with an inner diameter of 9.5 mm. The tube is attached to a port at the end of the penetrometer probe. A nonremovable, expendable metal tip plugs the port. When the penetrometer is raised and the grout pump is started, the pressure of the grout forces the plug out of the port. The expendable plug drops to the bottom of the hole and grout is pumped in to fill the cavity as the rod is pulled upward. The penetrometer rods are typically furnished in 1-m lengths. After each length is lifted, the grouting is halted while the top 1-m length is removed from the string. If a 44-mm diam rod is used, 1.5 liters of grout must be pumped to fill the cavity left by one length of rod. If the grout infiltrates the surrounding soil, more grout may be used.

4 RESULTS OF THE FIELD TRIALS

Field trials run with portland cement-based grouts pumped through the penetrometer rod have shown that it is possible to obtain continuous, durable grout seals in penetrometer holes. Minor squeezing of the hole may occur, but the grout placement is generally fast enough to allow access to the hole. In field trials in clayey soil, grout columns that were recovered averaged 41 mm in holes that were formed with a 44-mm diam rod. Cores taken over grouted holes showed a continuous column of grout over the depths investigated.
CONCLUSIONS

This work demonstrated the following:
1. A portland cement-based (or pozzolan-based) grout can be used in a through-the-rod penetrometer grouting system.
2. A small-capacity progressive cavity pump with proper controls that permit intermittent operation can function as an injection pump.
3. With proper care, consistent, continuous grouting of the penetrometer holes can be accomplished.
4. Using grouting techniques discussed here, it should be possible to seal penetrometer holes so that they do not become conduits for movement of contaminants.

REFERENCES


Tomographic site characterization using CPT, ERT and GPR

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ABSTRACT: The integration of cone penetrometer logs (CPT) with electrical resistivity tomography (ERT) and ground penetrating radar (GPR) tomography increase the ability to characterize subsurface site conditions. Cross-hole ERT and GPR can compliment each other in regions of low resistivity where ERT is more effective and regions of high resistivity where GPR is more effective. The three techniques, CPT, ERT and GPR, are briefly described and results are presented for an infiltration test to demonstrate imaging a salt water phreone using pre-injection and post-injection tomograms. The test site consists of inter-bedded sand and clay layers that are imaged by both ERT and GPR. CPT resistivity and soil stratigraphy logs correlate with the ERT and GPR results.

1 INTRODUCTION

The US Department of Energy (DOE) is responsible for the clean up of inactive DOE sites and for bringing DOE sites and facilities into compliance with Federal, State and local laws and regulations. Significant savings, in both time and money, can be realized with better site characterization and monitoring techniques. These techniques are required to better characterize the physical, hydrogeological, and chemical properties of the subsurface while minimizing and optimizing the use of boreholes and monitoring wells. Today the cone penetrometer technique (CPT) is demonstrating the value of a minimally invasive deployment system for site characterization.

CPT uses a variety of sensors for measuring soil properties, such as, pore pressure, resistivity, temperature, pH, and seismic wave speed. Studies have shown that CPT site investigations at hazardous waste sites are a very cost-effective method for accessing the subsurface without drilling. Two new sensor packages for site characterization and monitoring have been developed for deployment with CPT. The two new CPT methods are:

• Electrical Resistivity Tomography (ERT) and

• Ground Penetrating Radar (GPR) Tomography.

Surface ERT and GPR have proven to be useful techniques for imaging subsurface structures and processes; however, depth of investigation is limited. Borehole use of ERT and GPR require the installation of system components via drilled boreholes. The installation of ERT electrodes and GPR antennas by cone penetrometer techniques reduces installation costs and total costs for ERT and GPR surveys. Using the cone penetrometer for environmental site characterization represents a new application of the technology. Significant advantages of the CPT include: eliminating drilling wastes and the need for treatment and disposal of drill spoils as hazardous material; reducing the possibility of cross contamination (by gouging the hole as the probe is withdrawn), and is faster than conventional drilling and sampling.

Technologies used for site characterization and monitoring have numerous and diverse applications within site clean-up and waste management operations. There is a need for sensors, sensor...
deployment means, and sensor data processing, including sensor data fusion methodologies for:

- Detection and monitoring of contaminants in soils, groundwater, and process effluents;
- Expediting site characterization; and
- Geological and hydrogeological characterization and monitoring of the subsurface environment.

Specific benefits are numerous where cost effective underground imaging is very important:

1. Delineating the continuity of soil layers between penetrometer holes;
2. Locating and mapping sand and clay lenses between penetrometer holes;
3. Mapping DNAPL plumes;
4. Defining spatial and temporal behavior of a steam flood for dynamic stripping;
5. Detecting leaks under tanks at Hanford, WA;
6. Monitoring the efficiency of air sparging;
7. Monitoring an ohmic heating thermal front;
8. Characterization of burial trenches and pits, including boundaries and contents; and
9. In situ measurement of physical properties, i.e., porosity, density and moisture content.

2. TECHNOLOGY DESCRIPTIONS

2.1 Cone penetrometer technique

The cone penetrometer was originally developed in the Netherlands in 1934 for geotechnical site investigations. The original cones involved mechanical measurements of the penetration resistance on a conical tip. A friction sleeve was added in 1965 (Begemann 1965). Electronic measurements were added in 1948 and improved in 1971 (de Reister 1971). Pore pressure probes were introduced in 1975 (Torstensson 1975; Wissa, et al. 1975), originally as independent probes, but were soon added to the cone penetrometer instrumentation. Modern CPT systems feature geotechnical sensors for tip stress, sleeve friction, pore pressure along with an inclinometer to measure the tilt of the probe, and resistivity and soil moisture sensors. This type of cone is primarily used for geotechnical investigations.

However, the significant advantages CPT provides for environmental work is leading to much wider acceptance by the environmental site characterization community. This is due largely to the development of new sensors that allow detection of chemical pollutants in situ.

Major components of the modern cone penetrometer system are the instrumented probe, the instrumentation conditioning and recording system, the hydraulic push system, and the vehicle on which the system is mounted. Enclosure in a van body allows all weather operation. The common configuration provides the reaction mass for a hydraulic push force of about 20 tons (18,000 kgf). Standardization for the geotechnical applications of the cone penetration test was established by the American Society of Testing and Materials in 1986. This standard allows for a probe diameter of 1.44 or 1.75 inches (3.658 cm or 4.445 cm). The most common for standard work is the 1.44-inch probe.

Recent environmental work, however, has led to the requirement to push deeper than possible with the 20 ton configuration. This has been accomplished by increasing the reaction weight to 30-35 tons (27,000 -32,000 kgf) and using the larger 1.75-inch probe and rod. This increases the rod buckling resistance at the higher loads. The maximum depth of penetration possible varies greatly with soil type. In soft damp soil, the 20-ton systems have penetrated 300 feet (91.5 m); but in gravelly soils, such as the Department of Energy's Hanford Site in southwestern Washington, these systems met refusal at 10-20 feet (3-6 m). A thirty-ton system using the larger diameter rods has reached depths of approximately 150 feet (46 m) in these same gravelly soils (Timian 1992).

2.2 Electrical resistivity tomography

In most environmental restoration applications the role of electrical resistivity is to assist in characterizing a site. The task includes not only specifying the location of contamination, but also mapping the physical and chemical properties of the ground that control their distribution and movement. In the most general sense, mapping electrical resistivity is important for conditioning or constraining the hydrological models of contaminant transport and retention. These models are usually based on drill-hole tests and suffer from the problem of extrapolation of point measurements to the volume between the holes.

For example, a channel of high permeability sand that is missed by a drill pattern illustrates the
problem of relying solely on drill holes. This channel would be the dominant feature of the site in terms of contaminant transport. Mapping the subsurface distribution of electrical resistivity could reveal the subsurface geometry and drastically change the hydrologic model.

Soil and rock resistivity (or conductivity) measurements have been used in the mining industry for many years, and recently have been used to locate contamination plumes. The electrical resistivity of most soils and rocks depends on the conduction paths afforded by fluids in the pore spaces. The porosity, saturation, pore fluid salinity, and clay content determine resistivity. Because the dissolved solids in groundwater influence resistivity, mapping it may be the only direct detection method for high concentrations of contaminants that form ionic species.

ARA includes a Resistivity Module in its cone penetrometer instrumentation for measuring resistivity in the adjacent soil. As part of the CPT push rod, the module consists of four circular electrodes in contact with the soil. Insulators separate the electrodes. The outer two electrodes are used to induce an electrical current into the soil matrix. The inner two electrodes are used to measure the strength of the induced electric field. The amount of voltage potential drop in the electric field is a function of the resistivity of the soil.

Daily et al. (1992) and Ramirez et al. (1993) at the Lawrence Livermore National Laboratory, developed and tested the Electrical Resistivity Tomography (ERT) method for mapping subsurface conditions between boreholes. Applications included monitoring water movement in the vadose zone and monitoring an underground steam injection process for soil decontamination. ERT uses a dipole-dipole measurement technique, similar to those used in conventional surface resistivity surveys (Keller and Frischknecht 1966) to measure the bulk electrical resistivity distribution in the soil mass between two boreholes.

Processes such as steam injection can be monitored by taking measurements before the process is started and then repeating the measurements over time as the process proceeds. Each tomographic data set is then subtracted from the original background measurements to produce a "time lapse" image set of resistivity variations between the boreholes.

To image the resistivity distribution between two boreholes, several electrodes are placed in each hole, as shown in Figure 1. Each electrode must be in contact with the formation. A known current, I, drives two electrodes, and the resulting voltage difference, V and I, is measured between other electrode pairs. This process is repeated until all the linearly independent combinations are measured. Each voltage-to-current ratio is a transfer resistance.

The goal is to calculate the distribution of resistivity in the vicinity of the boreholes given the measured transfer resistance.

The ERT image creation process involves solving both the forward and inverse problems. A finite element mesh, N elements wide (between the boreholes) and M elements long (along the boreholes) model the image reconstruction plane. Image resolution is a complicated function of many factors, including reconstruction pixel size, data signal-to-noise ratio, electrode and borehole separation, the subsurface resistivity distribution, and the degree to which the resistivity matches the two-dimensional model of the forward calculations. Resolution can be no better than one pixel; typical pixel size is 1 to 3 meters. The best resolution is obtained close to the electrodes, and the worst resolution is obtained along a vertical stripe midway between the boreholes. Thus, resolution improves as borehole spacing decreases.

2.3 Ground penetrating radar

Ground penetrating radar (GPR) has been used for
over twenty years (Merey 1972; Daniels, et al. 1988) at chemical and nuclear waste disposal sites as a non-invasive technique for site characterization (Olofson 1992; Sandler, et al. 1992). Standard GPR surveys are conducted from the surface of the ground providing geotechnical information from the surface to depths of 5 to 50 feet, depending on GPR frequency of operation and soil conductivity. Commercially available GPR systems operate over the frequency range 50 MHz to 1000 MHz. The lower frequencies provide better penetration but poor resolution, while the higher frequencies give poor penetration but good resolution. There are many critical environmental monitoring situations where surface GPR does not provide the depth of penetration or necessary resolution. Borehole radar (Bradley and Wright 1987) can place the sensor closer to the region of interest, overcoming the high signal attenuation in the near-surface soils.

Figure 2 is a schematic diagram showing possible data collection approaches for GPR measurements. These transmission measurements include hole-to-hole and hole-to-surface measurements. For cross-hole tomography (CPT), one CPT antenna is held stationary while the other unit is moved. The process is repeated until the volume between the holes is covered. As the radar pulse propagates, it is attenuated due to conductivity and slowed due to the dielectric constant. Therefore, GPR tomography maps variations in conductivity and velocity from which it is possible to estimate soil characteristics, such as water content, density and contamination.

For GPRT data, a tomographic reconstruction is attempted using first arrival times in a SIRT (simultaneous iteration reconstruction tomography) algorithm, initially with straight ray paths. However, if difficulty is experienced with convergence, then a perturbation method is used which allows for curved ray paths. The region under investigation is divided into a regular grid (similar to ERT) and the radar wavelet velocity and attenuation are iteratively calculated for each cell and combined to generate a map of the region between the holes. Spatial resolution is governed by the dominant wavelength of the pulses in the medium; at 100 MHz resolution is on the order of 0.5 to 1.5 meters.

3 RESULTS

A test program was initiated at ARA’s Vermont Test Site, near South Royalton, VT. Figure 3 shows the layout of GeoWells on the corners of a square with the infiltration well near the center of the square. CPT logs to a depth of 70 feet at each location delineate the soil strata/layer, resistivity and pore pressure. Relatively thick layers of sand (10 to 15 feet) separated by thinner layers of clay and silty-clay (1 to 2 feet) characterize the site. The GeoWells are composed of sections of 2-
diameter PVC pipe with 10 ERT electrodes spaced 7' apart to a depth of 60 feet. About 100 gallons of saline water was injected in the 15-foot deep infiltration well. Both ERT and GPR cross-hole tomography data were taken from the GeoWells before and after the water injection.

Figure 4 is an example of an ERT tomogram between GeoWells 1 and 3 with the corresponding CPT resistivity log (jagged curve). Superimposed on the resistivity log are resistivity segments or layers calculated from the resistivity log data. The contour values in the ERT image are the log of resistivity in ohmmeters. Note the low resistivity at 38 feet and 50 feet in both the CPT log and the ERT image. Low resistivity occurs in the clay layers. ERT data above 20 feet was not usable because the soil was too resistive for adequate current injection.

An effective way of monitoring an ongoing process is to make a series of measurements over time and then display only the changes that occur. Figure 5 is an example of an ERT image showing the difference between pre-injection and post-injection results. GeoWell 1 is common to the two panels (see Figure 3). The resistivity ratio is calculated by dividing the post-injection by the pre-injection data. The greatest changes (dark) occur above the clay layers. The water plume seems to be moving along the top of the clay layers.

Figure 6 is an example of GPR tomographic images between GeoWells 1 and 4 showing the results of pre-injection and post-injection data.

An example of GPR tomographic results are shown in Figure 6 for pre-injection and post-injection measurements.

The dark area in the right panel is the water plume extending from the upper right to the lower left. Post-injection measurements were made one day after the salt water was injected.
4 CONCLUSIONS

CPT methods can be used to install PV-electrode GeoWells to a depth of 60-feet. The same GeoWells can be used to make both ERT and GPR cross-hole measurements. ERT and GPR tomographic images will show the migration of a salt-water plume in interbedded sand and clay formations.

5 REFERENCES


SCAPS characterization of VOC contaminated sites

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ABSTRACT: The Tri-Service Site Characterization and Analysis Penetrometer System, or SCAPS, was developed to address many of the current problems in site characterization. SCAPS combines traditional cone penetrometer technology with contaminant sensors and samplers to quickly and inexpensively provide a profile of contaminants and geophysical properties at hazardous waste sites.

SCAPS is a research, development, and technology demonstration tri-service coordinated program, with the U.S. Army Environmental Center as the lead. In 1987, the development of the SCAPS program began at the U.S. Army Corps of Engineers Waterways Experiment Station, under the sponsorship of the Army Environmental Center, which initiated an effort to enhance traditional cone penetrometer probes with sensors for characterizing subsurface contamination. In 1989, it was designated a Tri-Service program, with Naval Research and Development representing the U.S. Navy and Armstrong Labs representing the U.S. Air Force. The Tri-Service coordination ensures the elimination of any duplication of effort and builds on the expertise of each organization.

The SCAPS consists of a hydraulically operated cone penetrometer test unit mounted in a custom-engineered 20-ton truck with on-board computers that provide real-time acquisition/processing of sensor data. The truck is capable of pushing instrumented cones that provide subsurface soil stratigraphy and groundwater depth. The SCAPS has also been designed to accommodate samplers for use in collecting data on specific classes of subsurface contaminants.

The DOD users have identified four areas of contamination concern: petroleum, oil, and lubricants (POL); volatile organic compounds (VOCs); heavy metals; and explosives contamination. Focusing on these users' needs, SCAPS has sensors in various stages of development.

The first sensor developed through this program was the Laser-Induced Fluorescence sensor, or LIF (figure 1), which was the first to pursue regulatory acceptance. The LIF has been licensed to the private sector and is used worldwide. This sensor detects petroleum, oil, and lubricants contamination in the subsurface. Using a near-ultraviolet laser to induce fluorescence in POL contamination, the probe carries the fluorescence energy to the surface through a fiber optic cable for real-time spectroscopic analysis.

The volatile organic compound (VOC) sensors/samplers are the next in line behind the LIF and have begun the process of seeking regulatory acceptance. The Thermal Desorption Sampler and the Hydrosponge Sensor detect VOCs in the soil and groundwater, respectively.

The Thermal Desorption Sampler probe (figure 2) is pushed by the SCAPS truck to a predetermined depth. The central actuator rod is retracted. Then, the probe is pushed further into the soil, collecting a 5 gram soil plug in the sample chamber. The soil is heated, releasing the VOCs. These VOCs are carried to the surface by helium gas. This analyte stream can be directly interfaced with an Ion Trap Mass Spectrometer (ITMS) on-board the SCAPS truck. The VOC contaminants are analyzed in real-time, without sample preparation. This process can be repeated at multiple depths during a pass.

The technique of thermal desorption is primarily used in soil; therefore, an additional sensor is necessary to detect VOCs in groundwater. The Hydrosponge Sensor (figure 3) utilizes a
commercially available Hydropunch direct push well to access the groundwater. The Hydropunch is pushed to the desired depth and the push pipes are retracted, exposing the Hydropunch screen to the
groundwater. The groundwater enters and comes to equilibrium, which generally takes 15 to 20 minutes.

Then, the in situ sparge module is lowered into the well. The in situ sparge module is operated at the groundwater surface, but samples the water about 18 inches below the surface. The sparge module purges the VOC analytes in situ from the groundwater using Helium gas. The analytes are then transferred to the on-board ITMS where the contaminants are analyzed in real-time.

Combined with the VOC sensors/samplers, the ITMS provides instantaneous analysis for the VOCs without requiring any sample preparation. The ITMS, using the EPA method under review as method 8265, is capable of detecting most VOCs in the sub-ppt range. The Field Portable Ion Trap Mass Spectrometer produces quantitative and qualitative data, obtained from the mass spectra.

The Thermal Desorption Sampler and the Hydrosparge Sensor are currently in the process of seeking regulatory acceptance. The Environmental Security Technology Certification Program (ESTCP) has established a program to accelerate acceptance and application of innovative monitoring and site characterization technologies that improve the way the nation manages its environmental problems.

Through ESTCP, the SCAPS program is able to perform technology demonstrations/validations for these two sensors. These demonstrations provide an opportunity to compare traditional site characterization methods with data obtained from the Hydrosparge Sensor and the Thermal Desorption Sampler.

A major obstacle to implementation of innovative site characterization techniques is the acceptance of new technologies by both federal and state regulatory agencies. Currently, the SCAPS program is pursuing regulatory acceptance in the California Certification program and reciprocity at the state level through the Interstate Technology Regulatory Cooperation for the two VOC sensor/samplers. The final effort, in the pursuit of regulatory acceptance, will be the drafting of a Standard Practice under the auspices of the American Society for Testing and Materials (ASTM).

The California Hazardous Waste Environmental Technology Certification Program (Cal/Cert AB 2060 program) is an innovative environmental technology certification program through the California EPA. The Interstate Technology and Regulatory Cooperation (ITRC) Working Group is a federal and state advisory committee primarily funded by DOE/EPA to facilitate state reciprocity of innovative environmental technologies.

The Cal/Cert AB 2060 certification program is intended to evaluate the effectiveness and reliability of environmental technologies in the hazardous waste field. This certification can be used to support marketing of the environmental technology, domestically or abroad.

The ITRC is a national coalition of state environmental regulatory agencies working cooperatively with federal agencies and other stakeholders to improve the acceptance and interstate deployment of innovative environmental technologies.

The purpose of the ITRC is to enable the formation of national/regional partnerships and interstate verification standards and mechanisms that promote: the sharing of information on innovative technologies that will encourage regulators and regulated entities to increase deployment, alternatives and competition to environmental solutions; and cost-effective...
solutions to improve environmental protection, site restoration, and resource conservation.

Regulatory acceptance would enable the SCAPS program to gain support in the transition of these two sensors/samplers, as well as others, to the user community. Combining the sensors/samplers with the cone penetrometer platform to provide rapid, on-site, in situ, subsurface detection, SCAPS reduces the time and cost required for site characterization.

Future Tri-Service research/development and demonstration efforts include SCAPS deployed sensors to detect explosives and heavy metals contamination. The SCAPS explosives sensor (figure 4) detects explosives contamination by heating soil in situ to generate nitric oxides, which are then detected using an electrochemical sensor inside the probe.

There are two sensors being developed in the SCAPS program to detect heavy metals, the XRF and the LIBS. The XRF, or x-ray fluorescence sensor (figure 5), uses an x-ray source which causes metals to emit unique fluorescence x-rays, which are then analyzed on the surface. The XRF can operate in the saturated and unsaturated zones.

The other heavy metals sensor, the LIBS, or laser-induced breakdown spectroscopy (figure 6), quantifies metals in soil by creating a laser-induced plasma in the unsaturated zone. Emissions from the plasma are then carried to the surface for analysis.

The SCAPS program brings proven methodologies for site characterization into the field by developing proofs of principle, demonstrating and validating the technologies in the field, gaining regulatory acceptance, and transitioning the technologies to the commercial sector.
Determining contaminant distribution and migration by integrating data from multiple cone penetrometer-based tools

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ABSTRACT: The cone penetrometer has been used for geologic characterization at the US Department of Energy-owned Savannah River Site (SRS) for the past 7 years. In addition to the standard suite of sensors (i.e., tip pressure, sleeve friction, capillary pressure), the cone penetrometer has been used with innovative sensors to perform contaminated site assessments and has also been used to install wells. By integrating geologic information from the standard cone penetrometer sensor with the depth-discrete chemical information obtained from innovative cone penetrometer-based samplers and sensors, an accurate rapid, and cost-effective characterization can be accomplished. Using the added capability of cone penetrometer-installed wells, the placement of targeted remediation systems can be initiated during the characterization. Cone penetrometer tests (CPT) provide high resolution, high quality data, are minimally invasive, and produce a minimum of investigation derived waste. These attributes are critical to cleanup operations at large hazardous waste sites with heterogeneous sediments.

