FIRST INTERNATIONAL CONFERENCE ON THE FLAT DILATOMETER

February 4, 1983

Edmonton, Alberta
Canada

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BACK

PROCEEDINGS

FIRST INTERNATIONAL CONFERENCE ON THE FLAT DILATOMETER

CONTENTS

BACK

TABLE OF CONTENTS

BACK

PROGRAM

1. PAST, PRESENT, & FUTURE OF THE FLAT DILATOMETER
   Dr. John H. Schmertmann, P.Eng.
   Schmertmann & Crapps Inc.
   Gainesville, Florida.

2. CASE HISTORIES INVOLVING THE FLAT DILATOMETER
   John A. Hayes, P.Eng.
   Site Investigation Services Ltd.
   Peterborough, Ontario.

3. FLAT DILATOMETER USE ON C.N. RAIL LINE, BRITISH COLUMBIA
   John Mekechuk, P.Eng.
   Senior Geotechnical Engineer
   C.N. Rail, Edmonton, Alberta

4. USE OF THE FLAT DILATOMETER IN THE BEAUFORT SEA
   Neil Burgess, P.Eng.
   Hardy Assoc. (1978) Ltd.
   Calgary, Alberta

5. CURRENT RESEARCH & DEVELOPMENT OF THE FLAT DILATOMETER
   Dr. R. G. Campanella, P.Eng.
   University of British Columbia
   Vancouver, B.C.

6. PANEL QUESTION PERIOD

7. EQUIPMENT DEMONSTRATION

BACK

ATTENDEES

AGAR, JIM	M. J. O'Connor
BARBOUR, LEE	U of Sask. - Civil Eng.
BEDNAR, RONALD	Sereda
BOURBONNAIS, JACQUE	Rock test Ltd.
BRADLEY, MIKE	Underwood McLellan Ltd.
BURGESS, NEIL	Hardy Associates (1978) Ltd.
CAMPANELLA, RICHARD	UBC - Civil Eng.
CLARK, VERN	Mobile Augers and Research Ltd.
CRAIG, CHRIS	Ontario Hydro
CROOKS, JACK	Golder Associates
COATES, BOB	MARL
CURRIE, DON	MARL
DAVIS, HANK	Mobile Drilling Company, Inc.
DIGGLE, DENNIS	Foundex Explorations Ltd.
DOOHAN, JIM	Shelby Engineering Ltd.
DUMONT, GENE	MARL
ERSOY, TURGET	Foothills Pipe (Yukon) Ltd.
FRIDELL, TOM	Curtis Engineering Ltd.
FUNNEGARD, ERIK	Western Geosystems Inc.
HARRIS, MURRAY	Thurber Consultants Ltd.
HAYES, JOHN	Site Investigations Services Ltd.
HEIN, FRAN	U of A Geology
HUGHES, JOHN	Situ Tech Inc.
HUNTER, GORD	Shelby Engineering Ltd.
INNES, RON	MARL
INNES, MURRAY	MARL

BACK

JEFFRIES, MIKE	Gulf Canada Resources Inc.
JUBIAN, BILL	CNR
DRUGER, KEN	BBT Geotechnical Consultants Ltd.
KRUGER, MANNY	CNR
MACHIBRODA, PAUL	P. Machibroda Engineering Ltd.
MARTIN, BOB	EBA Engineering Consultants Ltd.
MCROBERTS, ED	HARDY ASSOCIATES (1978) LTD.
MEKECHUK, JOHN	CNR
MORRISON, IAN	Klohn Leonoff Ltd.
PARITI, MURTHY	Alta. Transportation
PETERS, JOHN	Underwood McLellan Ltd.
PLEWES, HOWARD	Hardy Associates (1978) Ltd.
ROBERTSON, PETER	UBC - Civil Engineering
ROSS, BRIAN	BBT Geotechnical Consultants Ltd.
SCHMERTMANN, JOHN	Schmertmann & Crapps
SELMECZY, NICK	Pioneer Drilling Ltd.
SHEERAN, DON	McMurray Resources
SWEENEY, Gerald	Aardvark Drilling Sales Ltd.
ZACHARKO, Nestor	MARL

SCHMERTMANN

BACK

4 Feb 1983

PAST PRESENT AND FUTURE OF THE DMT

by: John H. Schmertmann, P.E., Ph.D.
Schmertmann & Crapps, Inc.
Gainesville, Fla.

1. PAST

1.1 History of development: Initially conceived by Professor Silvano Marchetti as a test to get lateral modulus response of laterally loaded steel piles (Olsson did same in Sweden about 1919 to invent the vane shear test). Temporarily abandoned this as he saw opportunities to correlate against soil properties. He gradually, by insight-trial-discovery, developed correlations.

1974 - Started to develop insitu tool for horiz. modulus.

1975 - Short paper to Raleigh ASCE Spec. Conf. introducing DMT (Vol. 2, p. 255).

1977 - Jamiolkowski starts using DMT in consulting projects for correlation purposes.
- Introduced DMT at IX ICSMFE, Tokyo, Spec. Session #10.
- Inspired by Burland Tokyo SOA statement"...it can be concluded that testing should be aimed at establishing the simple in-situ parameters. The most important appears to be the one-dimensional compressibility m_v or the equivalent effective vertical Young's modulus E'_v and the variation with depth". (Vol. 2, p. 518)
- Discovered early E_D vs. ($M = 1/m_v$) correlations.

1978 - Design revised to streamline shape of blade and give it a sharper, curved cutting edge to minimize insertion disturbance.

1979 - Marchetti sends DMT equipment to Schmertmann & Crapps, Inc. Fla. for trial and evaluation. S&C start using in practice.

1980 - Marchetti publishes ASCE GD paper in March (p. 299), with detailed description of flat blade dilatometer and the correlations. Schmertmann discussion, with Marchetti closure in June 81.

1981 - Marchetti visiting Prof. at Univ. of Fla., starts research on DMT, leads to 1st PhD on DMT (Boghrat, 1982).
- GPE, Inc. becomes N. Amercian distributor for DMT equipment.
- Commercial use starts in Canada.

BACK

1982 - Two ESOPT II (Amsterdam, May) papers on DMT, by Schmertmann (theor. o prediction) and Marchetti (liquefaction), with floor discussions about DMT. 1st discussions at an international conf.

1983 - First conference devoted entirely to DMT, Edmonton.

1.2 Marchetti's evolved philosophy:

a. Penetrometer type test: because of its speed, potential independence from boring operations, potential for near-continuous profiles of data and results.

b. Use of a sharpened blade: simulates plane strain conditions and ameanable to two-dimensional modeling; experiments showed much less disturbance than around a cone penetrometer, and much more uniform in the zone of measurement; amount of disturbance relatively constant because of independence of operator technique and therefore making correlations eventually more accurate because of less variability, extrapolation to no-blade field condition values potentially more accurate because of shorter extrapolation.

c. Blade dimensions: chosen as rugged (solid stainless steel) as practical to permit hammer impact as well as quasi-static penetration and thereby greatly increase potential of range of materials in which the DMT could penetrate.

d. Non-electronic: intent to make it simple and rugged to use, non-sensitive to ordinary field testing abuse, and most importantly - repairable in the field.

f. Very small membrane deformation: keeps stresses in the over-consolidated and nearly-elastic range and thus allows effective stress changes under undrained conditions. The DMT thus permits some evaluation of clay consolidation characteristics.

g. Critical attitude: tries to be first to find errors, problems with DMT; modest claims, supports research by others.

1.3 Exceptionally high-quality correlations:

a. Special association with Professor M. Jamiolkowski (S.M. his first graduate student at Univ. of Torino) - who also heads a high-quality geotechnical consulting company in Italy, one of the best in Europe.

b. Prof. J.M. subsidized the use of DMT in conjunction with major projects wherein very high quality data of more conventional types also obtained and thus available to help establish correlations.