At SRS, nearly 4 million pounds of chlorinated organic solvents were released to the subsurface. Because this waste was released primarily as a dense nonaqueous phase liquid (DNAPL), the contaminant migration has been controlled by the saturation and morphology of the confining units in the vadose zone and the topology of the units below the water table. Standard CPT sensors were used to precisely determine the location and extent of these confining units and innovative sensors (e.g., fiber optic spectroscopy, electrical resistivity, soil moisture probes) and samplers (e.g., ConeSipper®) were used to obtain a targeted evaluation of the location, magnitude and potential pathway of the contamination. Conventional borehole logs and well sample analyses were also evaluated to provide additional perspectives. By using the combined data from the standard and new CPT sensors as well as conventional data, a more accurate conceptual model of the subsurface contamination at the site was achieved.

1 TOOLS ON THE CONE PENETROMETER

The standard suite of sensors on the cone penetrometer includes tip resistance, sleeve friction, and pore pressure. This ensemble is commonly called a piezo-cone configuration. Electrical resistance measurement capability has recently been included as a standard tool in several cone penetrometers. This measurement has a long history as a standard borehole logging tool but has only lately been commonly implemented with the cone penetrometer. The data from these sensors have not been routinely integrated with the soil type algorithms to provide refined determination of subsurface geology. Other tools that have been implemented with the cone penetrometer at SRS include:

- seismic cone (geophones in the cone penetrometer tip)
- inclinometers
- vibratory cone (to determine liquefaction potential)
- laser induced fluorescence (primarily for poly aromatic hydrocarbon detection)
- non-laser induced fluorescence (same as previous)
- Raman spectroscopy (for identification of separate phase liquid contaminants)
- gamma probe
- fiber optic TCE sensor (TCE in the vapor phase)
- various in-cone chromatographic systems (for contaminant analysis in the liquid or gas phase)
- samplers (soil, liquid, and gas)
- POLO (subsurface position locating system)
- soil moisture sensors

and others. In this text we will describe two site characterization activities on the Savannah River Site that have benefited from the integration of data from multiple cone penetrometer tools.

2. M AREA BASIN PLUMB

The M area basin was designed to receive waste from metal processing operations in the A/M area that included dissolved metals (nickel, aluminum, uranium, lead), acids, caustics, salts, and solvents (Looney et al., 1992). The aqueous and nonaqueous waste stream was carried through a process sewer line (approximately 1 km-long) to the unlined basin (8 million gallon capacity). Contaminants were released as a result of leaks in the sewer line and from the unlined basin. Approximately 500,000 lbs of acids, caustics, and salts, and 140,000 lbs of solvents consisting primarily of tetrachloroethylene (PCE) and/or trichloroethylene (TCE) were released to the basin per year (Jackson et al., 1996). The wastes were normally released in batches, probably producing slugs of dense fluids in the process lines. Approximately 260 million gallons per year of process water were also released, but as a continuous stream. A total of approximately 2 million pounds of solvents were released to the M area basin. More than 300 monitoring wells have been installed in the A/M area since groundwater monitoring began in 1979. DNAPLs (95% PCB) have been identified in only two wells immediately adjacent to the M area basin, although high aqueous-phase concentrations of organics (suggestive of DNAPL presence) have been detected in several other wells in the area. Groundwater gradients for the water table aquifer in this area produce predominantly, vertically downward flow with a significantly smaller lateral component to the west-south-west. Aqueous phase concentrations of TCE and PCE have been detected in the water table aquifer and the semi-confined aquifer below it over a large area (approximately 2 miles) around the M area basin and other release points in the A/M area (Looney et al., 1992).

Because of the large amount of DNAPL released we recognized that finding and remediating this long term source offers the most expedient and cost-effective strategy for clean up. To date there is no single, consistently successful method for detecting DNAPL in the subsurface but through an integration of several complementary data sets, more accurate characterization is possible.

In addition to standard characterization methods such as conventional drilling and soil and water
sampling techniques, the cone penetrometer was used to map the top of a known subsurface confining unit called the green clay. The exercise consisted of identifying the green clay unit using the friction ratio and electrical resistivity logs from 25 cone penetrometer pushes combined with existing core information from drilled wells. The combined data sets resulted in a map of the undulating top of the green clay which provides a gravity driven conduit for the movement of DNAPL (Looney et al., 1992). In Figure 1, it is evident that there is a trough feeding from the M area basin to the west and north-west. The presence of this potential DNAPL pathway explains unusually high concentrations of chlorinated organics found in well samples along this trough up to one half mile from the basin. These highly contaminated wells are flanked by wells outside of the trough whose concentrations are more than three orders of magnitude lower.

In another study, cone penetrometer resistivity data were combined with standard well resistivity logs to evaluate the impact of disposal, and to trace the historic pathway of inorganic waste migration in comparison with chlorinated organic migration pathways.

A review of the borehole and cone penetrometer logs for the A/M Area conducted by Nelson and Kibler (1996) of the USGS provides evidence for postulated DNAPL behavior in the subsurface. Specifically, the data confirm that DNAPL transport is dependent on the topology of subsurface confining units and gravitational forces not necessarily coinciding with the direction of groundwater flow in the region. Two separate contaminant pathways are evident near the M area settling basin corresponding to the movement of DNAPL and to dissolved phase transport.

In their report, Nelson and Kibler reviewed electrical resistivity borehole and cone penetrometer (CPT) logs, and gamma-ray logs acquired in the A/M area. The electrical resistivity borehole logs were acquired using both short (16-inch) and long (64-inch) tools and were corrected for borehole fluid and borehole diameter effects. Gamma-ray logs were smoothed to reduce statistical noise. The cone penetrometer electrical resistivity logs did not require correction.

Nelson noticed a trend of anomalous negative resistivity gradients (increased conductivity with depth) in several wells and CPT pushes corresponding to specific areas around the M basin and sewer line. The wells and CPT pushes exhibiting the trend are located along the process sewer line and generally south and southwest of the M area basin. They postulated that the increased conductivity around the basin could be due to the historical release of conductive fluids (acids, caustics, and salts) in the process waste stream. This theory is reinforced by the observation that the more conductive wells tend to be located near waste release points while the wells without a negative resistivity gradient do not.

From the disposal history, groundwater flow data, historical well analyses for VOCs, CPT confining surface data, and the electrical resistivity data, a plausible conceptual model of the fate and transport of organic contaminants released to the M area basin can be developed. Organic and inorganic waste were released as slugs from operations areas to outfalls, and to the M area basin through the process sewer line. The organic slugs were immiscible with water and proceeded down through the vadose zone where they were trapped in pores of fine grained materials by capillary forces, or suspended on the surface of water-saturated fine grained zones (possibly collecting in local depressions on these surfaces). When the capacity of these materials was exceeded, the DNAPL moved downward towards the water table. The DNAPL continued to move down through the saturated zone until encountering fine grain zones (confining surfaces) then began moving laterally on the surface driven by gravity and directed by the topology of the fine grained units. Water contacting the DNAPL dissolved a portion of the organics and advanced in the direction of the hydraulic gradient but the majority of the DNAPL remained intact and moved by gravity. One DNAPL pathway resulting from the M area basin is evidently on a west-northwest path not corresponding to the groundwater flow direction.

The inorganic slugs, although miscible, were also more dense than uncontaminated water and initially followed a similar path to the DNAPL. The dense aqueous phase liquid (DAPL) also moved downward through the vadose zone and into the water table and then, probably, along the surface contours of the confining unit traveled by the DNAPL. The DAPL slug, however, dissolves...
at a much faster rate than the DNAPL and could more easily move into the semi-confined aquifer so its movement was soon controlled by the hydraulic gradients in the direction of flow to the south.

In figure 2, the conductive wells are denoted by solid symbols and are located slightly to the west but predominately to the south of their source release points. This is consistent with the theory of initial gravity controlled movement followed by flow with the natural groundwater gradient. The arrows indicate the direction of suspected DNAPL movement to the west of the source. This clearly shows that DNAPL movement is not controlled by groundwater flow in this area.

To test the conceptual model for DNAPL movement around the M area basin, a strategy was needed for rapidly detecting DNAPL in the subsurface. One of the methods tested was Raman spectroscopy (inelastic scattering of light by interaction with solids, liquids, or gas) through a CPT. A probe was developed by ARA Inc. and BIC Inc. (Bratton et al., 1994) that could perform remote Raman spectroscopy through an optical window in the CPT tip via fiber optic links to the surface source and detector optical and electronic components. CPT pushes were made immediately adjacent to two wells from which DNAPL samples were collected. Spectra produced from this activity yielded no useful Raman spectra because of an interfering fluorescence signal produced by elastic scattering of the source light. Fluorescence is a base of Raman spectroscopists because it can overwhelm and mask the inherently much weaker Raman scattered light. An argon ion laser source (wavelength ~ 514 nm) was used in this field test because it produced the best signal to noise response in laboratory tests with SRS soil. Since this field test, new lasers have been developed at infrared wavelengths which should significantly reduce the confounding fluorescence signal.

An important result of this work was the observation of the fluorescent signal as a function of depth. These data indicated a significantly higher (three times the background signal) fluorescence signal at the precise depth that DNAPL is known to occur in this location (Bratton et al., 1994). Subsequent optical and mass spectral analysis of DNAPL samples from the nearby wells identified co-constituents of the PCN and TCE including long chain hydrocarbons and alkyl esters.
which may explain the fluorescence at the relatively high excitation wavelength from the argon laser. The absorption spectra for pure PCE and the collected DNAPL sample at various concentrations are shown in Figure 3. This figure illustrates the relatively low absorption wavelength range of pure PCE (250-280 nm) in comparison with the higher wavelength range of SRS DNAPL (250-330 nm). The wavelength difference is significant because commercially available fiber optics will substantially attenuate light of wavelengths below 300 nm. The low absorption wavelength of pure PCE (and subsequently low emission wavelength) has discouraged most researchers from attempting to use fluorescence to locate DNAPL. With the presence of fluorescent co-constituents, fluorescence for DNAPL may be a viable technique.

3 321 M SOLVENT STORAGE TANK AREA
This site is within the A/M area and is therefore part of the larger plume from the historical release of process solvents and fluids from the various outfalls. In addition, this site was contaminated by a 1200 gallon leak of PCE from one of the solvent storage tanks in 1985. It is this recent spill that is currently being characterized.

Several cone penetrometer pushers were conducted at this site to support characterization associated with the installation of a vacuum extraction well and catalytic oxidation remediation system. Nearby core logs from hollow stem auger drilling operations indicated the presence of a shallow, yet substantial, low permeability zone in the area. Because of the importance of low permeability sediments in determining the fate and transport of dense non-aqueous phase liquids, a program to carefully characterize the vadose zone around the solvent storage tank was begun. Standard piezocone measurements were made to characterize the geologic features with high resolution. Continuous soil gas measurements were made using the Cone Sipper® gas/liquid sampler and a Bruel & Kjaer Model 1302 multi gas analyzer. Gas concentrations approaching the vapor saturation limit were detected near the clayey zones in the shallow subsurface which suggests the presence of non-aqueous phase liquids. Soil samples were taken with the cone penetrometer using the MOSTAP sampling system and split spoon. Concentrations of PCE in some of these samples exceeded 1000 mg/kg, confirming the presence of DNAPL in these zones. Because DNAPL can serve as a groundwater contamination source for many years, it is imperative to locate this source and remove it to effectively clean up a site.

![Figure 3 Absorption spectra of PCE and SRS DNAPL.](image-url)
The movement of DNAPL in the subsurface can be quite complicated in the vadose zone and may be determined by different forces than aqueous phase contamination. Specifically, the morphology of geologic units, and capillary forces will dominate the transport of DNAPLs. It is therefore critical to map the topology of confining surfaces. In the vadose zone, a surface will constrain the flow of DNAPL, depending primarily on its pore size and moisture content. With the addition of a volumetric soil moisture probe to the standard piezo-cone data were taken at the 321 M solvent storage tank area that provided an indication of potential confining surfaces that would control the movement of DNAPL in the vadose zone. Figure 4 depicts the plots of sleeve to tip friction ratio and volumetric soil moisture with respect to depth.

From this plot it is clear that the clayey zones at 9 feet, 15-20 feet and 23-30 feet will constrain the transport of organic contaminants when the zones are saturated with water. The soil data show DNAPL-likely PCE concentrations (1400 mg/kg at 8 feet, 1100 mg/kg at 12 feet, 1200 mg/kg at 22 feet) on top of these clay zones, indicating that the water-saturated, fine grain sediments are excluding occupation of the pore space by the organic contaminants. Therefore, DNAPL would migrate on top of these clays controlled by gravity and the topologic structure of the clay. Relatively high concentrations of PCE were also found within the clay ranging from 150 mg/kg to 460 mg/kg. We can explain the fact that DNAPL-like concentrations were found below confining structures and within the clayey units by noting that the soil moisture content of sediments in the vadose zone vary and PCE may have migrated through the clay during a period of lower soil moisture content in the clay thereby allowing the inhibition of contaminant into the pore space.

CONCLUSIONS

Many innovative tools are currently available for deployment with a cone penetrometer. These tools use several sensing methods and address the interrogation of both the inherent subsurface media (sediments, groundwater, etc.) and introduced materials (contaminants). In addition to enabling the use of an array of subsurface tools, the cone penetrometer has the added advantages of providing rapid access to precise depths in the subsurface; being minimally invasive; minimizing the production of investigation derived waste; and being relatively inexpensive when compared with standard access technologies. The high resolution data available from cone penetrometer sensors greatly enhances the conceptual model of a site characterization.

At SRS, data from several different cone penetrometer sensors has enabled us to focus remediation efforts and additional characterizations to target contamination hot spots and likely-source zones. As with all subsurface sensing methods, data from individual CPT sensors are not unconditionally correct in their detection of contaminants or determination of subsurface properties but when combined with other complementary data sets, a much more
accurate picture of the subsurface contaminant fate and transport is obtained.

REFERENCES


Brine plume mapping using cone penetrometer and geophysical methods

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ABSTRACT: Naturally occurring brines (concentrated solutions of chloride salt and water) are pumped from deep wells and are transmitted by pipeline to industrial processing facilities. An accidental brine release affected shallow groundwater quality at a site in Michigan. The chloride brine plume was delineated using non-intrusive surface geophysical, minimally invasive cone penetrometer, and downhole geophysical methods. Surface electromagnetic surveys were used to map the areal extent of the plume. Direct push cone penetrometer soundings with soil electrical conductivity measurements were used to profile stratigraphy and detect electrical conductivity anomalies associated with brine intrusions. Anomalous zones were sampled, using a geophysical groundwater sampling, for direct chemical analysis. Geophysical monitoring stations were then installed in boreholes to allow periodic monitoring of the plume's response to remediation.

1 INTRODUCTION

Soil electrical conductivity (EC) measurements are uniquely useful during geo-environmental exploration of brine contaminated groundwater. Both surface and downhole methods can be used to monitor increases in EC resulting from the migration and diffusion of brine through groundwater. Surface methods have the utility of rapidly covering large areas to identify where brine intrusions might have occurred. Downhole methods are very useful in pinpointing the vertical extent of contamination as well as in estimation of brine concentrations. Downhole measurements can be obtained in cased boreholes, or through use of cone penetrometer soundings which include EC measurements.

Soil electrical conductivity is controlled by the conductance of the soil particles and the conductance of the fluid occupying the soil pore spaces. The ratio between pore fluid and combined soil-pore fluid electrical conductivity is termed the formation factor (Archie, 1942). Clay particles can be electrically conductive due to adsorbed water and ionic electrical charges on the clay platelets, so clay electrical conductance depends on mineralogy, porosity and pore fluid characteristics. Sand grains are typically nonconductive, so sand conductance depends primarily on pore fluid characteristics and porosity. The addition of brine to groundwater greatly increases soil electrical conductance.

A brine groundwater contamination exploration method was developed based on several technologies. First, surface electromagnetic (EM) surveys can be used to rapidly map affected areas, and give reasonably good indications of the degree of impact of brine releases on groundwater, to depths as great as 60 m (200 ft). However, surface EM methods lack the ability to adequately quantify brine concentrations, cannot provide detailed definition of the depth intervals affected by brine intrusions, and cannot easily differentiate between natural electrically conductive clayey soils and zones of elevated chlorides.

Downhole geophysical induction logging can be performed to great depths in suitably cased boreholes. Data quality is high, allows vertical delineation of affected intervals, and allows estimation of brine concentrations. However, drilling and casing boreholes, and the disposal of large volumes of exploration derived wastes from drilling activities, are expensive and time consuming. Further, borehole stability can be a significant problem where thick sequences of saturated sands are encountered.

A high capacity, truck mounted geo-environmental cone penetrometer system,
including downhole EC logging and penetrometer groundwater sampling, can be used to vertically profile subsurface conditions and obtain direct groundwater samples. The penetrometer exploration method is fast and accurate, and provides high resolution lithologic and electrical conductivity data. The direct push penetrometer exploration method requires no borehole, so no soil cuttings are generated, and borehole stability is of no concern.

2 GEOPHYSICAL METHODS

The electromagnetic (EM) geophysical method determines electrical properties of earth materials by inducing electromagnetic currents in the ground and measuring the secondary magnetic field produced by these currents. An alternating current is generated in a wire loop or coil above the ground's surface. Both the primary magnetic field (produced by the transmitter coil in the instrument) and the secondary field (produced by currents in the earth) induce a corresponding alternating current in the receiver coil of the instrument. The coils are kept at a fixed distance and orientation relative to the ground to simplify data analysis.

After compensating for the primary field, both the magnitude and relative phase (in-phase and quadrature) of the secondary field are measured. The quadrature-phase component, using simplifying assumptions of homogeneous and isotropic conditions, is converted to a value of apparent soil electrical conductivity (EC). This value represents an estimate of the local average soil EC. The depth of measurement is dependent on the instrument's coil spacing, orientation, and operating frequency, and the actual subsurface EC variations. Averaging limits discrimination of thin, high concentration brine intrusions from broader, more diffuse plumes. Multiple profiles using differing coil spacing can be performed to bracket approximate depths of brine affected groundwater. Data quality may be degraded by cultural interference as caused by utility lines, steel fences or other large metallic objects.

Surface electromagnetic measurements were taken by Geoscope with Geonics EM38, EM31 and EM34 systems, using coil separations of 1 m (EM38), 3.7 m (EM31), and 10 m or 20 m (EM34). The nominal explored depth is proportional to coil spacing. For the shallow, near field EM38 instrument, the explored depth is about 1.5 m; for the intermediate EM31 it is about 6 m; and for the deep, far field EM34 instrument, the explored depth is about 15 m with a 10 m coil spacing, 30 m with a 20 m spacing, and 60 m with a 40 m spacing.

Instruments were carried manually, and measurements were digitally logged during profiling at the chloride brine release site. Shallow and intermediate readings in the source area were typically taken at 15 m (50 ft) intervals along lines 15 m apart using the EM38 and EM31 instruments. EM34 (deep far field) coverage was conducted on lines spaced 60 m (200 ft) apart in the western portion of the plume, where the chloride brine had migrated to much greater depths.

Permanent geophysical monitoring stations were installed using hollow stem auger drilling techniques. A water filled, PVC casing was grouted into place to provide an access conduit for lowering downhole geophysical logging tools. Screened intervals are not necessary (or even desired) as there is no need for direct communication with the surrounding groundwater/soil system. In induced logging, the field is induced in the surrounding soil through the casing itself. Electrically conductive (i.e., steel) casing materials cannot be used.

Periodic downhole logging of the geophysical monitoring stations was performed at the chloride brine release site using a Geonics EM39 induction system. The induction technique has been used for decades by the petroleum industry for formation characterization in oil and gas wells. The Geoscope application of these methods to shallow groundwater exploration includes increased resolution due to slimmer, compact logging tools (1.6 x 130 cm or 1 x 50 in) and slower logging rates. This permits the detection and resolution of formation conductivity changes across vertical intervals of less than 0.3 m (1 ft). The EM39 induction logger is capable of measuring with an accuracy of 2 mS/m (milli-Siemens/m) in the range of 0 to 1000 mS/m.

The geophysical monitoring station provides significant advantages over traditional screened monitoring wells at brine affected sites. A continuous EC profile is obtained from the surface to the bottom of the cased hole, where traditional monitoring wells only provide data across the screened interval. This is significant as brine plumes typically sink with time, retreating from screened intervals and eventually emerging at the surface. The geophysical monitoring station avoids the costs of installing monitoring well clusters, and eliminates data gaps which occur where monitoring wells are screened in different portions of the aquifer. Monitor well purging and analytical testing are also eliminated, leading to additional cost savings.

Like surface EM and penetrometer CPT/LC methods, the borehole EM39 system measures the
bulk (soil and pore fluid) EC. With suitable assumptions of formation factors, a value of groundwater conductivity, and thus approximate concentration of a predominant ion such as chloride, can be estimated from downhole EC measurements.

3 CONE PENETROMETER SYSTEM

The STRATIGRAPHICS cone penetrometer system is used during geo-environmental and geotechnical exploration programs in difficult soil conditions. The heavy (240 kN and 300 kN, or 24 and 30 ton) truck mounted rigs are fully self-contained, and include data acquisition systems, dry and wet work areas, water tanks, steam cleaners, decontamination and grouting systems, separate rodstrings for sounding and sampling, optional dynamic rod driving, and heavy duty downhole equipment for use in glaciated terrains (Figure 1).

Cone penetrometer (direct push) systems require no borehole to advance probes and samplers and result in very little exploration derived waste. Downhole equipment is decontaminated during retrieval using an automatic rodwasher, and the open hole is pressure grouted. Most exploration activities are performed inside an enclosed portion of the rig, providing all-weather capability and a low visual profile during operations. Truck mounted penetrometer systems can be very productive, with as much as 400 m (1300 ft) of stratigraphic logging per day, with depth capacity exceeding 60 m (200 ft). As many as 18 groundwater, or up to 30 soil or soil gas samples, can be acquired in a day.

Soil resistance to penetration acting on the tip and soil friction on the sides of the penetrometer are separately measured during CPT. These measurements are accurate and repeatable, and have been used for the evaluation of stratigraphy and geotechnical parameters for decades. Performance of geo-environmental exploration has improved with ASTM Standards D5778, D6057 and Grade D6001.

The CPT tip resistance increases exponentially with soil grain size. Tip resistance in a sand aquifer is typically one to two orders of magnitude greater than in a clay aquitard. For example, the CPT tip resistance varies from about 10 to 40 MPa (100 to 400 tons per square ft (psi)) in a dense sand aquifer, while tip resistance in a stiff clay aquitard ranges from about 0.5 to 1.5 MPa (5 to 15 psf). The friction ratio (proportion of friction to tip resistance) is also used as an indicator of soil type. The friction ratio allows discrimination between loose sands and hard silts and clays, where tip resistances can be similar. The friction ratio ranges from about 1% in a sand to greater than about 3% in a clay. Silts have intermediate friction ratios.

High resolution, continuous soil profiling (sounding) for geo-environmental exploration is most often performed by STRATIGRAPHICS using the Piezometric Cone Penetration Test with soil Electrical Conductivity measurements (CPTU-EC). The CPT tip and friction resistance measurements (CPTU-EC) are evaluated for soil types and geotechnical parameters (Douglass and Olsen, 1981). The piezometric measurement (CPTU-EC) allows evaluation of soil saturation, hydraulic conductivity, potentiometric surfaces, and soil types (Saines et al., 1989, Robertson et al., 1986). The soil Electrical Conductivity measurement (CPTU-EC) provides information on soil moisture in vadose zone soils and indications of groundwater quality in saturated soils. The EC measurement has proved very useful in exploration for inorganic (metal, brines and landfill leachate) contamination and somewhat useful in non-aqueous phase liquid (NAPL) exploration.

Soil EC is measured by STRATIGRAPHICS using a rugged two electrode array mounted on the tip of the penetrometer (Stratinsky, et al., 1991). A 3 kHz AC voltage is applied to the array to control polarization and contact resistance effects. EC is computed based on in-plane currents induced across the array and a reference resistor. The EC...
measurement has a resolution of about 1.3 cm.

The STRATIGRAPHICS Penetrometer Sampler can be deployed in groundwater, soil or soil gas sampling modes by using interchangeable components. The groundwater sampler is a heavy wall, shielded wellpoint sampler. The shield prevents cross contamination and clogging of the sampler screen. The shield is retracted to allow groundwater to flow through a 0.5 m (20 inch) long screen into the barrel. Sample can be decanted from the barrel or can be pumped to the surface using an inertial pump. A small diameter pressure transducer can be lowered into the sampler to log the rate of groundwater inflow. Inflow results can be analyzed to estimate soil hydraulic conductivity.

Penetrometer exploration can be used at sites where predominant soil grain sizes are less than about \( \frac{1}{4} \) the diameter of the downhole tools, i.e. less than about 3-4 cm (medium to coarse gravel). A small fraction of larger particles can be tolerated, as long as the coarser particles are within a matrix of much finer soil grains, as is common in many glacial till units. Deep penetration can be limited by excess friction on the rod string where soft, squeezing clays are encountered. Rod friction can also be a problem where thick sequences of hardpan silts or very fine, saturated sands are encountered. Thick sequences of very soft soils, such as peat layers, can limit deep penetration as little lateral support is provided to the slender penetrometer rod string in the weak layer, while attempting to push through dense soils at depth. Extremely dense (SPT blowcounts greater than about 100) can be difficult to penetrate. Dynamic rod driving may help in this situation.

SITE CHARACTERIZATION

Chloride brine production (40% chloride salt solution) from well fields in Michigan is transported to a processing facility through pipelines buried at shallow depths. At one site, galvanic corrosion at a welded pipe joint led to a release of about 40,000 gallons of chloride brine. The initial response removed over 100,000 gallons of surface water and brine, but a portion of the released brine percolated into a thick sand aquifer, and some seeped into a nearby creek. Chloride brine migration was primarily controlled by differences in density between the heavy brine and lighter fresh groundwater, and by the generally westward regional groundwater flow.

Soils in the release area consist of about 180 m (600 ft) of sands, silts and clays, which overlies shale bedrock. These soils represent a sequence of lakebed and shoreline deposits in a paleo environment where water levels in the Great Lakes were much higher than at present. The sand aquifer is probably the result of dune and beach activity. The aquifer ranges in thickness from about 6 m (20 ft) near the brine release point, to over 40 m (130 ft) at a distance 600 m (2000 ft) west of the release point. An aquitard consisting of interlayered clays, silts, and silty sands underlies the sand aquifer.

The suspected release point and surrounding area were initially profiled to depths of about 6 m using near field EM38 and intermediate EM31 surface geophysical instruments. Within a week, the shallow plume was mapped to an area of about 35 acres. Purge and monitoring wells were installed to control the migration of the chloride brine through the aquifer. Additional geophysical profiling, and installation of more purge and monitoring wells, was performed in several phases over a period of years, as it became apparent that westward plume migration was continuing.

Due to the dense chloride brine sinking to the top of the westward dipping aquitard layer and the westward regional flow of groundwater, a far field EM34 surface geophysical instrument had to be used for profiling the central portion of the plume. At a distance of about 600 m (2000 ft) west of the release point, the plume had migrated deeper than the layer resolving capabilities of the EM34 instrument. Further plume characterization was performed with the CPTU-EC cone penetrometer method.

A series of 33 CPTU-EC soundings, to depths as great as 50 m (165 ft) and totaling 1061 m (3481 ft) of data, and 27 penetrometer groundwater samples, were completed to characterize the deep, westward extension of the plume. A thin zone of groundwater with slightly elevated chloride concentrations was eventually found to extend along the top of the confining aquitard unit to as far as 1200 m (4000 ft) west of the initial release area.

The indirect CPTU-EC data indicated that the sand aquifer was relatively homogeneous, with very few, thin, apparently discontinuous clay or silt seams or layers. In contrast, the aquitard was found to be much more heterogeneous than the aquifer, with interlayers of clays, silts, and silty sands. EC data (both CPTU-EC and geophysical) indicated groundwater unaffected by the chloride brine is low in electrical conductivity, which provides a very clear contrast to groundwater with elevated chloride content. Piezometric testing during CPTU-EC indicated aquifer hydraulic conductivities to range from about 1E-3 to over 1E-2 cm/sec. Aquitard
hydraulic conductivities ranged from about 1E-4 to less than 1E-8 cm/sec, depending on whether the testing was in silty sand or silty clay layers. The indirect piezometer evaluations of stratigraphy, hydraulic conductivity and EC agreed very well with indirect geophysical, direct sampled borehole, and pump test data collected at the spill site. The CPTU-EC data were evaluated to delineate the extent of the sand aquifer and clay aquitard units. Depth intervals within aquitard soils, where peak EC measurements were encountered, were selected for direct piezometer groundwater sampling. Sample targeting using the continuous CPTU-EC sounding logs significantly reduced the number of samples and analytical testing required to adequately characterize groundwater chloride concentrations. Near the plume source, where CPTU-EC measurements were greater than about 400 mS/m (4nS/cm), direct piezometer groundwater sampling was typically not performed, as high chloride concentrations were not in question for these conditions.

A comparison between CPTU-EC piezometer data and a drilled, sampled and geophysically logged monitoring station is shown in Figure 2. The two exploration points were within about a 9 m (30 ft) distance of each other, with the CPTU-EC sounding slightly closer to the plume axis. Very good correspondence is observed between the soil types evaluated from the CPTU-EC measurements and visual classification of obtained samples. The continuous CPTU-EC logs provide data across unsampled intervals in the discontinuously sampled borehole, and greatly increased resolution of this layering.

Comparison between the CPTU-EC and downhole EM39 data also shows very good correspondence. The CPTU-EC data provide higher resolution (2-3 cm versus 15-30 cm for EM39) of layering. In many instances, this can be advantageous (for example, during other projects when trying to detect thin LNAPL or DNAPL layers). A disadvantage to the high CPTU-EC resolution is that in gravelly soils where particle size is comparable to EC sensor resolution, a significant amount of noise can be exhibited in the EC data.

The CPTU-EC piezometric data in the example sounding log indicate that the bottom of the aquifer unit is slightly less permeable than at shallow depths. This fact, not apparent from visual descriptions of drilled samples, may explain the somewhat unusual shapes of the EM39 geophysical and CPTU-EC logs, and thus the distribution of chloride concentrations at the bottom of the aquifer.

The peak value of EC (and thus, highest chloride brine concentration) might be expected to fall at the interface between the sand aquifer and the underlying aquitard at 25.5 m, if the aquifer was homogeneous in hydraulic conductivity. Instead, the peak EM39 and CPTU-EC values were measured at a depth of about 21 m. This is the depth at which aquifer hydraulic conductivities slightly decrease, as indicated by CPTU-EC piezometric measurements. EC measurements slowly decrease with depth from this point, and somewhat abruptly decrease at about 23 m. The CPTU-EC piezometric data indicate a thin (<10 cm) silt seam at this depth, not detected during drilled sampling. A very abrupt decrease in EC finally occurs at the aquifer/aquitard interface, at 25.5 m.

A comparison between peak EC data measured in the CPTU-EC soundings and average EC data from surface geophysical surveys is presented in Figure 3. Generally good correspondence can be seen between the two exploration methods. As expected, the CPTU-EC peak data are much higher than the average data from the surface EM surveys. The deep, low concentration chloride affected groundwater in the western portion of the site was only detected using
CPTU-EC data. The surface EM geophysical survey was of great benefit where phone depths did not exceed about 20-30 m.

Correlations were developed from comparing CPTU-EC and downhole EM39 data to laboratory chloride concentrations obtained from groundwater samples in order to allow estimation of chloride concentrations. For the CPTU-EC data, and for chloride concentrations less than about 1000 ppm (parts per million), the following relationship was derived: chloride concentration, in ppm = (0.8 * EC (in uS/cm))-171. For chloride concentrations up to 50,000 ppm, an alternate relationship was developed: chloride concentration, in ppm = (1.9 * EC (in uS/cm))-1029. For a regulatory maximum chloride concentration in groundwater of 250 ppm, the corresponding CPTU-EC value had to be less than about 500 to 550 uS/cm.

It is important to note that these relationships were developed at a site where background groundwater was very low in dissolved minerals, and thus of low electrical conductance. Application of these correlations to other sites is limited - site specific correlations should be performed at each project site.

5 CONCLUSION

An exploration technique for groundwater affected by chloride brine was developed based on surface and downhole geophysical, and direct push cone penetrometer methods. Indirect and direct measurements using these methods were obtained and compared to results of traditional drilled and sampled boreholes and monitoring wells. The new technique provided good technical and economic justification for extensive use at the described site. Use of the combined geophysical and cone penetrometer exploration technique was then adopted for use at other brine release sites in the area. These programs met with very good success and acceptance by regulatory agencies.

6 REFERENCES


Groundwater drinking supply protection using cone penetrometer methods

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ABSTRACT: The Ohio Environmental Protection Agency (OEPA) has the responsibility to oversee public water systems in Ohio. When chlorinated solvents were detected in the municipal wellfield of Bridgeport, OEPA decided to try cone penetrometer exploration methods to study the problem. Penetrometer stratigraphic profiling, with groundwater, soil and soil gas sampling, and field GC/MS chemical analysis, were used to map the plume impinging on the wellfield, and locate the source of the plume. This method was then applied to studies of contaminant plumes affecting the wellfields of Coal Grove and the Belmont County Sanitary Sewer District #3, among others. This paper will present penetrometer equipment descriptions, exploration program philosophy, and short case histories of these investigations.

1 INTRODUCTION

OEPA investigations were undertaken at the Bridgeport, Coal Grove and Belmont County Wellfields, among others, to locate the sources of contaminant plumes affecting the wellfields. Another investigation objective was to gather data to evaluate potential response actions to ensure safer water supplies for the future. The choice of exploration method would be primarily controlled by four factors: 1) cost effective plume delineation - it was important that stratigraphy and groundwater chemistry be rapidly evaluated, to allow subsequent exploration points to be located based on actual site conditions, rather than on an arbitrary grid pattern. This approach would avoid expensive, multi-phased exploration typical of many groundwater studies; 2) very low contaminant concentrations - this would require use of relatively sophisticated field analytical equipment; 3) exploration in urbanized areas - it was important that methods be as unobtrusive as possible; and 4) geological conditions - wellfield aquifers were 15-30 m (50-100 ft) deep, and both gravelly and heaving sands might be encountered.

The consultant (Lawhorn and Associates) recommended, and OEPA approved, use of a cone penetrometer exploration system (Struytsky and Sainey, 1991) along with a field analytical laboratory to achieve the goals of the program. The consultant and OEPA provided senior professionals to immediately evaluate all exploration results and plan subsequent exploration locations, while the penetrometer company (STRATIGRAPHICS) provided a geotechnical engineer or hydrogeologist to evaluate stratigraphy and recommend sampling procedures. OEPA also provided oversight of the entire program.

Initial groundwater samples would be collected around the most contaminated well or lateral, in the case of the Rannay Well, to determine the direction of plume transport. A series of upgradient exploration lines would then be run transverse to the expected axis of the plume to delineate the plume and identify its source. A number of penetrometer soundings, depending on encountered geological conditions, would be performed to develop stratigraphic cross-sections. Groundwater samples would be collected at multiple depths at each location to determine the vertical distribution of contamination within the aquifer. Field analytical testing would allow a 3-dimensional profile of the plume to be rapidly developed, allowing subsequent exploration locations to be chosen based on the complete analytical database. Penetrometer soil and soil gas sampling could be performed, in addition to groundwater sampling, to confirm potential source areas.

2 CONE PENETROMETER SYSTEM

The STRATIGRAPHICS cone penetrometer system
was designed in 1986 for use during both geo-environmental and geotechnical exploration programs in difficult soil conditions. The heavy (240 kN and 300 kN or 24 and 30 ton) truck mounted rigs are fully self-contained, including data acquisition systems, dry and wet work areas, water tanks, steam cleaners, decontamination and grouting systems, separate rodstrings for sounding and sampling, optical dynamic rod driving, and heavy duty downhole equipment for use in glaciated terrains (Fig 1). Cone penetrometer (direct push) systems require no borehole to advance probes and samplers, and result in little exploration derived waste. Downhole equipment is decontaminated during retrieval using an automatic rodwasher, and open hole is pressure grouted. Most exploration activities are performed inside an enclosed portion of the rig, providing all-weather capability and a low visual presence. Truck mounted penetrometer systems can be very productive, with as much as 400 m (1300 ft) of stratigraphic logging per day, with depth capacity exceeding 60 m (200 ft). As many as 18 groundwater, or up to 30 soil or soil gas samples can be acquired in a day.

High resolution, continuous soil profiling (sounding) for geo-environmental exploration is most often performed by STRATIGRAPHICS using the indirect Piezometric Cone Penetration Test with soil Electrical Conductivity (CPTU-EC, Fig 2). The cone tip and friction sleeve resistance measurements (CPTU-EC) are evaluated for soil types and geotechnical parameters (Douglas and Olsen, 1981). The piezometric measurement (CPTU-EC) allows evaluation of soil saturation, hydraulic conductivity, potentiometric surfaces, and soil types (Saines et al., 1980). The soil Electrical Conductivity measurement (CPTU-EC) provides information on soil moisture in vadose zone soils and indications of groundwater quality in saturated soils. The EC measurement has proved very useful in exploration for inorganic (metal, brines and landfill leachate) contamination (Strutyhisky et al., 1998) and somewhat useful in LNAPL or DNAPL exploration (Strutyhisky et al., 1991).

The STRATIGRAPHICS Penetrometer Sampler can be deployed in groundwater, soil or soil gas sampling modes by using interchangeable components. The groundwater sampler is a heavy wall, shielded sampler. The shield prevents cross-contamination of the sample and screen clogging. The shield is retracted to allow groundwater to flow through a 0.5 m (20 inch) long screen into the barrel. Sample can be decontaminated from the barrel or can be pumped to the surface using an inertial pump. The sampler is typically tripped out after each sample to allow thorough decontamination. The rate of groundwater inflow and equilibrium levels can be recorded using a small...
diameter pressure transducer. Inflow test results can be analyzed using rising head slug test solutions. The soil sampler consists of a barrel, sealed with a locking piston. The piston is unlocked with a scribe line tool to obtain the sample. The sampler is tripped out of the hole for sample extraction and decontamination. The soil gas sampler is a smaller version of the groundwater sampler. Samples are contained within Tedlar bags, or are routed through portable vapor analyzers.

3 ANALYTICAL TESTING

A Hewlett-Packard Model 5890 Series II Gas Chromatograph (GC) and a Hewlett-Packard Model 5971 Series Mass Spectrometer (MS) were operated by the analytical company (Aquac Tech Environmental Laboratory-ATEL) in a trailer mounted, on-site field laboratory. A Tekmar 2000 purge and trap device, and a Tekmar 2016 auto sampler for automated sample handling, were also used. The samples were analyzed using EPA method 504.2 with a detection limit of 0.5 ug/l. The contaminants of concern (COC's) varied for each site, but were primarily chlorinated solvents, and their breakdown products.

4 BRIDGEPORT WELLFIELD INVESTIGATION

The Bridgeport wellfield is located in southeastern Ohio, along a channel of the Ohio River, and serves 3600 residents. The wellfield is within the unglaciated Appalachian Plateau Ohio River Aquifer. Beginning in 1989, very low levels of TCE and cis-1,2-DCE were detected during routine wellfield monitoring; soon PCE was also detected. After multiple well sampling events confirmed the continual presence of the COC's, OEPA decided to perform an investigation during the summer of 1994.

Ten CPTU-EC soundings for stratigraphy, 26 dissipation tests for hydraulic conductivity and potentiometric surfaces, 52 groundwater (from 57 attempts), and 9 soil gas samples were acquired during the course of a 13 day field program. Soil gas samples were obtained in various zone soils around the suspected source after the groundwater sampling program defined the limits of the plume.

Evaluation of the CPTU-EC soundings revealed variability (sand and cinder fills, buried refuse, or clay) at shallow depths. Shallow soils were typically moist to wet. Deeper stratigraphy was more uniform, and typically consisted of sands and siltysands, with some gravelly layers. Saturated conditions were typically found below depths of 10-13 m (35-45 ft). The aquifer was characterized as a continuous water table aquifer, with few, apparently discontinuous, aquiclude interlayers. Bedrock was typically encountered within about 21-26 m (70-85 ft) of the surface.

CPTU-EC soundings in the identified source area indicated somewhat different stratigraphy, with finer grained soils predominating, and significant thicknesses of interlayered sands, silts and clays. Bedrock was also found at a shallower depth (18 m or 59 ft), consistent with a steep slope of a buried bedrock valley, as indicated by nearby rock outcrops.

Plume concentration maps were developed at three different depth intervals. The upper interval, concentrations ranged from less than 0.5 ug/l, to as high as 9,700 ug/l total COC's at the source area. The upper portion of the plume angles away from the wellfield following regional groundwater flow. In the middle interval, concentrations ranged from less than 0.5 ug/l, to as high as 1,900 ug/l total COC's near the center of the plume (Fig 3). In the lower interval, concentrations ranged from less than 0.5 ug/l to 105 ug/l total COC's, again with the highest concentrations near the center of the plume.

The results indicated that while most of the COC's are drawn towards the wellfield through the middle depth interval, contamination of the wellfield is actually occurring in the lower depth interval.

Figure 3
The investigation identified the source of the contamination as a dry cleaner located about 365 m (1200 ft) northwest of the wellfield. EPA positioned 6 permanent monitoring wells along the plume. These are used as an early warning system, allowing wellfield operators to modify wellfield production to lessen capture of the plume.

5 COAL GROVE INVESTIGATION

The Coal Grove wellfield is located on the banks of the Ohio River in southern Ohio. The four production wells are configured within about 1/2 hectare (1.5 acres), and serve 4700 residents. The production wells are within alluvial deposits associated with the Ohio River and a tributary, Ice Creek. TCE and DCE have been detected in some of the production wells since 1988. One production well (CG-2) was taken out of service in 1989 due to high contaminant levels, and has been intermittently pumped to waste to control contamination of the other production wells. Five monitoring wells had been installed upgradient of the wellfield. TCE and DCE detections increase with distance upgradient.

Upgradient (southeast) of the wellfield is a coal dock. Further upgradient is a closed facility which had operated for numerous years as a truck terminal and as a tanker truck repair and cleaning operation. In 1993, USEPA conducted an emergency removal action at this facility. During the removal action, on-site wastes were found to contain various volatile organic compounds (VOCs), including TCE. USEPA determined that additional groundwater data were required to protect the wellfield from further damage. The penetrometer method successfully used at Bridgeport was chosen for use at Coal Grove. The investigation had to proceed in two parts (1994 and 1995) due to lack of permission to enter the tanker truck cleaning facility.

Fourteen CPTU-EC soundings for stratigraphy, 38 dissipation tests for hydraulic conductivity and potentiometric surfaces, and 61 groundwater samples (from 64 attempts) were acquired during the course of the first 12 day field program. CPTU-EC soundings revealed 5.5-14 m (18-46 ft) of silty clay, underlain by granular soils. The gravelly sand to silty sand aquifer saturated thickness varied from about 1.2 m (4 ft) downgradient to 13 m (43 ft) upgradient of the wellfield. The aquifer ranged from water table to confined, depending on surficial clay thickness. Bedrock was encountered at the base of the aquifer, at depths between 17-26 m (55-87 ft).