BACK

1.4 History of S&C Inc. involvement:

a. Marchetti first approached JHS in 1977 - ignored, looked too simple!

b. 1979 - contact in Milano, after Prof. Jamiolkowski insisted that JHS look at some of the correlation information.

c. Marchetti sent JHS equipment in mid 1979, first used on consulting project in Aug 1979 (W&C consultants cooling towers in Florida).

d. Marchetti a visiting professor at UF - 1980-81.

e. GPE Inc. signs agreement to sell Marchetti flat plate dilatometers in USA and Canada.

f. continue to use DMT at every opportunity, with a major application use (1,000 tests, mostly from barges) for the geotechnical investigation for the Skyway Bridge across Tampa Bay.

g. JHS writes paper giving first theoretical soil property prediction using DMT - the friction angle in sands.

2. PRESENT

2.1 Current status of useage and research

a. Universities: UBC, UF, Clarkston College, now active; others that have either purchased or expect to purchase the equipment for research purposes; Purdue, LSU, NC State, Carleton in Ottawa.

b. other research organizations active: NGI, ENEL-Milano, Univ. of Torino, L'Aquila (Rome), Dr. A. Luttenegger of NSF grant to Bulgarian Academy of Science, Norwegian Road Authority (Oslo).

2.2 Use by North American engineering organizations in practice:

a. Mobile Augers & Research, Ltd. Site Investigation Services, Ltd. (Ont.), Hardy Assoc. (Alberta), Williams & Associates Inc. (Florida), Schmertmann & Crapps, Inc. (Florida), STS Consultants (St. Louis) (purchased but not yet used).

b. Consultants that have expressed a strong interest but not yet purchased: Ardaman & Assocs., Law Engineering & Testing, Hayward-Baker.

BACK

2.3 Other companies: Fugro (Netherlands); Pressuremeter Insitu Techniques (England); the SGI in Milano, Paler SA, Montagnola Ticino, Switzerland; Dicht SA, Zurich, Switzerland; Raymond International UK, London, England; Terramonitoring, Johnannesburg, South Africa.

2.4 Correlations appear acceptable in types of soil materials outside the basic Marchetti-Jamiolkowski correlation data:

a. Very weak soils - FL mining montmorillonite clay slimes, Fredericton clayey silt, FL surface peat, Norwegian weak and moderately sensitive clays,

b. Very strong soils - St. Petersburg hard clays at Skyway Bridge project, St. Louis coarse sand and gravel cofferdam cell fill at CE Lock and Dam 26 project,

c. Residual soils - Greensboro NC, Venezuela (Caracas) consultant Mr. Tapia,

2.5 Correlations, other than possibly modulus values, appear unacceptable in their present form in some soil materials:

2.5.1 Crushable soils:

a. The very variable, vuggy soft limestones (limerocks) in S. Florida can produce severe equipment damage and poor soil property predictions (even when blades and membranes not damaged).

b. Loess soils, low in clay content and also in their prewetted condition.

c. c-ø soils with a brittle, cemented structure?

2.5.2 loose, high permeability soils: Compactable by the vibrations transmitted to the blade by hammer (and vibro?) driving (sands with low to medium relative density). Static pushing better.

2.6 Current research: (by Prof. Dick Campanella)

3. FUTURE

3.1 With respect to versatility: Automatic accumulation and processing of data very likely to come soon for those who have a high volume of data to process and can justify the cost and field maintenance aspects.

3.2 With respect to accuracy: Correlations will improve as the data base expands and the research results come in. Possibility that pore pressure information in conjunction with present dilatometer information will greatly expand usefulness - but at the price of complexity and vulnerability.