The results from field GC/MS analytical testing on obtained samples showed that the VOC plume followed a linear flow path, apparently originating to the southeast of the coal dock facility, on the northern boundary of the closed tanker truck cleaning facility (Fig 4). The plume narrowed with distance upgradient of the wellfield. The ability to rapidly obtain data from the field analytical lab, as well as stratigraphic information from the cone penetrometer, was instrumental in locating this narrow plume. As the investigation proceeded, it became apparent that if sampling had been strictly conducted on the sample grid which was originally surveyed at the site, the narrow, most contaminated portion of the plume would have been entirely missed, with significantly different conclusions as to the source of groundwater contamination.

To complete this investigation and confirm the source of the VOCs, the first program was used to support an administrative search warrant for exploration at the closed tanker truck cleaning facility. With the unexpected accompaniment of local TV news reporters, the warrant was served, the property was entered, and additional exploration was conducted. Four CPTU-EC soundings for stratigraphy, 6 dissipation tests for hydraulic...
conductivity and potentiometric surfaces, 37 groundwater samples (from 37 attempts), and 30 soil samples were acquired during the second 10 day field program.

The second program showed that the high concentrations of VOC's confirmed in a very narrow plume less than 15 m or 50 ft wide to a corner of the closed tanker truck cleaning facility's wastewater treatment plant. This confirmed that the source of the VOC contamination was the tanker truck cleaning facility. Following this sampling effort, OEPs conducted a pump test at the wellfield. The data obtained from the pump test, along with the groundwater analytical data, were used to evaluate future impacts at the wellfield. OEPs concluded that contaminant levels at the wellfield could be expected to remain constant for some time into the future. However, it was also concluded that, without the continued pumping to waste of production well CG2, higher contaminant levels would be expected in other production wells. OEPs is currently evaluating both technical and legal means to address the threat to the Coal Grove wellfield.

6 BELMONT COUNTY SANITARY SEWER DISTRICT 3 RANNEY WELL INVESTIGATION

The Belmont County Sanitary Sewer District 3 serves about 25,000 people in southeastern Ohio with water from a single Ranney Collector Well (BC#3). This well is located near the Ohio River, and consists of six laterals, which are screened near bedrock. The well can supply up to 25 million liters (6.5 million gallons) of water per day. Contaminants cis-1,2-DCE, 1,1,1-TCA, PCE and 1,1-DCA have been detected in the well.

The Ohio Department of Transportation (ODOT) discovered VOC contamination in soil and groundwater during an environmental investigation of a nearby property (Site) during a highway relocation project. The Site is about 365 m (1,200 ft) southwest of BC#3. Various chemicals, including chlorinated solvents and petroleum products, were distributed from a bulk plant located at the Site. ODOT had detected the following VOC in Site soils and groundwater: VC, PCE, TCE, cis-1,2-DCE; trans-1,2-DCE; 1,1,1-TCA; and BTEX compounds. Initial results indicated that VOC's were migrating in ground water north-northeast from the Site toward BC#3, and towards the southeast with regional groundwater flow.

ODOT notified OEPs of its findings in early 1996. During the fall of 1996, OEPs performed a field investigation to achieve the following goals: 1) identify all sources of BC#3's contamination; 2) determine the vertical and horizontal extent of contaminant plumes; 3) determine rates of migration; and 4) determine whether the suspect Site was impacting BC#3. The investigation was successful in achieving all these goals.

A major portion of the investigation was conducted using penetrometer exploration methods developed during the Bridgeport and Coal Grove investigations. Six CPTU-EC soundings for stratigraphy, 8 dissipation tests for hydraulic conductivity and potentiometric surfaces, and 55 groundwater samples (from 55 attempts) were acquired during the course of the 10 day field program. The CPTU-EC soundings revealed fine grained soils to depths of about 11-14m (36-47 ft), followed by a confined, gravelly sand aquifer. Groundwater sampling was conducted to determine the lateral and vertical extent of contamination within the aquifer.

The most common groundwater contaminant was found to be cis-1,2-DCE, with concentrations as high as 4,200 ug/l. The following cis-1,2-DCE detections were observed between the Site and BC#3: 300 ug/l about 135 m (440 ft) south of BC#3.
360 ug/l about 40 m (140 ft) south of BC#3, and 90 ug/l about 15 m (50 ft) south of BC#3. Data showed that the plume was being drawn northward from the Site, directly against the regional ground water flow direction, and into the end of the western lateral of BC#3 (Fig 5). This unusual, reversed flow path reflects the very large capture zone of the high capacity Kennedy Well. The investigation showed that VOC’s are migrating from the Site to BC#4 via ground water flow paths primarily in the upper portion of the aquifer. It also showed that the Site is apparently solely responsible for the BC#3’s cis-1,2-DCE contamination.

ODOT and OEPA agreed that excavation and off-site disposal of contaminated soils and debris prior to highway construction would best remediate the Site. Source removal, as opposed to treatment, was selected both to complete the highway project on time and to eliminate further contamination of the aquifer. The source removal took place during the summer of 1997. About 20,000 tons of contaminated, non-hazardous, solid waste and about 1,550 tons of hazardous waste were excavated and removed from the Site.

7 CONCLUSIONS

Thirty four CPTU-EC soundings, totalling 685 m (2250 ft) of data, 205 groundwater, 9 soil gas, and 70 soil samples (755 m or 12210 ft of sampler deployment) were obtained during the 45 days of field exploration for these 3 projects. Penetrometer costs (less mobilizations) totaled about $133,000. A total of 264 samples, plus numerous QA/QC samples, were analyzed by the field laboratory using GC/MS. The cost for the field analytical laboratory totaled about $45,000 (less mobilizations). While this cost is comparable to costs for off-site analyses with a 24 hr turnaround, having analytical results within 15-30 minutes of sample acquisition allowed optimal placement of subsequent exploration locations. Accurate targeting of exploration locations was the key factor in significantly decreasing overall cost and duration of these investigation programs.

OEPA rapidly and cost-effectively investigated contaminated municipal wellfield groundwater supplies by the use of high capacity penetrometer stratigraphic profiling, rapid penetrometer groundwater, soil and soil gas sampling, a sophisticated field analytical laboratory, and immediate evaluation of acquired data by senior professionals. Complex contaminant plumes, which often followed unusual groundwater flow paths, were characterized within days rather than months or years. Plumes were delineated, sources identified, and realistic models developed for planning long term monitoring and site remediation activities. OEPA has used these projects during in-house training sessions as examples of innovative and effective groundwater exploration.

REFERENCES


Geoenvironmental sampling and testing
Monitoring well casing material behaviour subject to different groundwater contaminants

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ABSTRACT: Groundwater sample integrity is of paramount importance because of the significance that the analytical result may have in subsequent remedial decisions. Contaminants in the ground water may adversely affect the well casing material, or the casing material may adsorb or desorb chemicals. Organic solvents are known to have adverse effects on fluoropolymers and thermoplastics but the effects of concentration and mixtures has only recently been addressed. Corrosion of metallic casings is a concern under oxidizing and reducing conditions and when high dissolved solids are present. This paper is a synthesis of recent studies available in the literature put into perspective by reference to current practice.

INTRODUCTION

Sampling of potentially contaminated groundwater from monitoring wells is used to determine if expensive remedial actions are required. Often as detection limits of individual contaminants become lower due to advances in analytical equipment, allowable concentrations of contaminants of concern decrease. Finally, installation of these wells represent a significant investment, so their long-term performance is important. In this light, the materials which are used for the wells are under increasing scrutiny to ensure they will not affect the water sample integrity, and they will not degrade in adverse subsurface conditions.

Three conditions to be met by an ideal well casing are strength at the desired depth, resistance to degradation in the exposed environment, and preservation of water quality (Runney and Parke, 1995). This paper will only discuss the first two concerns.

CURRENT WELL CASING MATERIALS

Fluoropolymers

Fluorinated polymers have a number of desirable characteristics including: nearly resistant to chemical and biological attack, oxidation, weathering and ultraviolet radiation; broad useful temperature range (-240°C to 287.8°C); almost chemically inert; high dielectric constant; low coefficient of friction; anti-stick properties and greater coefficient of thermal expansion than most other plastics and metals. The drawbacks are (Aller et al., 1989): cost (approximately ten times the price of PVC), and more difficult to handle as well casings (heavier, less rigid, and slippery when wet due to the low coefficient of friction). Polytetrafluoroethylene (PTFE) (Teflon) is the most widely used and produced fluoropolymer. Fluoroelastomer Polypropylene (FEP) is the second most widely used fluoropolymer possessing the same physical characteristics as PTFE with a smaller temperature.

Thermoplastics

Polyvinyl Chloride (PVC) resin can be combined with various stabilizers, lubricants, pigments, fillers, plasticizers and processing aids. These additives can be varied to produce desired properties for specific applications. Type I PVC (rigid, unplasticized polymer), denoted as RPVC (FPVC is flexible PVC), is used for well casings. Acrylonitrile Butadiene Styrene (ABS) is the combination of three different monomers (acrylonitrile, butadiene, and styrene). The ratios of the components can be tailored for specific applications. For well casings, rigid, unplasticized ABS is used resulting in good heat resistance and impact strength. The high temperature resistance and ABS ability to retain other properties better at high temperatures is an advantage in wells in which grouting causes a high test of hydration. (Aller et al., 1989). Some advantages are: light weight, high...
ablation resistance, high strength to weight ratio, durable in natural groundwater environments, require low maintenance, flexible and workable, relatively low in cost, and complete resistance to galvanic and electrochemical corrosion. However, long-term exposure to ultraviolet rays and/or low temperature can cause brittleness and gradual loss of impact strength. Many manufacturers now include protection against degradation by ultraviolet light, but brittleness is still a problem especially during installation. (Aller et al., 1989).

Metals
Metallic well casings are stronger, more rigid, and less temperature sensitive than the other materials. The downfall in its susceptibility to corrosion in certain subsurface environments in the long-term. (Aller et al., 1989). The most common stainless steel alloys used are the type 304 and type 316. Type 304 has a greater resistance to sulfur-containing species and sulfurous acid solutions. (Aller et al., 1989). Other steel casings used include: carbon steels, low-carbon (copper) steels possessing 0.2% carbon, and galvanized steel.

Fiberglass Materials
Fiberglass products used as well casings include fiberglass-reinforced plastic (FRP) and fiberglass-reinforced epoxy (FRE). FRP is made of 70% fiberglass and 30% polyester resin while FRE is a combination of 75% high-silica glass and 25% high-purity, closed molecular epoxy. (Ranney and Parker, 1994).

CHEMICAL RESISTANCE
Chemical attack includes: galvanic/electrochemical corrosion (metallic group most susceptible to this) and chemical degradation (thermoelastic group most susceptible to this). Increasing organic content of the solution results in direct attack on the polymer matrix or more subtle effects due to solvent absorption, absorption, and/or leaching. (Barcelona et al., 1983).

Fluoropolymers
The general consensus in the literature is that fluoropolymers are the most inert to both types of chemical attack (Ranney and Parker, 1995). Aller et al., (1989) state the PTFE is unaffected by extremely aggressive acids (i.e. hydrofluoric, nitric, sulfuric, hydrochloric) and organic solvents.

However, recent laboratory studies have shown some degradation of fluoropolymers. PTFE and FEP had slight weight gains from exposure to five different organic compounds (chloroform, methylene chloride, trans-1, 2-dietheroxyethylene, tetraetheroxyethylene, trichloroethylene) with FEP showing slightly less gain than PTFE (Table 1). There were, however, no signs of softening, swelling, or strength decrease. Results also showed excellent resistance to alkaline and acidic conditions.

Thermoplastics
Polymeric materials have superior resistance under acidic or high-dissolved-solids conditions but PVC is noted to have definite weaknesses under chemical exposure to low-molecular weight ketones, aldehydes, and chlorinated solvents (Barcelona et al., 1983; Aller et al., 1989). They also have excellent resistance to biological and chemical attack by soil, water, and other naturally occurring substances present in the subsurface. Weight gain on a number of thermoplastics in the presence of various chemical compounds are reported in Table 1. As the organic content in an aqueous solution increases, the degree of chemical attack on the polymer matrix is enhanced. (Aller et al., 1989). According to Barcelona et al., 1983) the general chemical resistance of thermoplastics is improved through the incorporation of less ingredients in their formulation (i.e. plasticizers).

Recent work at the US Army Cold Regions Research Laboratory (CRRRL) (Parker and Ranney, 1994, 1995; Ranney and Parker, 1995) have examined in detail the degradation of PVC due to organic solvents. Their work expanded on that of Beers (1985) which predicted the permeation of organics through PVC pipe based on PVC samples exposed to a range of aqueous concentrations of the solvents. The behavior was related to the Fick-Huggins equation:

\[ \ln(V_1) = \ln(V_0) + (1 - V_0) + \chi (1 - V_1)^2 \]  [1]

Where \( a \) is the activity of the solvent, \( V_1 \), the volume fraction of the organic chemical in the polymer, and \( \chi \) is the interaction parameter. The activity, \( a \), can be approximated by dividing the compound concentration by its solubility in water. A plot of [1] is shown in Figure 1 and a summary of their results is in Table 2. PVC is only softened by solvents with \( \chi \) value less than 1 at ambient temperatures; the lower the
value of $\chi$ the greater the effect. A neat solvent with $\chi < 0.5$ can completely dissolve PVC (Berens 1985). Table 3 lists some values of $\chi$. Generally they found reasonable agreement to the Berens predictions for solutions of single organic compounds except methylene chloride where softening was observed with $a = 0.1$ after 20 weeks. When mixtures of organic solvents were used at low activities (3 solvents each with $a = 0.3$, and 18 solvents each with $a = 0.05$) softening of PVC was observed. In these experiments the cumulative activities was 0.9. Thus there is a cumulative effect for mixtures of solvents, and may have been synergistic effects. Additional studies are being conducted with mixtures of solvents at lower activities.

Through these studies, they concluded that at lowest activities, diffusion of organic solutes in RVPVC is purely Fickian (concentration independent) and slowest (taking thousands of years), while at higher activities diffusion becomes successively concentration-dependent and then diffusion becomes anomalous (Case II or frontal) increasing by several orders of magnitudes.

**Metallics**

The weakest attribute of metallics is electrochemical corrosion under oxidizing and reducing conditions, which is aggravated by high dissolved solids. Potential sites for a variety of chemical reactions and absorption are created on corroded surfaces which may cause significant changes in the dissolved metallic or organic compounds in water. A common solution to this problem is flushing, however this may not be sufficient to minimize such bias. (Barcelona et al., 1983; Aller et al., 1989). Carbon steels have improved resistance to atmospheric corrosion, but this is only achieved through alternate cycles of wet and dry (Barcelona et al., 1983). In most monitoring well situations, water fluctuations are not sufficient in duration or occurrence to provide these conditions (Aller et al., 1989). Copper (low-carbon) steels are equivalent to carbon steels in corrosion resistance. In reducing conditions, higher levels of truly dissolved metallic corrosion products are generated. For both carbon and copper corrosion, the products of this process are: iron, manganese, trace metal oxides, and various metal sulfides. Under oxidation, the principle products are solid hydroxides oxides which may accumulate in the well or most importantly in the well screens (Barcelona et al., 1993). Galvanized steels provide a slight improved corrosion resistance over conventional steels. Its corrosion products include iron, manganese, zinc, and trace cadmium (Barcelona et al., 1983, and Aller et al., 1989).

Stainless steels are the most resistant to corrosion. Exposure to oxygen (most monitoring situations) is required to attain its highest corrosion resistance because oxygen combines with part of the stainless steel alloy to form an invisible protective film on the surface (Aller et al., 1989). Some researchers suggest biological activity may alter the geochemistry near stainless steel wells inducing degradation (Aller et al., 1989; Parker, 1992). Stainless steel is most susceptible under (Raney and Parker, 1995):

- $pH < 7.0$ (acidic conditions)
- Dissolved oxygen content $> 2$ ppm
- $H_2S$ Hydrogen sulfide content $> 1$ ppm
- Total dissolved solids concentration $> 1000$ ppm
- $CO_2$ (carbon dioxide) concentration $> 50$ ppm
- $Cl^-$ (chloride ion) concentration $> 500$ ppm

Stainless steel has a poorer resistance under oxidizing conditions (than other chromium-nickel steels), however reducing conditions are most often encountered (Barcelona et al., 1983). Lab studies have shown rusting normally occurred at the weld, thus threaded stainless steel weld casings are recommended (Parker, 1992). Stainless steel 316 is less vulnerable to pitting or pin-hole corrosion caused by organic acids or hydrolysis solutions than stainless steel 304 (Barcelona et al., 1983).

Long term exposure of either type to very corrosive conditions result in chromium and nickel contamination of the samples (Aller et al., 1989).

**Fiberglass**

Raney and Parker (1995) found that FRB showed no effects to alkaline conditions, swelling.
### Table 1: Material percentage weight gain in organics after 112 days (After Parker and Runney (1995))

<table>
<thead>
<tr>
<th>Compounds</th>
<th>PTFE</th>
<th>FEP</th>
<th>PVC</th>
<th>ABS</th>
<th>FRP</th>
<th>FRE</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Hydrocarbons (aliphatic and aromatic)</strong></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>Benzene</td>
<td>0.4</td>
<td>0.3</td>
<td>48.7</td>
<td>D</td>
<td>0.8</td>
<td>0</td>
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<tr>
<td>Gasoline (93 octane, unleaded)</td>
<td>0.3</td>
<td>0.2</td>
<td>0.1</td>
<td>61.9</td>
<td>0.1</td>
<td>-0.1</td>
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<tr>
<td>Kerosene (-1)</td>
<td>0</td>
<td>0</td>
<td>8.9</td>
<td>D</td>
<td>0.2</td>
<td>0</td>
</tr>
<tr>
<td>Toluene</td>
<td>0.2</td>
<td>0.2</td>
<td>51.4</td>
<td>D</td>
<td>0.9</td>
<td>0</td>
</tr>
<tr>
<td>o-xylene</td>
<td>0.1</td>
<td>0.1</td>
<td>65.7</td>
<td>D</td>
<td>0.2</td>
<td>-0.1</td>
</tr>
<tr>
<td><strong>Chlorinated solvents (aliphatic and aromatic)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Carbon tetrachloride</td>
<td>0.6</td>
<td>0.4</td>
<td>0.1</td>
<td>317.2</td>
<td>0.2</td>
<td>0</td>
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<tr>
<td>Chlorobenzene</td>
<td>0.3</td>
<td>0.3</td>
<td>159.8</td>
<td>D</td>
<td>7.8</td>
<td>0.2</td>
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<tr>
<td>Chloroform</td>
<td>1</td>
<td>0.8</td>
<td>223.9</td>
<td>D</td>
<td>L</td>
<td>7.3</td>
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<tr>
<td>1,2-dichlorobenzene</td>
<td>0.2</td>
<td>0.1</td>
<td>217.7</td>
<td>D</td>
<td>1.1</td>
<td>0.1</td>
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<tr>
<td>Methylene chloride</td>
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<td>0.8</td>
<td>D</td>
<td>D</td>
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<tr>
<td>Tetrachloroethylene</td>
<td>0.9</td>
<td>0.6</td>
<td>1.7</td>
<td>251.2</td>
<td>0.5</td>
<td>0</td>
</tr>
<tr>
<td>Trichloroethylene</td>
<td>1.3</td>
<td>1.1</td>
<td>70.9</td>
<td>D</td>
<td>L</td>
<td>0.3</td>
</tr>
<tr>
<td><strong>Oxygen-containing compounds (either a ketone, alcohol, aldehyde, or ether)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Acetone</td>
<td>0.3</td>
<td>0.2</td>
<td>157.8</td>
<td>D</td>
<td>5.6</td>
<td>2.7</td>
</tr>
<tr>
<td>Benzyl alcohol</td>
<td>0</td>
<td>0</td>
<td>0.1</td>
<td>D</td>
<td>0.5</td>
<td>0.1</td>
</tr>
<tr>
<td>Methyl alcohol</td>
<td>0</td>
<td>0</td>
<td>0.4</td>
<td>27.8</td>
<td>1.9</td>
<td>7.7</td>
</tr>
<tr>
<td>Methyl ethyl ketone</td>
<td>0.3</td>
<td>0.2</td>
<td>D</td>
<td>D</td>
<td>4.8</td>
<td>3</td>
</tr>
<tr>
<td><strong>Nitrogen-containing compounds</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N-Methylamine</td>
<td>0.2</td>
<td>0.1</td>
<td>D</td>
<td>D</td>
<td>1</td>
<td>L</td>
</tr>
<tr>
<td>Nitrobenzene</td>
<td>0.1</td>
<td>0</td>
<td>D</td>
<td>D</td>
<td>I</td>
<td>0.4</td>
</tr>
<tr>
<td><strong>Acids and bases</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Acetic acid (glacial)</td>
<td>0.4</td>
<td>0.3</td>
<td>0.4</td>
<td>76.8</td>
<td>1.5</td>
<td>F</td>
</tr>
<tr>
<td>Hydrochloric acid (25% w/v)</td>
<td>0</td>
<td>0</td>
<td>0.3</td>
<td>1.2</td>
<td>-5</td>
<td>-7</td>
</tr>
<tr>
<td>Sodium hydroxide (25% w/v)</td>
<td>0</td>
<td>0.1</td>
<td>0.1</td>
<td>0.9</td>
<td>1.5</td>
<td>0.2</td>
</tr>
</tbody>
</table>

F Fibers separated; L Glass fiber sheets separated; D Dissolved or disintegrated upon handling

### Table 2: Results of softening RVPVC by aqueous solutions of organic solvents

<table>
<thead>
<tr>
<th>Compound</th>
<th>Duration</th>
<th>Activities Required</th>
<th>Berkens Prediction</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVC swelling agents</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>methylene chloride</td>
<td>4-168 hours</td>
<td>0.6, 0.8</td>
<td>&lt; 0.6</td>
</tr>
<tr>
<td>trichloroethylene</td>
<td>1-20 weeks up to 68 days</td>
<td>0.4, 0.2, 0.1</td>
<td>0.88</td>
</tr>
<tr>
<td>Mixed-organic solvents</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>methylene chloride</td>
<td>2-21 days</td>
<td>Softening Occurred</td>
<td>n/a</td>
</tr>
<tr>
<td>1,2-dichloroethane</td>
<td>Significant hardness reduction</td>
<td>21 days</td>
<td>21 days: total weight gain – 10% rubbey and bent easily</td>
</tr>
<tr>
<td>1,2-dichloroethane all with χ &lt; 0.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>each activity = 0.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Miscible-solvents</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>acetone</td>
<td>3-7 days</td>
<td>Softening Occurred</td>
<td>n/a</td>
</tr>
<tr>
<td>dimethylformamide</td>
<td>12 days</td>
<td>≥ 50% concentration</td>
<td></td>
</tr>
<tr>
<td>pyridine</td>
<td>8 days</td>
<td>60% concentration</td>
<td></td>
</tr>
<tr>
<td>tetrahydrofuran</td>
<td>&gt; 7 days</td>
<td>40% concentration</td>
<td></td>
</tr>
</tbody>
</table>
Table 3: Interaction parameters (χ) for PVC organic liquid systems. (After Berens, 1985)

<table>
<thead>
<tr>
<th>Compound</th>
<th>χ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Methylene chloride</td>
<td>&lt; 0.53</td>
</tr>
<tr>
<td>Chloroform</td>
<td>0.64</td>
</tr>
<tr>
<td>1, 1-dichloroethane</td>
<td>&lt; 0.68</td>
</tr>
<tr>
<td>1,1,1-trichloroethane</td>
<td>0.85</td>
</tr>
<tr>
<td>1,1,2-trichloroethane</td>
<td>&lt; 0.56</td>
</tr>
<tr>
<td>Tetrachloroethylene</td>
<td>1.17</td>
</tr>
<tr>
<td>Benzene</td>
<td>0.83</td>
</tr>
<tr>
<td>Toluene</td>
<td>0.80</td>
</tr>
<tr>
<td>Xylenne</td>
<td>0.89</td>
</tr>
<tr>
<td>Vinyl chloride</td>
<td>0.98</td>
</tr>
</tbody>
</table>

Softening, and hydrocarbons and aromatic solvents, but had slight (about 5%) weight losses in acidic environments probably due to loss in epoxy resin. FRP was more easily degraded than the fluoropolymers and PTFE as 8 organics caused delamination (separated fiber sheet), both acidic and alkaline conditions caused weight variations, and there was some swelling. FRP showed no effects due to hydrocarbons and non-polar chlorinated solvents, but it was more sorptive to dilute organics solutes than PVC.

GROUNDWATER INTEGRITY

Casing material can bias sampling of groundwater by sorption of chemicals (organic and metallic) from the water onto or into the casing, and leaching of casing constituents into the water. Adsorption results in excess contaminant concentration at the solid surface followed by absorption which is diffusion of the contaminant into the solid (Bedient et al., 1994). Sorption is influenced by many factors including: chemical form and concentration of the contaminant, solution characteristics (pH, total dissolved oxygen, salinity, hardness, complexing agents, dissolved gases, suspended matter, and microorganisms), properties of the casing (chemical composition, surface roughness, surface cleanliness, surface area to volume ratios, and history of the containment), and external factors (temperature, contact time, exposure to light, and agitation) (Parker, 1992).

Leaching is the release of chemical constituents of the casing material into groundwater in the presence of aggressive aqueous solutions. Factors which may influence leaching include: solution pH, temperature and ionic composition, exposed surface area, and surface porosity of the material (Aller et al., 1989).

Sorption Of Organics

Fluoropolymers are generally accepted as having the least effect on groundwater integrity (Barcelon et al., 1983), however more recent studies have shown that this may not be true in some instances. Parker et al. (1990) showed significantly greater sorption of some organics by PTFE compared to RPVC. Gillham and O'Hannesin (1990) noted that PTFE sorbed some aromatic hydrocarbons (toluene, ethylbenezene, and p-xylene) more rapidly than RPVC. Parker et al. (1990) studied sorption at ppm concentrations and Parker and Runrey (1993) studied sorption at ppb levels of numerous organics to PTFE, PVC, and SS and supported these observations. The sorption study of trace-level organics by Runney and Parker (1994) included FEP, ABS, FRP, and FRP and indicated that PTFE and FEP were intermediate in sorptive capability (between PVC and ABS). The results of these studies are contained in Table 4.

Parker and Runney (1993) concluded that there was no difference in rates of sorption between the ppm and ppb levels of the organics tested for PVC and PTFE. Thus the sorption process is absorption involving partitioning and dissolution into the polymeric matrix, and diffusion is independent of the concentration. They concluded that at ppm and ppb levels of organics PVC well casings are suitable for monitoring wells, however if high concentrations of a PVC solvent (concentrations with activities ≥ 0.6) were anticipated then PVC should not be used.

Considering fibreglass materials, FEP (equal to PVC) was observed to be the least sorptive of organics while FRP was comparable in performance to the fluoropolymers (between FEP and ABS). Sorption of organics by metallic well casings is not generally a concern (Parker, 1992).

Leaching Of Organics

Berens (1985) and other researchers comment on the ability of PTFE to desorb sorbed organics, as it was found that the amount desorbed did not parallel the amount sorbed. The conclusion made was that smaller molecules may be more easily or rapidly desorbed. The most concern over the susceptibility of

681
<table>
<thead>
<tr>
<th>Compound</th>
<th>PTFE</th>
<th>FFPE</th>
<th>PVC</th>
<th>ABS</th>
<th>SS</th>
<th>FFE</th>
<th>FRP</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Benzene</td>
<td>24-48</td>
<td>168</td>
<td>48-96</td>
<td>8</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Benzene</td>
<td>72-168</td>
<td></td>
<td></td>
<td></td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Toluene</td>
<td>3-6</td>
<td></td>
<td>24-48</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Ethylbenzene</td>
<td>1-3</td>
<td></td>
<td>12-24</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>m-xylene</td>
<td>3-6</td>
<td>8</td>
<td>12-24</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>o-xylene</td>
<td>6-12</td>
<td>24</td>
<td>12-24</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>p-xylene</td>
<td>1-3</td>
<td></td>
<td>12-24</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
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<tr>
<td>RDX</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>Trinitrobenzene</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>chloro-1,2-dichloroethylene</td>
<td>500</td>
<td>168</td>
<td>24</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>trans-1, 2-dichloroethylene</td>
<td>168</td>
<td>168-1000</td>
<td>24-72</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>Trichloroethylene</td>
<td>8</td>
<td>8</td>
<td>160-1000</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>Chlorobenzene</td>
<td>24</td>
<td>24</td>
<td>160-1000</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>o-dichlorobenzene</td>
<td>24</td>
<td>24</td>
<td>160-1000</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>p-dichlorobenzene</td>
<td>24</td>
<td>24</td>
<td>160-1000</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>1,1,1-trichloroethane</td>
<td>24</td>
<td>24</td>
<td>160-1000</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>Bromoform</td>
<td>&gt;800</td>
<td>&gt;800</td>
<td>&gt;800</td>
<td>&gt;800</td>
<td>&gt;800</td>
<td>&gt;800</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>Tetracloroethylene</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>24</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>Tetrachloroethylene</td>
<td>&lt;1</td>
<td>168-1000</td>
<td>72</td>
<td>1</td>
<td>&gt;1344</td>
<td>&gt;1344</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>1,1,1,3-tetrachlorobenzene</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>24</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>2,4,6-trinitrotoluene</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>24</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>Nitrobenzene</td>
<td>500</td>
<td>500</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>O-nitrotoluene</td>
<td>1000</td>
<td>1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>p-nitrotoluene</td>
<td>168</td>
<td>500</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>&gt;1000</td>
<td>1000</td>
<td>1000</td>
</tr>
</tbody>
</table>

Sources: 1 Ranney and Parker (1995); 2 Gillham and O'Hannas in (1990); 3 Parker and Ranney (1993); 4 Parker et al. (1990); 5 Reynolds and Gillham (1986).

a Less than 5 minutes; b Time for significant sorption but < 10%

PVC to bias water samples is due to the leaching of components. Barcelo et al. (1983) found the vinyl chloride monomer leaching into water at low pH levels under prolonged solution exposures. However, Parkers (1992) notes that vinyl chloride monomer leaching has now become regulated by the National Sanitation Foundation thereby reducing this problem. PVC contain less than 0.01% plasticizers and is therefore not expected to leach large quantities but may contain up to 5% of additives (pigments, antioxidants, thermal stabilizers, inorganic filler) which may contribute to sample bias. PVC may be coated with natural or synthetic waxes, fatty acids or fatty acid esters while solvent prsescents and cements used often contain solvents (methyl-ethylketone, and dimethylformamide) which may cause severe problems in the determination of priority pollutants. Other sources of bias include additives present as compounding ingredients (pipe colors, protection from oxidation or exposure to sunlight, etc.) The most probable important factor contributing to leaching is prolonged exposure to aggressive aqueous organic mixtures. Again there is significant bias initially, with slow diffusive release thereafter which is expected to continue for some time.

Another source of leachable bias is due to the lubricants and joining compounds at casing joints used in thermoplastics. For this reason, Barcelo et al. (1983) suggests the use of threaded joints to avoid these problems. Ranney and Parker (1994) also found ABS leached several constituents used in manufacturing.

Sorption Of Metals

Ranney and Parker (1996) examined sorption of metal ions to PTFE, FEP, PVC, FEP, FEP and FRP. Anions did not strongly associate with polymeric surfaces, however sorption of cations did occur. The results are shown in Table 5. PEFP and PTFE were found to be the least sorptive of metals. Under static conditions there was relatively little sorption of several cations (lead, cadmium, copper, iron), but under dynamic conditions (water sampling procedures) significant amounts of cations were sorbed by PTFE (cations, lead, iron) though substantially lower than SS. Further
Table 5: Mean Normalized Concentrations of Metal Ions After 72 Days
(After Ranney and Parker, 1996)

<table>
<thead>
<tr>
<th>Material</th>
<th>PTFE</th>
<th>FEP</th>
<th>PVC</th>
<th>FRE</th>
<th>FRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arsenic</td>
<td>0.99</td>
<td>0.99</td>
<td>0.99</td>
<td>0.93</td>
<td>1.00</td>
</tr>
<tr>
<td>Cadmium</td>
<td>0.99</td>
<td>0.98</td>
<td>0.79</td>
<td>0.94</td>
<td>0.94</td>
</tr>
<tr>
<td>Chromium</td>
<td>0.99</td>
<td>1.00</td>
<td>0.99</td>
<td>0.98</td>
<td>1.00</td>
</tr>
<tr>
<td>Lead</td>
<td>0.93</td>
<td>0.93</td>
<td>0.56</td>
<td>0.45</td>
<td>0.52</td>
</tr>
</tbody>
</table>

studies into this occurrence demonstrated that FEP was the only polymeric material with no effect to cadmium cations. PTFE sorbed more cadmium but as the study progressed sorption decreased as leaching began to govern. FEP and PTFE were found to sorb insignificant amounts of lead cations.

Most lab studies show stainless steel sorbs and leaches metals the most. Parker (1992) reports tremendous variability in the concentration of cadmium and chromium sorption. Parker et al. (1990) found both SS 304 and SS 316 to slowly sorb lead (10% sorption of arsenic in 24 hours, and 13% sorption of chromium in 8 hours by SS 316). Sorption of cadmium was only reported for SS 304 and then only under low dissolved oxygen, while both SS 204 and SS 316 were sorptive of lead.

Leaching Of Metals
Ranney and Parker (1996) studied leaching of Barium, Cadmium, Chromium, Copper, Lead, Nickel, Zinc and Silver from PTFE, FEP, PVC, FRE and FRP. In all cases but one, after 40 days metal levels were well below EPA drinking water standards. Lead was leached in significant quantities from FRP. Parker (1992) explains that leaching of metals may be a concern for PVC due to the stabilizer used in its formulation. Outside the US, lead stabilizers may be used, with leaching attributed to dissolution of a lead-rich layer on the inner surface of new pipe followed by minimal additional leaching once this layer has been removed. This layer can be removed by pretreatment with either an alcoholic sodium hydroxide solution or a mixture of dilute nitric and perchloric acids.

Hewitt (1993) examined the leaching and sorption of metals by stainless steel. Chromium was consistently leached from SS 304 in the three shortest residence times while the release of significant amounts of copper and nickel by SS 304 was independent of residence times. SS 316 also leached significant amounts of copper and nickel. SS 304 leached the most chromium and nickel.

CONCLUSIONS
The above literature review illustrates that no one well casing material will satisfy all groundwater monitoring situations; however, not one situation (to date) requires such a totally inert material. The

---

Table 6: Factors Affecting Well Casing Selection for Monitoring Wells
(After Ranney and Parker, 1996)

 Degradation by chemicals
(from least degraded to most)
PTFE < FEP < FRE < FRP < PVC < ABS
PVC, ABS, FEP, FRP can be degraded by high concentrations of some organic solvents when:
- organic solvent is a good solvent of the polymer, and
- solvent is present in concentrations > 0.1 times the chemical's aqueous solubility.

Corrosion of Stainless Steel
pH < 7.0
DO > 2 ppm
H₂S ≥ 1 ppm
Total dissolved solids > 1000 ppm
CO₂ > 50 ppm
Cl⁻ > 500 ppm

Sorption of Organic Solutes
(from least sorptive to most)
FE, PVC < FEP, PTFE, FRE, FRP << ABS

Leaching of Organic Constituents
(from least leached to most)
PVC, FEP, PTFE < FRE < FRP << ABS

Impact on Metal Concentration
(from least to most)
FEP, PTFE < PVC < FRE < SS

Cost of Casing Material
(cheapest to most expensive)
PVC < FRE < FRP < SS 304 < SS 316 < FEP, PTFE
material chosen must be based on which one best suits the subsurface and budgetary requirements. The above literature review is summarized in Table 6.

Based on the results discussed herein, PVC remains the first choice for most monitoring applications. Where higher strength is required or high concentrations of organic solvents are present, stainless steel is likely the most appropriate. If either PVC or SS cannot be used, FRE may prove quite useful, especially in deeper wells.

REFERENCES


Engineering significance of ground sulphates

A. Brian Hawkins
HIM Geotechnics, Charlotte House, Bristol, UK

ABSTRACT: Analysis of ground chemistry is commonly specified in site investigations to determine the potential aggressivity of the ground towards emplaced concrete and/or steel. However, little consideration is given to the dynamic nature of ground chemistry and the changes which may occur as a consequence of engineering works.

This paper reviews the development of ground sulphates in dark pyritic mudrocks. The lowering of ground water levels as a consequence of engineering construction creates conditions conducive to the proliferation of bacteria which initiate or accelerate the development of ground sulphates and modify the pH value of the soil. The paper discusses the importance of appropriate sampling and testing to assess not only the influence of sulphates on buried concrete/steel but also the potential for sulphate induced heave. It cautions against relying solely on the analysis of ground water samples, stressing the importance of considering present and potential ground conditions and the purpose for which the sampling and testing is undertaken.

INTRODUCTION

Almost every site investigation includes some testing of ground chemistry to assess the sulphate content and pH of the soils in or on which a structure is to be built. With the exception of arid environments, this is generally undertaken to assess the potential aggressivity of the ground conditions towards emplaced concrete. More recently, it has been appreciated that the development of sulphates in soils may cause ground heave and in addition, that the presence of these salts can influence the chemical reactions which take place when lime is used to “enhance ground strength”, again resulting in heave.

Many engineers undertaking site investigations do not appear to appreciate the variability in ground chemistry within a particular stratum and the changes which may occur as a natural process, or be initiated/accelerated by the activities of man. Too often when a site investigation takes place before construction, the results obtained are for an environment which may be very different from that which pertains after the development has taken place. As a consequence, the initial interpretations are likely to be inappropriate for a consideration of the long term integrity of the works.

Since the pioneering work of Ogilvy and Vogan (1970) who studied heave problems in the Ordovician mudrocks of the Ottawa area, a number of other case studies have been reported indicating the importance of sulphate development in creating heave beneath ground bearing floor slabs. Although a number of workers have drawn attention to the significance of sulphate development and its interaction with other clay minerals and emplaced lime in “stabilised” ground, to date there is no readily available Standard advising on the most appropriate way of sampling/testing for such events.

This paper describes the nature of ground sulphate development in dark mudrocks and discusses some of the problems which may result from a change in ground chemistry due to engineering works. It highlights the dynamic nature of the ground chemistry drawing attention to both spatial and temporal variations and provides some guidance on appropriate sampling and testing of a green field site, taking into account its eventual use and the ground conditions which are likely to prevail when the engineering construction has been completed.

SUSCEPTIBLE STRATA

Whilst some arid/semi-arid strata contain large evaporite deposits, in temperate latitudes these are not normally significantly affected by engineering works; except where a change in the environment creates Wet Rock Head and dissolution takes place resulting in ground subsidence. In contrast to these generally red terrigenous (oxidised) strata, in marine sediments iron sulphide develops in anoxic, saturated, salt-rich conditions and disseminated pyrite forms as discrete crystals or frambooids. Most marine sediments contain between 2% and 5% iron sulphide, although the pyritic content may be as high as 10%. In these
anoxic conditions, the scarcity of macro-organisms means there is little bioturbation and hence commonly these dark marine sediments are well bedded and frequently referred to as fissile mudrocks or shales.

DEVELOPMENT OF GROUND SULPHATES

Iron pyrite crystals remain relatively stable as long as the environmental conditions in which they were formed persist. In the Tertiary and Quaternary, the changing climatic conditions resulted in variations in the ground water level, hence the evaporation front and zone of aeration extended deeper during the drier or warmer periods. Whilst these changes took place gradually over geological time, engineering construction can induce similar changes not only over a short time period but also without any modification of the climatic/weather conditions.

In most temperate latitudes, climatic changes result in a seasonal variation in the depth of aerated or partially unsaturated ground, below which the strata remain fully saturated. Beneath the seasonally changing ground water level, the weak rocks/engineering soils are generally unoxidized, hence retain their grey/dark grey coloration and the minerals remain largely in their original, chemically unaltered state. Above the ground water level, however, where the surface materials are at least partially aerated, decomposition of many of the contained minerals initiates a chain of other reactions, including dissolution of carbonates and the oxidation of iron minerals. The change in state from ferric to ferrous iron causes the brown or orange brown colour characteristic of most near-surface marine sediments.

Figure 1 shows the variation in iron and calcite contents with depth through a typical profile of a trial pit in dark laminated mudrocks in Cardiff, Wales. It will be noted that above the average ground water level at 1.3 m there is an inverse relationship between the ferrous and ferric ion; the percentage FeO reducing towards the ground surface while the percentage Fe₂O₃ increases. Effectively no calcite was present above 1 m in this trial pit while 6 to 7% CaCO₃ was recorded below 1.25 m. The leaching of near-surface calcite frequently results in the formation of a zone in which pea sized nodules/concretions of calcium carbonate develop.

When the ground water is low and an aerated zone is present, the weathering of the iron sulphides results in the creation of a thin veneer of decomposed material around the crystals. Using the scanning electron microscope such workers as Kelly, Norris and Brierly (1979) have identified Thiobacillus ferrooxidans, Thiobacillus thiooxidans and Sulfobolus on the face of a decomposing pyrite crystal. It is now generally accepted that T. ferrooxidans is the main bacterium involved in the oxidation of iron sulphide. Thiobacilli only thrive in oxygenated, moist environments, such as exist above the ground water level. These bacteria use pyrite as a nutrient source and by removing the protective weathered skin on the crystals, allow further decomposition to take place. Their activity in converting sulphur to H₂SO₄ results in an increasingly acidic environment. The bacteria are most prolific in acidic conditions and as a consequence of their multiplication, the pH of the host material is progressively lowered.

In addition, the sulphuric acid released by the bacteria is available to interact with other minerals with which it has a chemical affinity, including calcite (to produce the mineral gypsum) and the clay mineral illite (to form jarosite). The jarosite generally occurs as a yellow brown discontinuous coating on oxidised marine mudrocks while the gypsum (calcium sulphate) is found mainly in the crystal form of anhydrite. In Britain during the winter, dry periods in the past, the ground water level would undoubtedly have been lower and it is considered that it was during these periods in the Tertiary/Quaternary that the large anhydrite crystals developed in the main marine Clay stratigraphic units. These crystals, frequently with a short axis of more than 5 mm, appear to have formed within a weathered soil mass. In contrast, the more recently formed sucrose gypsum (small, sand-sized crystals of <1 mm) generally occurs along discontinuities - either bedding surfaces or the random fissuring resulting from near-surface stress release.

Most chemical analyses for sulphate and pH are undertaken to determine the potentially aggressive effect of these minerals on concrete and other contained reinforcement. This is sufficiently well documented to need no further description. Unfortunately, the possibility of sulphate-related heave and the effect of ground sulphates in lime stabilization programmes is frequently not taken into account.

Some brief case histories are given below to illustrate the problems which may arise in structures and with highways as a result of sulphate growth.

GROUND HEAVE DUE TO SULPHATE GROWTH

Seasonal ground heave related to the

![Figure 1: Changes in calcite, ferrous and ferric iron contents with depth, west pit, Llandough Hospital.](image)
shrinking/swelling characteristics of clay-rich soils is a world-wide phenomenon. Although not extensively studied in Britain until the drought period in the mid-1970s, engineers now too readily attribute this explanation to any form of ground heave in such material. Despite the problems which occurred in the 1960s at the Rudas Health Centre, constructed in the dark Ordovician shales at Otowa (Quigley and Vogan, 1970; Quigley, Zajic, McKyes and Yong, 1973) there is little in the standard publications or Codes describing the phenomenon of sulphate heave. Quigley et al noted that gypsum has double the volume of the two minerals from which it initially forms (calcite and pyrite). As a consequence of the site drainage and the increased temperature of the shales beneath the Health Centre, new selenite growth was initiated which caused the ground bearing floor slabs to rise, although little movement was noted in the area of the load-bearing walls.

Hawkins and Fancher (1987 a, b; 1997) describe a similar situation at Llandough Hospital, Cardiff, constructed partially on very dark mudrocks of Upper Triassic age. In this case selenite crystals were not present in the trial pits excavated outside of the hospital, but between approximately 0.9 and 1.75 m in the interior pit, the bedding surfaces of the dark mudrocks were effectively covered with platy selenite crystals. Comparing the floor level in the disturbed area to a reference datum on a different stratum indicated a rise of 81 mm (Figure 2). It is unlikely this was entirely due to heave as undoubtedly there would have been some inaccuracies in the floor level when it was laid some 50 years previously. Nevertheless, from the measured dimensions and a study of the outer walls, it would be reasonable to assume that in the area of distress, the floor slab had risen by at least 60 mm.

In the late 1980s, severe distress occurred in a number of homes built ten years previously on the Lias Clay at Gloucester. When trial pits were dug around these houses and in some cases beneath the floors, selenite selenite was seen to have developed within the semi-horizontal bedding planes; the crystal growth occurring in bands up to 3.5 mm thick. In an area where the original ground level had been reduced in height as part of the works, much of the trial pit profile between 0.5 and 1.25 m indicated 10 to 20% of the vertical height consisted of gypsumiferous material. It was not surprising therefore that the surveyors confirmed some floor slabs had risen by over 25 mm in a three year period (Hawkins, 1997).

The above examples demonstrate the significance of ground heave related to chemical reactions which can be initiated when the ground water level, temperature regime and climate temperature regimes are changed as a consequence of construction on dark slightly calcareous pyritiferous mudrocks.

**Lime Stabilisation**

Mitchell, in his 1986 Toshioji lecture, emphasised the importance of a full assessment of the ground chemistry when considering lime stabilisation. He highlighted the early work of Dumbelton (1962) and Sherwood (1962) and regretted that more attention had not been paid to their research. Unfortunately, even following that lecture, too little attention has been given to how the ground is assessed before lime stabilisation is specified. Despite the case studies reported by Mitchell (1966) and Hunter (1988), in August 1989 lime stabilisation was used for part of the M 40 in Oxfordshire, UK. At that time the Highways Specification stated that where more than 1% SS3 was present, lime stabilisation was inappropriate. However, no details were given of how frequently the ground should be tested, either vertically or horizontally, nor how to assess the likelihood that changes would occur if excavation was undertaken, the ground water level lowered and the fresh mudrocks allowed to erode.

In April 1990 it was noted that the profile of the southbound carriageway over a section of road in cut did not conform to the design. It was apparent that the highway was rising in three areas; the maximum potential heave being in the order of 150 mm. Table I reports the heave as it occurred over a three year period as a percentage of the final total. It is

<table>
<thead>
<tr>
<th>Period</th>
<th>Heave (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sept 1989 to Jan 1990</td>
<td>35</td>
</tr>
<tr>
<td>Jan 1990 to May 1990</td>
<td>25</td>
</tr>
<tr>
<td>May 1990 to Sept 1990</td>
<td>15</td>
</tr>
<tr>
<td>Sept 1990 to Jan 1991</td>
<td>10</td>
</tr>
<tr>
<td>Jan 1991 to May 1991</td>
<td>6</td>
</tr>
<tr>
<td>May 1991 to Sept 1991</td>
<td>4</td>
</tr>
<tr>
<td>Sept 1991 to Jan 1992</td>
<td>3</td>
</tr>
<tr>
<td>Jan 1992 to May 1992</td>
<td>2</td>
</tr>
<tr>
<td>May 1992 to Sept 1992</td>
<td>2</td>
</tr>
</tbody>
</table>

Table I: Percentage of heave experienced in a three year period relative to that measured in April 1992 (after Hawkins, 1997; data from Snedeker 1996).
noted that approximately 50 mm occurred between September 1989 and January 1990 and most of the heave took place within the first fifteen months after lime "stabilisation". Fortunately, in this case, the problems manifested themselves before the cutaway was opened.

Snedker (1996) notes that the site investigation for this motorway had taken place over a period of 18 years, with five separate exploratory contracts providing some 140 total sulphate content analyses. These have been plotted in Figure 3 and indicate that 90% of the total tests had a total sulphate content of less than 0.6%, the maximum measured being 3%. When further tests were taken at the time of the heave, only a quarter of the results were less than 0.6% while the highest value determined was 7.7%.

The trial pits undertaken following the recognition of the heave recorded a thickness of the stabilised zone significantly greater than had been specified. X-ray diffraction analysis on samples taken from these trial pits proved the presence of ettringite. This mineral occurs mainly as fibrous crystals which have sufficient strength to separate the particles and weaken the physico-chemical bonding of the clays. Once the crystals have begun to form, the mineral continues to grow as long as the necessary conditions persist. Thaumasite was identified in later analyses and hence this failure is very similar to that reported by Mitchell (1986). In his Las Vegas example, the distress appeared some two years after construction and by the spring of 1978 the heave had amounted to "several inches".

When the presence of significant amounts of ettringite and thaumasite was appreciated, Mitchell recommended "if sulphates are present in a soil, then the use of lime or Portland cement as admixture stabilisers should be approached with great caution, and, in many cases, may have to be avoided altogether." He regretted that insufficient attention had been given to the early work of staff at the British Transport and Road Research Laboratory and highlighted the importance of a full understanding of the ground conditions if lime stabilisation is to be used. Mitchell also drew attention to the fact that satisfactory test results shortly after lime treatment do not guarantee satisfactory performance over a long period.

ASSESSMENT OF A GREEN FIELD SITE

Sampling

The geotechnical engineer should always carefully consider where samples are taken to ensure that the most representative data are obtained and that they are relevant to the specific project. It is the writer's impression that insufficient attention is given to this; the samples frequently being taken in a very random manner with no apparent relevance to either the specific ground conditions or the proposed engineering works.

Figure 4 presents an idealised trial pit, showing the characteristics of the aerated zone, water saturated zone and the zone in which the main seasonal change in ground water level occurs. Above the upper evaporation front where oxidation may take place throughout the year, most of the pyrite has been oxidised and removed and the caliche has leached; although towards the base a zone of concretions may have developed. Below this, in the water saturated zone where the ground water levels vary seasonally, the pyrite may have been partially oxidised and caliche is

![Figure 3: Sulphate analyses obtained pre-construction (A) and post-construction (B) for the lime stabilised Lias Clay on the M-40 motorway (after Snedker, 1996).](image)

![Figure 4: Idealised trial pit with information relevant to sample collection (after Hawkins, 1997).](image)
generally present. Sulphates frequently occur at this level as a result of chemical interaction. The depth of the aerated zone will depend in part on the topography. Where the ground is relatively flat the aerated zone may be only some 0.5 to 1 m deep, while on a sloping site or road cutting the ground water level may be lower in the upper part of the site and higher downslope. The presence of more highly permeable interbeds of limestones or sandstones may also affect the ground water regime, complicating the typical conditions.

It will be appreciated, therefore, that the amount of sulphates recorded in a chemical testing programme will depend on the levels at which the samples are taken relative to the ground water regime. In the idealised trial pit in Figure 4, Samples 1 to 4 and 11 and 12 will have a low sulphate content while the maximum sulphates are likely to be recorded in Samples 7 to 9. Engineers need to take due cognisance of this variation with depth and to consider the purpose of the testing:

a) Is it to assess only the effect of sulphate attack on concrete placed in the ground?
b) Is it to assess potential sulphate leaching?
c) Is it to assess the possible implications of using lime or other chemical stabilisation methods?

They will also need to establish whether the proposed engineering works will modify the ground water regime under ground temperature. This could induce further chemical reactions which may change the long-term ground conditions, possibly affecting the integrity of the structure.

Without appropriate sampling, it is quite possible that the data of laboratory results produced will be of little value for the particular project and may actually be misleading. If the new construction will be affected only by the conditions in the aerated zone, the importance of the sulphate content and pH may be relatively small. However, if either buried concrete or pipelines are to extend into the motilled zone, the existing and potential ground chemistry will have a much more significant effect and hence it is particularly important to ensure appropriate sampling is undertaken to assess both the present and future conditions. Should the structure extend into the saturated zone and the ground water regime be modified such that these relatively fresh materials become aerated, again it is essential that the potential for the development of ground sulphates is explored.

Figure 5 shows a profile of total acid soluble sulphate content with depth through a trial pit in the Lower Lias Clay near Gloucester, UK. It will be noted that there is very little SO4 in the leached zone above 1.2 m while at approximately 1.5 m there is a pronounced peak extending to more than 7%. Examination of the in situ material indicated this peak was caused by the presence of sucroce gypsum while the more variable sulphate percentages below 1.6 m were produced by a mixture of sucroce and fine crystals; the proportion of sucroce gypsum decreasing with depth. Crystals of up to 30 mm in size were seen at the bottom of the trial pit.

Testing

Many authorities and Codes of Practice, such as the British Building Research Establishment (BRE Digest 363:1996) advocate the testing of ground waters. In an inert situation, it is the mobility of the sulphates present in the ground which determines whether they will penetrate into the concrete and have a deleterious effect. The assessment of the salt concentration within the ground water is therefore clearly important if concrete aggressivity is the prime consideration. However, it must be remembered that the concentration of sulphates within the ground water will be influenced by the solubility of existing sulphates and hence will undoubtedly change seasonally. Consequently the analysis of ground water samples taken on one specific occasion may not indicate the composition of the ground water during other seasons, nor the increase in available sulphates which may occur as a result of chemical reactions.

It is the writer's experience that reports in which ground water analyses are provided rarely give any indication of the sampling conditions - if the material has been flushed by the drilling medium or faster ground water movement following a wet period or whether the water has been relatively static for some time prior to the test being undertaken.

Ground water analyses provide little information regarding the likelihood of sulphate leaching. If this is the purpose of the testing, it is important not only that the total acid soluble sulphate within the soil mass is determined, but also that an assessment is made of the potential for future development when the environmental conditions are modified. Such further development can sometimes be seen in the field where dark grey, pyritiforous, weakly calcareous mudrocks which have been exposed for one to three months show a speckling of small white crystals. It may also become apparent in the laboratory, as was noted by Hawkins and
Table II: Changes in soil chemistry between November 1982 and March 1984 for samples from Llandough Hospital, Cardiff.

<table>
<thead>
<tr>
<th></th>
<th>WEST PIT</th>
<th>INTERIOR PIT</th>
<th>EAST PIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>SO₂ (%)</td>
<td>Nov 1982</td>
<td>0.29</td>
<td>2.30</td>
</tr>
<tr>
<td>SO₂ (%)</td>
<td>March 1984</td>
<td>1.20</td>
<td>4.15</td>
</tr>
<tr>
<td>pH</td>
<td>Nov 1982</td>
<td>7.10</td>
<td>3.75</td>
</tr>
<tr>
<td>pH</td>
<td>March 1984</td>
<td>6.25</td>
<td>2.65</td>
</tr>
</tbody>
</table>

Pinches (1987a) when samples were kept in an aerated condition. A study of the bacteria both inside and outside of the block confirmed their proliferation in the warmer conditions beneath the ground heating floor slab, with the maximum numbers noted just above the ground water level.

In view of the fact that Thiobacillus proliferate more readily in warmer temperatures, Hawkins and Pinches undertook controlled experiments to assess the influence of temperature on sulphate development. Figure 7 indicates that even for the creation of normal house temperatures, say in the upper 20°C, the heat from an underfloor system may be sufficient to result in the SO₃ content more than doubling.

It is recommended that where major engineering works are to be constructed in material containing pyrite, the site investigation should consider:

a) establishing the total amount of iron sulphide and calcite present in the ground - to determine the percentage of those minerals which are likely to interact if conducive environmental conditions are created;
b) establishing the total acid soluble sulphate existing at the time of sampling and retaining some of the sample from each test for re-analysis after a period of storage in aerated conditions. Four months should be sufficient to indicate whether significant sulphate development is likely to occur if fresh saturated ground becomes aerated.

Figure 6: Diagrammatic section through the structure of the ward block, Llandough Hospital, Cardiff.

Figure 7: Change in SO₃ content when dark mudocks were placed in ovens at different temperatures over periods of up to 15 weeks.
Further research by Hawkins and Pinches included experiments which indicated that reactions may be accelerated in the laboratory by continually wetting a finely broken sample stored at 40°C or by placing the broken sample in a relative humidity oven at a similar temperature. From these experiments they considered that if no obvious change occurs after two and a half months, it is unlikely further sulphate generation will be a problem.

If the use of lime stabilisation is being considered in pyrite-rich mudrocks, it is essential that tests with various admixtures of lime are undertaken in different temperatures and with different moisture contents to determine whether ettringite/lime/montmorillonite is formed. Hunter (1988) provides a useful summary of the chemical reactions which take place. If possible, the expansion created by the growth of these minerals should be determined during the tests.

Table II indicates that changes in pH occur associated with the development of ground sulphates. This is mainly due to the sulphuric acid produced by the bacteria which not only increases the acidity of the ground but by doing so, makes it an environment more conducive to their proliferation. In view of this "chain reaction", only a small proportion of calcium carbonate is required in the host material in order to result in the creation of the maximum acid soluble sulphates. If large quantities of calcium carbonate are present, the impact of the bacterial activity will be less, hence the soil will not become as acidic and although the bacterial/chemical reaction will continue, it is likely to take place at a slower rate. The best estimation of the potential sulphate content therefore requires not only a determination of available pyrite but also an assessment of the presence of other minerals with which it may react, such as calcite.

Many Standards draw attention to the importance of acidic ground conditions for engineering works, frequently quoting a pH of 5.5 at the threshold below which special consideration is required, eg an assessment of the relative mobility of the ground waters. For pipelines, modern practice is to surround the pipes in a free-draining aggregate, such that adjacent ground water levels may be lowered and previously saturated ground become aerated. Such conditions will facilitate the development of sulphates and an associated reduction in pH. Where the ground water flow is continuous, salts may be leached from that area and hence aggressive conditions may not arise. In flat or low-lying areas where the ground water is relatively static, however, the salts may accumulate such that the conditions become more aggressive than those established at the time of the field investigation.

**DISCUSSION**

Much has been written in the last decade concerning the importance of sulphate salts in concrete attack. It has been advised in such documents as BRE Digest 363 and some recent British Standards that the sulphate content is best assessed from ground waters. BRE Digest 363.1996 comments that the sulphates measured in the ground water probably closely resemble those in the pure water and hence by obtaining results on such liquids, a fair indication of the concentration of aggressive salts can be obtained. Comments are made on the importance of the mobility of the salts contained within the ground water and the opinion expressed that determining acid soluble sulphates from soil samples may result in higher values and hence may exaggerate the potential deleterious effects for buried concrete.

However, there is a marked deficiency in the literature concerning the dynamic nature of ground chemistry and the way in which man's activities may induce changes by the modification of such environmental conditions as ground water level or ground temperature. Any engineer taking responsibility for a construction scheme should ensure he obtains information not only on the presence of sulphates prior to the works, but also on the likely values following completion.

In order to assess the potential increase of sulphates, Hawkins and Pinches (1987b) constructed an experiment using five polystyrene cups stacked one above the other with the lower cup in a water bath. As shown in Table III and Figure 8, the sulphate content of the soil samples increased rapidly such that after only two and a half months the original value had increased three-fold. The table also provides the analysis of the liquid in the water bath, which again showed a marked increase in SO\(_3\) concentration, although from the relatively few measurements a distinct linear trend cannot be assumed. A similar increase in sulphate content also occurred when the

| Table III: Increase in SO\(_3\) in samples placed in a stack (18-20°C) from an initial proportion of 0.53%.

<table>
<thead>
<tr>
<th>Stack position</th>
<th>7 days</th>
<th>14 days</th>
<th>28 days</th>
<th>74 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (top)</td>
<td>0.55</td>
<td>0.93</td>
<td>1.03</td>
<td>1.06</td>
</tr>
<tr>
<td>4 (top)</td>
<td>0.69</td>
<td>0.69</td>
<td>1.11</td>
<td>1.10</td>
</tr>
<tr>
<td>3</td>
<td>0.66</td>
<td>0.78</td>
<td>1.12</td>
<td>1.83</td>
</tr>
<tr>
<td>2</td>
<td>0.70</td>
<td>0.86</td>
<td>1.06</td>
<td>1.89</td>
</tr>
<tr>
<td>1 (bottom)</td>
<td>0.53</td>
<td>0.54</td>
<td>0.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Mean</td>
<td>0.64</td>
<td>0.80</td>
<td>0.91</td>
<td>1.85</td>
</tr>
<tr>
<td>Water (ppm)</td>
<td>186</td>
<td>242</td>
<td>312</td>
<td>285</td>
</tr>
</tbody>
</table>

![Figure 8: Change in SO\(_3\) in Wessex soil samples stacked in moist aerated conditions over 74 days.](image)

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mudrock samples were heated (Figure 7); significant amounts of sulphate developing within a few weeks in a temperature of 20 to 25°C and saturated conditions.

The writer does not support the oversimplified comments in BRE Digest 363 (1996) that "Suitable soil samples may be obtained from the test boreholes made for engineering purposes. They should be taken every 1 to 2 m and wherever an obvious change in stratum occurs. Economic considerations will govern the number of soil samples analysed." This advice appears to take no cognisance of the specific ground conditions and to assume that the sulphate content is related simply to the generalised nature of the individual stratum layers with little account taken of the actual weathering, oxidation and ground water conditions, which will vary in different environmental situations.

BRE advocate that the sulphate and pH testing is undertaken as quickly as possible after the sample has been obtained. Samples put aside for laboratory testing should be retained in airtight containers in a cool temperature. In practice however, soils samples are still commonly kept in normal laboratory temperatures until the depth of the required test is specified, frequently not until some weeks later. As a consequence, particularly if the sealing of sample bags or jars is not adequate, some aeration takes place encouraging faster decomposition of any contained pyrite and an accelerated growth of calcium sulphate within the sample (Hawkins and Pinches, 1986). If the purpose of the testing is to assess the long term potential for sulphate development in changed ground conditions, this is valuable information. However, if it is assumed the results from partially aerated samples represent the present ground conditions, they are likely to be misleading.

CONCLUSIONS

Too often engineers specify chemical tests without a proper understanding of the significance of these and the possible changes in ground chemistry which may occur with time and as a consequence of the engineering construction.

It is recommended that consideration is given not only to concrete aggressivity when a determination of the ground water chemistry may be applicable, but also to the potential for heave if sulphates develop in the changed environmental conditions created by the engineering works. In this case, analysis of soil samples is essential, including both a determination of the existing iron sulphides and calcium carbonates and an assessment of the potential for future sulphate generation.

Attention should also be given to the dynamic nature of the ground chemistry when programmes for lime stabilisation are planned.

Appropriate consideration can only be given if the sampling and testing is undertaken with an appreciation of the susceptible strata, the depths at which chemical interactions are most likely to be initiated and the implications of these for the specific works envisaged.
Tracer methods for groundwater flow and pollution transport characterization

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ABSTRACT: One-borehole and multi-borehole tracer methods enable to determine important characteristics of groundwater flow and contaminant transport in porous or fissure media. These methods are used in Slovak republic to solve important hydrogeological survey topics for huge structures, to monitor seepage through dam, levee and weir subsurfs, to check deep excavation scaling element efficiency, to protect water resources in a dangerous construction, oil or radioactive contamination nearness. Apart from information of methodological character the paper gives particular examples and results from contaminant transport studies.

1. INTRODUCTION

International conferences regularly organised since 1963 by International Atomic Energy Agency residing in Vienna have had a great importance for tracer methods development. Especially the problems connected with artificial and natural radioactive substances have been proceeded there. Important conferences have also been organised since 1966 by International Association of Tracer Hydrology.

The paper deals with only those methods which are most often utilised in the conditions of the Slovak republic. These are the one-borehole methods enabling to determine filtration velocities, permeability coefficients based on monitoring of electrolyte solution vertical motion or its dilution process. Flow directions, water velocities, dispersion coefficients or dispersivities are determined by multi-borehole methods.

2. ONE-BOREHOLE METHODS

These methods require a perforated monitoring tube with inner diameter of 60 to 150 mm with a filtration fill at a given depth. Measurement results are representative for the nearest borehole surrounding given by a multiple of its diameter.

Deeper boreholes connect different pressure horizons and vertical water flow takes place there almost in all cases. Less intensive vertical flow emerges due to non-uniform temperature distribution and because of the fact that borehole does not run along an equipotential line.

Interesting papers published by Halevy, Moser, Zellhofer & Zuber (1967), Drost (1970) and others.

2.1 Vertical flow measurement

An equipment set schematically depicted in Fig. 1 can be used for vertical water flow measurement in a borehole. An immersion probe is put into the borehole. The probe is connected to battery-charged measurement devices with tracer (NaCl solution) jet control panel positioned on the surface. Concentration curves can be directly seen on the computer screen and the maximum concentration time \( t_{\text{max}} \) can be determined. The design time \( t_d \) was determined from the widespread laboratory research by calibration:

\[
 t_d = 0.266 t_{\text{max}}^{1.04} \tag{1}
\]
Figure 1. An equipment set for vertical water motion measurement in a borehole: d - perforated tube inner diameter, \( L_e \) - gauge distance, \( c \) - concentration, \( t \) - time, 1-3 immersion probe (1 - tracer jet, 2 - upward flow gauge, 3 - downward flow gauge), 4 - connection cable and solution supply, 5 - tracer jet control, 6 - computer converter, 7 - portable computer

This value is to be used for average vertical velocity value estimation:

\[ v_v = \frac{L_e}{t} \]  

(2)

(\( L_e \) = path given by tracer jet and respective gauge, approximately 0.5 m).

Vertical discharge can be determined from the continuity equation:

\[ q_v = v_v \cdot A = v_v \cdot \frac{\pi (d^2 - d_i^2)}{4} \]  

(3)

(\( \Lambda \) = observation tube discharge lateral section area; \( d \) = inner diameter; \( d_i \) = outer immersion probe diameter).

Measurements are repeated in suitable intervals in order that all the saturated permeable parts of the borehole are uniformly covered by results and in order that depth vertical discharge relation can be plotted.

Filtration velocity in the surrounding medium (approximately in the horizontal direction) is calculated from the results of vertical water flow in vertical borehole using the formula:

\[ v_f = \frac{\Delta Q}{\bar{a} \cdot \Delta h} \]  

(4)

where \( \Delta Q \) = vertical discharge increase or decrease in the part of the borehole with height \( \Delta h \); \( \bar{a} \) = coefficient of drainage borehole influence at vertical flow (approximately \( \bar{a} = 20 \)); \( d \) = observation tube inner diameter.

Filtration velocity calculations using equations (1) to (4) are made with personal computers, results are plotted as functions of depth. Average filtration velocity for the respective borehole is determined using the formula:

\[ v_f = \frac{\sum v_f \cdot \Delta h}{\sum \Delta h} \]  

(5)

and in the situation is usually depicted as a vector. Permeability coefficient stems from Darcy’s law.

More intensive vertical flow in a borehole can be better measured with adjusted hydrometric flaps.

2.2 Dilation method

Dilation method is used for observation objects with low water column. The indicator (NaCl) is usually being introduced to water in a powdered status. Immersion electrode probe and simple battery conductometric device are used for process observation. Filtration velocity is estimated using the formula:

\[ v_f = \frac{\pi d}{4 \alpha t} \ln \frac{c_n - c_v}{c - c_v} \]  

(6)

where \( d \) = observation tube inner diameter; \( \alpha \) = borehole drainage influence coefficient for dilation method (\( \alpha = 2 \)); \( c_v \) = initial concentration; \( c \) = concentration at time \( t \); \( c_n \) = natural concentration.)
Formula (6) anticipates that the solution dilution is caused by water flowing perpendicularly to the borehole axis. If flow directed parallelly with the borehole axis takes place the basic assumptions of the interpretation formula are not fulfilled and such results cannot be used. In order to eliminate the vertical flow there are devices available at some departments which protect the measurement space against vertical flow with infeasible seals (Drost 1970).

3. MULTI-BOREHOLE METHODS

These methods were originally only used to determine water flow continuity, groundwater flow directions and water velocities. Nowadays these methods are more often used to determine contaminant transport characteristics.

3.1 Flow directions and water velocities

In situ experiments are held with the aid of two boreholes (minimum) which are to be built in the direction of groundwater flow (Fig. 2). Tracer solution is introduced into the injection borehole and its transmission is watched in the observation borehole. Several boreholes are suitable to be built and observed in the anticipated flow direction. The flow direction is given by the connecting line of the injection borehole and that borehole for which maximum water velocity is estimated.

Water velocity

\[ v = \frac{x}{t_{\text{max}}} \]  

\( (x = \text{distance}; \ t_{\text{max}} = \text{maximum concentration time}). \)

3.2 Transmission characteristics

Laboratory experiments in order to get transmission characteristics are often held using tubes with soil through which water flows (D.Klotz & H.Moer 1974). A tracer or a contaminated solution are introduced to the surface of the soil fill as a single-shot in a form of a thin film, or continually.

In the conditions above one-dimensional dispersion takes place. Such dispersion with a single-shot conservative tracer introduction is described by Bear (1972), Lenda & Zuber (1970). From their equations the longitudinal dispersivity

\[ \alpha_l = \frac{1}{\pi x} \left( \frac{c_j \sigma_j}{2c_m \sigma_j} \right)^{1/2} \]  

\( (8) \)

In the equation (8) \( x = \text{distance}; \ c_j = \text{initial concentration; \ } V_j = \text{initial volume; \ } c_{\text{max}} = \text{maximum concentration; \ } A = \text{discharge area; \ } n_d = \text{effective porosity.} \)

For nonstable tracers or contaminants liable to processes of decay (radioactive), degradation (organic), which can possibly be adsorbed to solid soil particles, the transmission equation can be adjusted as follows:

\[ \frac{c_{\text{max}}}{c_i} = \frac{V_j}{2A_0 \sigma_j} \exp \left( -0.693 \frac{v^2}{D} \right) \]  

\( (9) \)

Apart from the symbols already known \( K = \text{correction factor}; \ R = \text{retardation factor; } T = \text{half-life period. When taking adsorption into account it is assumed that motion of such solutions is slower than that of clean water.} \)

Two-dimensional transmission tracer process takes place at one-dimensional groundwater flow in conditions of in-situ multi-borehole experiments.
For the boreholes being in the streamline the two-dimensional dispersivity can be simplified by a one-dimensional one (Fig. 2). The transmission angle $\beta$ (about $35^\circ$) is known from the past experiments in gravel soils. The initial volume of contaminants ($V_{i}$) can be expressed:

$$V_{i} = Lbh$$

where $b = x \tan \frac{\beta}{2} \approx 0.63x \quad (10)$

and formula (9) can be used.

Dependence of dispersivity on the flow trace in gravel soils has distinctly showed off in complex in-situ experiment results available summary which have been obtained and published in our country and abroad. For the dependence mentioned an equation has been derived:

$$\alpha_{x} = \exp \left( \sqrt{12515} - \ln x - 12515 \right)^{2} - 4.77 \right) \quad (11)$$

($\alpha_{x}$ and $x$ is introduced in m).

4. EXAMPLE OF PRACTICAL UTILISATION - RADIOACTIVE CONTAMINATION

Three reactors are placed in the area of the nuclear power stations of Jasslovske Bohunice. First of them was shut down because of a failure in 1977. Radioactive substances, especially tritium have leaked into the groundwater.

Surface layers are formed by loess with thickness ranging between 2 and 20 meters, below there is a saturated layer of gravel soils with thickness of 15 to 20 meters, deeper strata are formed by clay. The groundwater table in the area of the power plant is about 20 meters below the surface.

A monitoring system is built in the surrounding of each power station inside as well as outside the area. It is being gradually expanded, nowadays it is formed by 144 boreholes (some of them are in Fig. 3). Almost all the boreholes can be used to pump out the contaminated water if necessary. Hydrogeological and radiation situation is being regularly monitored. Punctual tests and tracer experiments took place in many boreholes.

Figure 3 depicts average filtration velocity vectors determined with formulas (4) and (5). Their values range between $10^{-6}$ and $2 \times 10^{-3}$ m s$^{-1}$.

For each borehole by means of tracer method the permeability coefficient from Darcy’s law was estimated ($k_{p}$). Except for these results, we used the permeability coefficients from the pumping tests ($k_{p}$) and from the grain analysis according to Beyer-Schweiger ($k_{p}$) and Carman-Kozeny ($k_{p}$) formulas. Statistical analysis results for the tested area are shown in Fig. 4. It can be noted that, coefficients of permeability $k_{p}$ are generally less than $k_{p}$. The median and average value from vertical movement tracer method is lower. It can be clearly seen that pumping tests ($k_{p} = 4.6 \times 10^{-3}$ m s$^{-1}$), vertical flow tracer measurement medium ($k_{p} = 5 \times 10^{-3}$ m s$^{-1}$), and Beyer-Schweiger estimated medium ($k_{p} = 3.7 \times 10^{-3}$ m s$^{-1}$) give comparatively the same results.

The reasons of the results in details are evident. Grain size analysis arise from disturbed samples in which the sand particles are missing.
due to careless quartering. One-borehole tracer methods give results representative for the nearest surrounding of each borehole. However, activities connected with direct or transmitted supply of different layers can take place there. Pumping tests pump water from a wider surrounding, so the permeability coefficient has to be different from grain size and one-borehole methods. Bigger differences, however, do not take place at statistical evaluation of the representative files.

An experimental system for determination of contaminant radioactive substances transamination has been built in the area of the nuclear power stations. This system consists of nine boreholes and enables to observe the transmission of the substances in natural regime as well as artificial regimes influenced by pumping or water injection (Fig. 5).

The basic characteristic for the transmission in a saturated zone is dispersivity ($\alpha_x$). In the area of the power stations it has been determined with the first communication experiment under natural conditions of groundwater flow. The second experiment was held in 1996 with a forced flow. During the experiment an amount of clean water $q = 0.004$ m$^3$s$^{-1}$ was permanently injected into the N-2 borehole. Hydroisohyphes for the conditions mentioned are depicted in Figure 5.

Water solution of NaCl was introduced into the N-2 borehole. The changes of electric conductivities and concentrations were observed in the N-2 and the surrounding boreholes. The NaCl solution was flowed out of the borehole into the surrounding gravel soil at a depth of 21 to 31 meters approximately in the horizontal direction. Deeper it was transmitted in the borehole vertically and flowed out into the surrounding soil at its lower part.

In all the boreholes the most suitable positions for quartering were determined based on vertical water flow measurements. These positions are characterized by the most intensive water influxes into the borehole from the soil ($\alpha_x > \alpha_0$ max).

Groundwater flow velocities within this relatively small experimental area ranged relatively widely, i.e. between $9 \times 10^{-4}$ and $1.9 \times 10^{-3}$ m$s^{-1}$. Our results for longitudinal dispersivity fully respected the equation (11). Measured experimental values and functions calculated with different data are depicted in a form suitable for the transmission prognosis in Fig. 6. They are valid for impulse contamination and simplified one-dimensional transmission. The results best for practical purposes are depicted with a full line. The outcomes of substituting two-dimensional transmission with a one-dimensional one are surely smaller than the outcomes of substituting partially continual contamination with a impulse.
weapon tests in 1963 reached the value of 270 Bq dm$^{-3}$. Neither the present radioactive groundwater contamination in the area of the power stations nor its indirectly modeled development prognosis for the future can threaten people’s health. Despite of all this clean water supply is provided centrally from absolutely non-defective resources.

5. CONCLUSIONS

One-borehole and multi-borehole methods have successfully developed over the recent years. They have reached the level enabling their use for the solution of wide range of practical problems. They are suitable especially for determination of characteristics of groundwater and sewage water transmission as well as characteristics of contaminant substance transmission by groundwater in porous and fissure media. The important advantage of these methods is the possibility to follow time development of the respective characteristics under the natural conditions or under conditions influenced by technical remedies. Particular example documents the results which depict the conditions under which radioactive substances contaminating groundwater are transmitted into the surrounding of the nuclear power station.

REFERENCES


Site characterization performed to support a municipal solid waste environmental recuperation in Recife, Brazil

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ABSTRACT: This paper describes the site characterization performed in the largest municipal solid waste landfill of Recife, the Muribeça Landfill. The geological and geotechnical studies includes morphological and hydrogeological studies, solid waste property assessment and the monitoring of ground and surface water.

1 INTRODUCTION

The Muribeça Landfill is the largest solid waste landfill of the Metropolitan Region of Recife on the Northeast coast of Brazil. It receives about 2500 tons of domestic, industrial and hospital wastes per day, which are deposited directly on soil. Therefore, there was no area any longer available. The adopted solution was the treatment of the landfill.

This landfill is a solid waste disposal unit since 1985. In the last 12 years about 6,000,000 tons of domestic and industrial solid waste were disposed directly over soil surface on a 60ha area with an average height of 15m. The Muribeça's waste composition is shown in Figure 1.

Three years ago Recife City government has started a program to recuperate Muribeça Landfill area. This program includes construction of cells for waste confinement (Figure 2), preliminary studies of polluted areas and subsequent treatment of them, and the assessment of some waste properties. A multidisciplinary research group from Federal University of Pernambuco performed these studies.

This paper deal with the geotechnical studies that are being carried out in this landfill and the conclusions related to this subject.

2 PRELIMINARY STUDIES

The local morphology of Muribeça's area was prospected by Global Position System method. Tridimensional coordinates of several points along the landfill were determined and the elevations ranged from 10m up to 10m.

Many types of sediments compose the site geology. Granite pre-Cambrian rocks outcrop in the area, over which there are non-consolidated quaternary sediments and sandy-clayey sediments of Barreira Formation.

The hydrogeological properties of Muribeça is shown in Table 1.

![Figure 1. Muribeça's waste composition](image1)

![Figure 2. The waste confining cells of Muribeça Landfill](image2)
Table 1. Hydrogeological properties of Muribeca

<table>
<thead>
<tr>
<th>Material</th>
<th>Formation</th>
<th>Aquifer Type</th>
<th>Flow Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediments</td>
<td>Alluvial deposits</td>
<td>Phreatic water</td>
<td>High permeability</td>
</tr>
<tr>
<td>Sediments</td>
<td>Basin floor</td>
<td>Phreatic Creep water</td>
<td>Low permeability</td>
</tr>
<tr>
<td>Bedrock</td>
<td>Basement Shield</td>
<td>Low permeability</td>
<td></td>
</tr>
</tbody>
</table>

The climate is classified as warm and humid. The average temperature and humidity ranges from 23° to 26°C and from 70 to 80%, respectively. Rainfall is concentrated from February to July and the average annual rainfall is around 1,750 mm. The monthly evapotranspiration ranges from 100 mm to 190 mm.

3 MONITORING OF GROUND AND SURFACE WATER

The monitoring process that is in currently operation, comprehends monitoring of rivers and fonts existents in the area (Figure 3). Chemical and bacteriological analysis are performed by samples collecting on and at the vicinity of the landfill. Sampling points were chosen in order to cover ground and surface water. For ground water monitoring, 4 m depth wells were excavated downstream from the landfill. Other points were domestic wells MW 8 and MW 9. The ground water analyses are shown in Table 2. Since Muribeca Landfill's soil is formed by fine soil with low permeability, the fluid flows only superficially and does not reach the deeper water. The surface water analyses were carried out in

Table 2. Analyses in monitoring wells

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Month</th>
<th>MW8</th>
<th>MW9</th>
<th>MW8</th>
<th>MW9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ph (mg/L)</td>
<td>Jun</td>
<td>5.7</td>
<td>6.5</td>
<td>5.9</td>
<td>5.9</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>-</td>
<td>6.4</td>
<td>6.4</td>
<td>6.2</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>5.5</td>
<td>6.2</td>
<td>6.1</td>
<td>6.1</td>
</tr>
<tr>
<td>COD (mg/L)</td>
<td>Jun</td>
<td>2</td>
<td>5.6</td>
<td>18.7</td>
<td>40.4</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>-</td>
<td>7</td>
<td>10.6</td>
<td>13.1</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>31</td>
<td>34.9</td>
<td>45</td>
<td>25</td>
</tr>
<tr>
<td>BOD (mg/L)</td>
<td>Jun</td>
<td>0</td>
<td>2.5</td>
<td>1.9</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>-</td>
<td>1.6</td>
<td>1.2</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>2.3</td>
<td>1.6</td>
<td>0</td>
<td>2.6</td>
</tr>
<tr>
<td>Alkalinity (mg/LCO2)</td>
<td>Jun</td>
<td>18.9</td>
<td>34.9</td>
<td>44.9</td>
<td>40.9</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>-</td>
<td>37</td>
<td>55</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>16</td>
<td>48</td>
<td>56</td>
<td>35</td>
</tr>
<tr>
<td>Conductivity (µS/cm)</td>
<td>Jun</td>
<td>159</td>
<td>53.2</td>
<td>60</td>
<td>52.7</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>-</td>
<td>46</td>
<td>1372</td>
<td>79</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>150</td>
<td>106</td>
<td>84</td>
<td>49</td>
</tr>
</tbody>
</table>

Legend: MW = monitoring well

Figure 3. Map of Muribeca Landfill Area
Table 3: Surface water analyses

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Month</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>P4</th>
<th>P5</th>
<th>P6</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>Jan</td>
<td>6.6</td>
<td>7.8</td>
<td>7.2</td>
<td>6.4</td>
<td>7.4</td>
<td>7.6</td>
</tr>
<tr>
<td></td>
<td>Jul</td>
<td>6.6</td>
<td>8.3</td>
<td>7.3</td>
<td>6.6</td>
<td>7.7</td>
<td>7.7</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>6.5</td>
<td>7.1</td>
<td>7.1</td>
<td>6.6</td>
<td>7.6</td>
<td>7.6</td>
</tr>
<tr>
<td>COD (mg/l)</td>
<td>Jan</td>
<td>23.7</td>
<td>13.6</td>
<td>56.1</td>
<td>29.3</td>
<td>565.6</td>
<td>785.6</td>
</tr>
<tr>
<td>(influent)</td>
<td>Jul</td>
<td>9</td>
<td>2.333</td>
<td>111</td>
<td>9.6</td>
<td>285</td>
<td>285</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>24.1</td>
<td>1,366</td>
<td>42</td>
<td>19</td>
<td>285</td>
<td>285</td>
</tr>
<tr>
<td>BOD (mg/l)</td>
<td>Jan</td>
<td>1.7</td>
<td>1,200</td>
<td>23.4</td>
<td>1.4</td>
<td>354.6</td>
<td>354.6</td>
</tr>
<tr>
<td>(influent)</td>
<td>Jul</td>
<td>1.6</td>
<td>2,138</td>
<td>64</td>
<td>2.3</td>
<td>183</td>
<td>183</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>2.3</td>
<td>487</td>
<td>17</td>
<td>1</td>
<td>194</td>
<td>194</td>
</tr>
<tr>
<td>Alkalinity</td>
<td>Jan</td>
<td>9.9</td>
<td>2,765</td>
<td>191.8</td>
<td>18</td>
<td>692</td>
<td>692</td>
</tr>
<tr>
<td>(mg/l)</td>
<td>Jul</td>
<td>12</td>
<td>2,970</td>
<td>96</td>
<td>21</td>
<td>561</td>
<td>561</td>
</tr>
<tr>
<td>CaCO₃ (%)</td>
<td>Aug</td>
<td>8</td>
<td>2,999</td>
<td>110</td>
<td>22</td>
<td>688</td>
<td>688</td>
</tr>
<tr>
<td>Conductivity</td>
<td>Jan</td>
<td>21</td>
<td>8,200</td>
<td>327</td>
<td>75</td>
<td>2350</td>
<td>2350</td>
</tr>
<tr>
<td>(μS/m)</td>
<td>Jul</td>
<td>70</td>
<td>8,320</td>
<td>572</td>
<td>68</td>
<td>1370</td>
<td>1370</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>52</td>
<td>9,710</td>
<td>327</td>
<td>70</td>
<td>2610</td>
<td>2610</td>
</tr>
</tbody>
</table>

rivers (P1, P4, P5, P13, P14 and P21) and leachate streams (P2). The obtained results are shown in Table 3.

4.2 Assessment of some geotechnical properties

It is known that there are many geotechnical problems observed in solid waste landfills. The great compressibility, the strength parameters changing with time and the lack of preceding information related to bearing capacity are contributory factors in a modeling of solid waste behavior. Thus, in order to assess some of the solid waste features, was conducted a large research program comprising of field and laboratory tests on Murhbeca Landfill’s wastes. The field tests carried out for this research were plate load tests, cone penetration tests, standard penetration tests and in situ density measurement. Laboratory tests consisted of moisture and volatile solid content tests for solid wastes.

Four Plate Load Tests (PLT) were carried out on the cell. The PLT No. 1, 2 and 3 were performed on the soil cover layer, and the last PLT No. 4 was performed directly on the solid waste, without soil cover layer (see Figure 4).

A decrease of the settlement with the soil cover layer increase was observed.

The modulus of subgrade reaction (Ks) and Young’s modulus were obtained for different situations from the Plate Load Test (see Figure 5). The modulus of subgrade reaction was determined as the ratio of an applied pressure (150 kPa) to the settlement at that pressure after two to five minutes of load application. Landva and Clark (1990)

![Figure 4. Plate Load Test results](image)

![Figure 5. Plate Load Tests](image)
Table 4. Modulus of subgrade reaction

<table>
<thead>
<tr>
<th>Plate Load Test No.</th>
<th>$K_s$ (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (d = 0.46m, D = 0.80m)</td>
<td>9.90</td>
</tr>
<tr>
<td>2 (d = 0.46m, D = 0.60m)</td>
<td>3.70</td>
</tr>
<tr>
<td>3 (d = 0.45m, D = 0.80m)</td>
<td>7.93</td>
</tr>
<tr>
<td>4 (d = 0.50m, D = 0.80m)</td>
<td>1.72</td>
</tr>
</tbody>
</table>

$d$ = height excavation

$D$ = soil cover layer thickness

pointed out that lower values of $K_s$ (<2 MPa/m) are associated with poor compaction, while higher values (>5 MPa/m) correspond to a particularly good compaction, or to a better grade of fill or even to thicker, well compacted earth cover. The modulus of subgrade reaction of Muribeca is shown in Table 4.

Penetration tests as SPT and CPT were carried out, in spite of the considerable uncertainty in assigning material properties based on these type of tests, since there are no published correlation between their results and domestic solid waste properties. The three SPT tests carried out in Muribeca Landfill showed similar results.

Two of them were carried out two years before the other one, and their locations were closed. During execution of SPT, hydraulic conductivity $10^{-7}$ m/s was determined. Moisture and organic content were also determined.

Results from SPT show a typical range from 3 to 9 blows. This range is in agreement with literature, from 5 to 10 blows for this type of landfill (Figure 6). CPT showed a slight increase of resistance with depth. At the shallow depths, the resistance is due to the poor decomposition of the material. At greater depths, the resistance tends to stabilize and to increase a little bit due to upper layer pressure (Figure 6). The penetrometric driving was highly affected by the random presence of some stiff bodies leading to maximum strength values measured by the standard procedures. SPT and CPT results show a constant resistance trend with depth.

CPT results show that at the first 5m, where the deposited material is fresher, there is a large concentration of sharp resistance points at this depth. After 5m, the resistance tends to stabilize due to the greater decomposition stage. Obviously the few sharp points in CPT results are due to some stiff object different from the other ones.

In situ density of solid wastes in Muribeca Landfill was determined using the following procedure: first a well was excavated. Then this well was sealed off and the excavated material was weighted. After this, the well was filled out with water. The volume of water that filled the well

![Figure 6. Results from Plate Load and Cone Penetration tests](image-url)
was also measured. Finally, in situ density was obtained by the waste weight and measured water volume ratio. The Maribeca's waste in situ density ranged from 1400 to 1900 $\text{kg/m}^3$. The obtained densities were higher than the expected, because the excavated material was very confined due to the thick soil cover layer and due to the high load underwent because the compaction machinery.

5 CONCLUSIONS

- The geological and hydrogeological studies showed that the solid waste landfill is over a crystalline basement. Thus, the water analyses do not indicate water contamination of deep aquifers.
- Large differences in physical and chemical properties of soil, waste and leachate mean that a number of possible interections must be considered when evaluating both contamination and recuperation of the area.

6 ACKNOWLEDGMENTS

The studies described in this paper were performed by a multi-disciplinary group from Federal University of Pernambuco, with the support of CNPq and EMLURB/PCR. The authors are also indebted to Professors and Engineers J Cabral (Hydraulics), A.A. Santos (Cartography), E.Felitoa (Geophysics), S. Calado (Chemistry), A.Menela, A.E.C. Juca (Sanitary), M.O.H. Mariano, V.E.D. Monteiro, G.Perrier and V.A. Meio (Geotechnics), and to the students J.J. Juca and E.A Santos (CNPq/UFPF).

7 REFERENCES


On-line monitoring of NOx gases with sensors based on oscillating crystals

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P.B. Jovanić
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ABSTRACT: As a joint project for visual monitoring of possible ecological accidental situation, R&D department of the NIS (Oil Industry of Serbia) refinery is developing a group of sensors for on line monitoring of various gases which could influence the quality of air from the plant. In this brief report we presented a sensor for monitor NOx gases based on the crystal with constant frequency of oscillating. We use a polymer as a adsorption agent for NOx gases. Such system is connected to local network and or visualisation system. The research is now focused on developing adsorption agents for other gases which could be pollutants of air from the plant.

1. INTRODUCTION

As a joint project for visual monitoring of possible ecological accidental situation, R&D department of the NIS (Oil Industry of Serbia) refinery, is developing a group of sensors for on line monitoring of various gases, which could influence the quality of air from the plant. In this brief report we presented a sensor for monitor NOx gases based on the crystal with constant frequency of oscillating. We use a polymer as an adsorption agent for NOx gases.

The idea comes from the usage of oscillating crystals with constant frequency in various applications.1

Constant frequency could be changed on different ways, and one is by adding weight to the crystal. Measuring changing of oscillating frequency one could measure weight of added material. This fact we used for measuring concentration of gases.

2. EXPERIMENTS

We made experimental measuring device shown on figure 1. Standard 10MHz crystal was used for this assembly and for experiments we applied HP frequency meter. For measuring gas concentrations, we must have absorbing agents.

For a desorbing agent we used polymer. For this there are two reasons. One is that polymer could be easily added to the surface of crystal, and the other is, that time of usage of this kind of material

![Figure 1. Experimental setup](image-url)
could be controlled./2/

As gas is adsorbed on to polymer it rises the weight of the assembly. Additional weight lowers the oscillation frequency of crystal. Differences in frequency with and without adsorbed gas is measured with frequency counter. Counter is connected to the data acquisition system which could collect data and analyze them.

For analysis this system we used two types of measuring systems. First one was our frequency counter system and the other is classical system with pump absorbent and spectrophotometer. This measuring we used as a control system. Results are shown on Figure 2.

As could be seen on the figure 2. method with frequency counter shows little higher values for NOx concentrations, then method with spectrophotometer. Differences are shown on figure 3.

As we see from that figure error of measuring is in the acceptable ranges, and we think that this methodology could be applied for the field measuring.

We have more 120 sensors, which are working in various areas of the plant.

Mean working time of each sensor is approximately 120 hours, when they get saturated.

Mean error of measure is between 7-12 % which depend on the site of measure and velocity of saturation of polymer.
When the sensor is saturated with gas, it could be cleaned with oxygen stream. This takes about two hours.

For connecting sensors to the network, we used voltage to current converters. This allowed us to transfer signals from measuring site to acquisition site.

Sensors were placed on specific areas of the plant and connected to the measuring network.

Data were analyzed every one hour and displayed on the control monitor.

Example of a measuring circle is presented on figure 5.
System is designed for on line measuring so the idea was to make network of sensors, which could be placed around area of interest.

Problems which we find are related to the informations or data transfer to place of analysis, rather then accuracy of measurement of gas concentration.

Second problem is maintenance of instrument because sensors could be saturated by other gases or artifacts then NOx. This problem is partially solved by applying specially designed shields for crystals.

During experimental run average of 5 sensor were out of duty.

3. CONCLUSION

Results obtained with this methodology proved to be acceptable for monitoring NOx gases in plant.

The research is now focused on developing adsorption agents for other gases which could be effluents of air from the plant.

4. LITERATURE

1. Mitrović M., Trends in Chemical Engineering, Hemija i Industrija N°12, (1996),2-23, (in English)
Developing methodology for visualization of ecological accidental situations in plants

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ABSTRACT: Today's trend in analyzing possible accidental situations in plants is going towards on-line monitoring possible accidental sites in plants, and presenting obtained data visually. Acquired data are shared via network to all monitoring stations for various analysis and actions. In this paper we present one of the approaches for visual monitoring possible accidental sites in refinery plant. The idea was to divide whole plant into smaller groups of autonomous plants, and to observe possible accidental influences. We call those autonomous plants objects. Interactions between objects on the large scale, presents macro behavior of refinery plant as a subject of possible ecological influence to the environment. This approach is used for developing a decision-making system for ecological monitoring of refinery plant.

1. INTRODUCTION

Process engineers dealing with the refinery technology must today include one more aspect of their analysis—ecology. It is at present quite clear that all those who are concerned with the ecological analysis of plants must have readily available large amounts of quantitative information for making decisions or simply to monitor the plant. Today's blasting grow of the sensors for monitoring air, water and land pollution, allows the engineers to upgrade this monitoring systems into the higher level operational systems, known as expert systems, with the introduction of knowledge base (Ikegani, 1992). Designing the expert system is not an easy task in any field. Practical approach for advanced analyzing of collected ecological data is to design the decision making support system, which will supply engineer with enough valuable data for making decision's in accidental ecological cases.

Principial goal of this paper is to presents the basic concepts of our research in ecological analysis of refinery plant. Intention was to make software which will be support for the decision making system (DMS). Decision making system are on the "half way" to an expert systems which are the ultimate goal of every engineer. DMS systems in this case have the role to supply messages with enough informations that decision could be made with minimum possibility of errors. In our case we want to design a DMS system which could be useful in cases of ecological incidents in refinery plants.

Problems in design DMS system comes from their complex structure. They are not only factual data banks, but systems which includes several data banks, various data evaluation and simulations procedures together with adequate system for data presentation.

Schematic diagram of DMS is presented on Figure 1.

Each DMS system depends on factual data banks and their organization. Problems of
designing factual data banks are beyond the scope of this paper, but it must be pointed that if they are not designed properly the whole system could not work.

Trends in today's designs of process control are to produce visualization of every possible event during plant operation. It means that visualization of simulation of plant operation is easier to control complex plants, then looking onto monitors with tables and curves. Visualization of factual data or evaluated data could be done in several ways, which in some cases is dependent on system which is used for visual presentation. Today's computer and software technology allows multitasking options to processing systems, which in turn gives the designer more freedom in evaluating data presentation systems. At the end decision making is category of engineers and people who have experience in managing systems. If DMS system supply to them enough quality information the decision is not hard to make. If it is not the case everything could happen.

2. BASIC CONCEPT

As it was pointed out before we develop DMS system for evaluation of possible ecological accidental situations in the refinery plant (Oct., 1994). We used factual data banks which were developed during various projects in purpose to make models for optimization of plant operations.

Basically we take whole area of plant as possible ecologically dangerous object. Than whole plant was assumed to consists of objects. Object are individual operational plants in refinery, such as boiler houses crude units and etc. These object are presumed to be places of possible pollution, weather we are talking of air, water or trash. On refinery level from the point of view of ecology, objects interacts on various levels with each other. Visualization of all possible interactions of object in the plant could give picture of possible accident in the plant and prevent them. Schematically we can presents the refinery as a group of objects and various interactions which has influence on the environment, Figure 2.

This system of analyzing object and interactions is giving researcher the opportunity to make better models of plant operation, which leads to better quality of evaluating data. Using this approach we devise refinery plant in many small individual plants for which we can apply all possible technologies of data acquisition and manipulation, so that factual data banks of that object are of top quality. In DMS design this approach could be one of the solution for the first step designing of factual data banks.

On this place we have to define what we assume by the interaction between objects. Interaction in this case is a mathematical model which describes distribution of pollutants from the source. Simultaneously analysis of models of objects shows possible aggregations of pollutants on specified position in the plant. Designed system will be presented on the case of boiler house (Oct., 1996).

3. OIL REFINERY-BOILER HOUSE

Three boilers are installed in the boiler house. The boilers are fired typically with heavy fuel oil with 2.5-3.0% of sulfur. Burner designed value of sulfur is up to 5%. And the oil is preheated to proper temperature for kinematics viscosity of 25cst (centiStokes). Usual temperature of oil is about 110-130°C before entering the burners. The oil from the supplier is pumped into service tanks and the oil is preheated of steam. Alternatively the boilers could be run by gasoline, occasionally during the start up procedure. Each boiler is equipped with four pressure jet atomizing oil burner, with maximum capacity of each at 1300 kg/h for heavy fuel oil and 1272 kg/h for gasoline.

Measurements were done on the boiler house when boiler no 2 was working with heavy fuel oil and net caloric value of 41181 kJ/kg, with sulfur content in the fuel oil of 1.33%. Table 1 shows measured values.
### TABLE 1. Measured values

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</table>

Figure 3. Simulation of air pollution from the boiler house
Measurements were done in 5 to 10 minutes intervals. On basis of this measurements two simulations were done. First one is analysis of pollution in the plant and the other is pollution of the city area. Assumption is that only one boiler in the plant was working at that time. Simulation is presented on the Figure 3.

For this paper we want to show only the principals of designing DMS visual system. As simulation presents refinery plant has dangerous concentration of pollutants in area near the boiler house, if there is no wind.

This area is permanent. Situation change when wind blows, but does not change the hazardous area. That means that boiler house is one of the places in the plant where care must be taken. In this case filters are designed for boiler house.

Problem is variable content of sulfur in the oil, which keeps polluted area permanently on place.

CONCLUSION

This paper presented developed DMS system for monitoring and simulating possible ecological accidents in refinery plants (Jovaní, 1996). This system is now working only as a monitor of power plants in the refinery which are pointed to be the main source of pollution. This paper presents only monitoring of air pollution but monitoring water and land pollution is the same.

Factual data banks on power plant operation is acquired automatically with data acquisition systems, developed by the third party companies. Whole system is connected via network.

LITERATURE
1. Bečejski D. MSC Thesis, Belgrade University, 1992
2. Ocić O., PhD Thesis, Belgrade University, 1993
Establishment of regional soil gas background values to support waste site screening investigations

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ABSTRACT: In waste site characterization, shallow soil gas geochemical sampling procedures have evolved into a tool to rapidly and economically evaluate an area for the presence or absence of buried volatile organic contaminants. The method allows relative magnitudes of specific compounds to be identified. Regional soil gas and soil plug surveys were conducted over and adjacent to the US Department of Energy’s Savannah River Site between 1992 and 1996. The purpose of these surveys was to establish common background values for various organic hydrocarbon compounds for use in evaluating data collected during waste site screening. These programs involved 2800 soil gas and soil plug samples collected in three sampling events over an area of 2600 km². The conclusion reached from the latest two sampling events (2251 samples) suggest that regional background soil gas data need to be considered when evaluating the results obtained when applying soil gas techniques to the study of specific waste unit characterizations.

1 INTRODUCTION

Soil gas geochemical methods have been successfully applied in petroleum exploration since the early half of the 20th century to aid in the defining areas of hydrocarbon potential. In that industry, it was noted that surface seeps of oil and gas often led to the discovery of economic quantities of hydrocarbons in the subsurface. As the analytical techniques became more sensitive, it only followed that the use of the “macroseep” to screen areas evolved to include the “microseep”. The basic premise in applying surface seep detection to exploration is that volatile hydrocarbon materials readily migrate from subsurface source to the near-surface environment via high permeability pathways such as faults, fracture system, along bedding planes, or through sediments and strata with high porosity and permeability. Further, the composition of the materials detected at the surface can reliably predict the composition of materials from whence they were sourced. In some instances, particularly when the source beds are relatively shallow (<500 m), one cannot discount vertical diffusion as an important transport mechanism as well. In deep deposits, the amount of geologic time required to migrate hydrocarbons to the near surface by diffusion alone would be prohibitively long.

These same geochemical methods lend themselves nicely to assist in the rapid and economic screening of areas involved in environmental issues. Soil gas methods have been employed at the US Department of Energy’s Savannah River Site (SRS) facility to ascertain what “typical” volatile backgrounds exist on and near the complex (Wyatt et. al., in press, Richers & Wyatt, 1996, Richers et al., 1994). It was assumed that since the site was situated on Tertiary sands/clays overlying predominantly Triassic “redbeds” and/or crystalline basement that the natural soil gas character would be exceedingly low, exhibit a very dry composition, and otherwise, be absent. As will be shown, early assumptions did not take into consideration the presence of shallow marine/lagoonal strata nor the presence of lignitic materials in the subsurface. Between 1994 and 1996, over 2800 soil gas and soil plug samples were obtained in three sampling events. This suite of samples covered an area of approximately 2600 km² along the southeastern Atlantic Coastal Plain Province (Figure 1). An earlier soil gas survey was conducted in 1992-1993 at SRS, which was the precursor to these follow up surveys. Data from that limited survey will not be considered in this report. Figure 2 depicts the local geology in the study area. Primarily, the area is in the Coastal Plain Province, which is comprised of varying mixtures of
sand, clayey sand, clays, and subordinate carbonate. The overall sediment packages thicken and dip to the south-southeast. Carbonates become more important further down dip, but are mainly patchy or discontinuous in the SRS area. The sediments pinch out northward where metamorphic and igneous strata are exposed in the Piedmont Province. Several samples were obtained north of the Fall Line from predominantly metamorphic facies for control.

Whenever conducting a near-surface study, it is prudent to apply several techniques to ascertain what method or methods will be most reliable for the particular geology and climatic conditions. After several years of study, it was ascertained that for the fairly humid, heavily vegetated soils of the southeastern United States, soil plugs and shallow free soil gas can be applied successfully. Richers (1984) showed that application of the free soil gas method and soil plug method at Rose Hill, VA worked equally well in defining the composition and extent of that field. Wyatt, et al., (1995) did show however, that barometric pumping effects will greatly alter the results of free soil gas samples. Consequently, application of that method alone can prove unreliable. For this reason, both techniques were applied at SRS during the 1994 survey, with the bulk of the samples being soil plugs. The observed soil gas values were exceeding low in that study, such that the ensuing follow up survey of 1995 was based solely on soil plugs.

If at all possible, it is prudent to obtain source rock samples from the area to aid in calibrating soil gas signatures. In these studies, several carbonate outcrop samples from the Santee Limestone (Cretaceous) were obtained. Two side-wall core samples from an undisclosed strata were also obtained from the Girard, GA city water well.

2 SAMPLING PROGRAM

The initial sampling program applied at SRS in 1992-1993 consisted of a "loose" grid of approximately 500 sites with a cell spacing of 0.8 kilometers (0.5 mile). From this grid, only 469 cells were accessible. A total of 618 samples were obtained, with 469 being collected from a 1 meter depth and the remainder being either duplicate samples and/or samples obtained from varying depths. A detailed description of this earlier survey can be found in Richers et al., (1994). Light hydrocarbon (C1-C4) gases were determined for free soil gas samples, whereas gasoline-ranged (C5-C10), some VOA (volatile organic compounds) and chlorinated hydrocarbons were determined for soil plug samples. Unfortunately, this earlier survey did not determine light hydrocarbons on the soil plug samples. Consequently, data from this earlier survey is not included in this study, but is merely referenced for historical purposes.

While the results of this early survey were intriguing, some hydrocarbon hints were discovered over the site associated with photoincrement. The coverage was limited and no soil plug C1-C4 data were available. Therefore, it was decided to conduct a follow up survey in 1994 that utilized both soil gas and soil plug samples. This new survey resampled the initial survey area as well as extended it for several miles beyond the SRS Site boundary. This survey resulted in the sampling of 1500 samples from a 1 meter depth, and an additional 10% as repeats (Figure 3). Again, sample were collected from a grid spacing of roughly 0.8 km. Constituents determined were methane, ethane, ethylene,
xylene, and decane, (C5-C10).

A subsequent follow-up study was conducted in 1995 to expand upon the work of the 1994 survey. In this latest study, the samples were obtained on a much coarser spacing than the previous survey, with cell dimensions being about 1.6 km (1 mile). Most of the samples were obtained west, south, and east of the original areas, although several samples were collected in the Piedmont to the northwest. Figure 4 shows the combined sample locations for data collected 1994-1996. This effort collected soil plugs at 545 sites (with 10% repeats for a total of 599 samples). Determinations were made for the light hydrocarbon gases C1-C4 and the gasoline-ranged hydrocarbons (C5-C10).

Several outcrop samples of petroleum carbonates were collected from an outcrop along the Savannah River near Griffin Landing, GA. Two of these samples were submitted for Soxhlet extraction and subsequent GC-FID analysis of C15+ hydrocarbons. Light soil gas analysis was also performed to ascertain the C1-C4 hydrocarbon content. In addition, two samples of side-wall core supplied by the USGS from a domestic water well near Girard, GA were submitted for Soxhlet extraction, GC-FID C15+ analysis, and GC/MS biomarker analysis. It appears that this well was reported to have encountered an "oily" substance during drilling and the USGS had sampled a portion of the well for further investigation.

3 RESULTS

Statistical evaluation of the regional data revealed that select areas along the Coastal Plain had specific hydrocarbon magnitudes several times above the
Figure 7. Saturates, Aromatics, and NSO ternary plot of Griffins Landing outcrop samples.

Figure 8. Gas chromatogram of Girard Well sample 1 (323' or 98.5m).

Figure 9. Gas chromatogram of Girard Well Sample 2 (325.5' or 99.2m).

Figure 10. Normal Paraffin Distribution Diagram - Study samples compared to Messel Shale.

Regional threshold. Diagnostic soil gas ratios calculated for the light (C1-C4) saturated hydrocarbons indicated that source materials and/or free hydrocarbons contained in the subsurface of the area were a) thermogenic, and b) relatively mature (oil and/or gas prone). The suggestion that thermogenic hydrocarbons and/or source materials are present in the region are corroborated by independent tests of subsurface and surface rock sample tests. Those tests indicate the presence of mature marine/terrestrial sourced hydrocarbons near Griffins Landing (GL), GA, adjacent to SRS, and a terrigenous (turfic?) source (albeit not as mature as GL) near Girard, GA. Strong ethane through butane soil gas anomalies are felt to represent the extent of the marine sourced hydrocarbons, whereas the soil gas toluene signal are felt to correlate with the terrigenous material.

Specific soil gas-soil plug values obtained in the study were:

Soil gas:

- methane - 0.9 ppmv
- ethane - 0.3 ppmv
- propane - 0.02 ppmv

Soil plug:

- methane - 5.01 ng/g
- ethane - 0.04 ng/g
- ethylene - 0.09 ng/g
- propene - 0.06 ng/g
- propylene - 0.08 ng/g
- n-butane - 0.004 ng/g
- i-butane - 0.01 ng/g
- pentane - 0.03 ng/g
- toluene - 0.54 ng/g
- o-xylene - 0.26 ng/g
For soil gas, the butanes and halogenated hydrocarbons were not detected. The soil gas had non-detects for hexane, heptane, benzene, ethylbenzene, and m-p xylene.

Outcrop samples and side-wall core samples revealed that at least two potential sources of hydrocarbons existed in the area. The chromatograms for the Santee samples are shown in Figures 5 and 6. These samples revealed a fairly mature, albeit highly biodegraded character. Most likely these represent a fairly mature, marine-sourced bitumen. Figure 7 is a ternary plot of Saturates-Aromatics-NSO Compounds. As indicated, a general migration and/or maturation trend of bitumens goes from NSO rich to a moderately aromatic-rich bitumen to a saturate-rich bitumen. The Gaffney Landing samples exhibit this saturate rich character. The Girard core samples on the other hand show a distinctly immature, paraffin-rich character indicative of Type III kerogen, (Figures 8 & 9). When considering Normal Paraffin Distribution Diagrams, Tissot & Welte, 1978, showed a similar trend in the Messel Shale, a distinctly terrigenous shale) as seen in the N-distribution of the Girard samples. Figure 10 shows the distributions of Santee, Girard, and Messel shales. Clearly the Girard samples track the Messel shale distribution.

Having determined at least two distinctly different potential source rock types in the study area, it is interesting to see that the actual soil gas data further supports two distinct source compositions and distributions. Figure 11 shows soil gas values relative to the reported 1 ppb (volume) threshold. The southwestern portion of the study area shows a significantly higher concentration of samples with elevated values. Figure 12 shows the propane/methane * 1000 ratio for soil gas samples. Jones & Urdzik (1982), found that this ratio faithfully reflects the hydrocarbon nature of an area. In this assessment, the largest cluster of wet samples (~ 200 C3/C1 *1000) occurs over the Dunbarton Basin. Toluene, on the other hand was restricted to areas west and south of the wetter values shown in this figure. This indicates two separate source units. The more mature, normal marine source beds giving rise to thermogenic propane/methane ratios, and a less mature, terrigenous source giving rise to toluene.
4 CONCLUSIONS

Source rock analysis in the central Savannah River Area indicate two primary source units, a mature, marine source, and an immature, terrigenous source. Soil gas distributions reveal that the area nearest the Dunbarton Basin show the presence of hydrocarbons more like those in the literature deemed as "normal marine, mature", whereas the areas west and southwest of the Dunbarton show a more terrigenous, immature character. Further study in and along the margins of other similar buried Triassic basins along the coastal plain is warranted to better understand the indigenous nature of hydrocarbons when applying soil gas methods to site characterization.

REFERENCES


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