BACK

3.3 Expanding the properties correlated: This has already begun to happen and should continue. For example:

a. Pore pressure, permeability, coeff. of consolidation, stratigraphy if flat dilatometer also becomes a "piezoblade".

b. Evaluation of soil compactibility by comparing data from pushed and driven dilatometers.

c. Evaluation of swelling potential by measuring changes in horizontal stresses using water control methods.

d. Coefficient of horizontal subgrade reaction predictions and lateral pile movement predictions.

e. Evaluating displacement pile friction behavior in special soils - such as calcareous sands where structure and crushability play an important part.

f. Evaluation of liquefaction potential.

3.4 General usefulness in practice: The DMT test is exceptionally versatile and practical for engineering use. It provides data of generally adequate accuracy for preliminary engineering design, which may also prove adequate for many final designs. Considering that the relatively rather crude SPT data is used for such purposes, then the superior DMT data will also be likely used for such purposes. I anticipate that DMT work will become routine in many engineering testing companies and will become a common test required by consulting engineers. Because of its special features of ruggedness, practicality for offshore work, and the quality of the data obtained, I also anticipate that it will become common for offshore work.

3.5 Horizontal stresses: The profession is just beginning to understand the possibly commanding importance of insitu horizontal stresses. It is quite possible, and perhaps likely in many circumstances, that the insitu stress conditions will dominate the behavior of mathematical models (as finite element) and physical models (as in the centrifuge) of complex problems. The DMT provides a rugged and cost effective insitu test that perhaps already can measure insitu horizontal stresses with adequate accuracy for many applications. I anticipate many important surprises with such measurements and anticipate that engineers will soon want to know their site horizontal stress conditions on a routine basis. The DMT provides an attractive method for obtaining such data and this will accelerate its acceptance and wide useage.

BACK

3.6 Immediate compressibility data: The availability of immediate consolidation-type test data (type soil, P_c and M), in volume, and with an accuracy acceptable for many applications, will soon spoil engineers. Why wait weeks for consolidation test results? Lab tests will gradually be used only for confirmation-check purposes, or in soil material conditions not suitable for the DMT.

3.7 Ground Improvement Test Monitoring: There exists a great need for an efficient insitu test to evaluate the before-need and the after-effect for ground improvement work such as dynamic compaction, vibroflotation, compaction grouting, etc. Such improvement often involves sands and silty sands and improving their liquefaction potential. The DMT seems ideal in many ways for such testing, expecially because it has the potential for separating the effects of the improvement's change in density and change in horizontal stress. I anticipate a growing demand for such ground improvement work and the DMT to monitor such work.

3.8 Evaluating very weak soil materials: The DMT seems to have exceptional sensitivity in very low strength (s_u = +/- 0.002 b) and very high compressibility (M = +/- 0.5 b) soil materials. Increasing interest in materials such as mine tailings, waste disposal areas, using rather than removing organic soils, etc. should create an increasing demand for a test such as the DMT.

3.9 Resistance to using DMT: Present investment in expensive laboratory equipment as well as inertia of profession to change provide resistance to using DMT. Time will be required to overcome these objections.

BACK

HAYES

BACK

CASE HISTORIES INVOLVING THE FLAT DILATOMETER

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1.0 INTRODUCTION

Since obtaining our first Flat Dilatometer testing unit in the summer of 1981, we have carried out a large number of tests on more than 15 different projects. My first impression after seeing the introductory information provided by Dr. Marchetti and Dr. Schmertmann was that here was an in-situ testing instrument that would be useful, practical and easily adapted to the conventional test boring and sampling techniques used in southern Ontario. We have not been disappointed.

For most routine geotechnical investigations, we require reasonably accurate information regarding soil type, soil strength and soil compressibility. Precision data is not usually necessary and for that reason a majority of the foundation designs in our practice have been based on conventional standard penetration test (SPT) correlations augmented by laboratory compression and shear tests. So far in our experience with the dilatometer, we have found that it not only provides accurate information regarding soil type, strength and compressibility, but also that it provides a much more precise picture of subsurface soil characteristics than is provided by SPT techniques. The precision derives partly from the relatively sensitive pressure measurements and partly from the quasi-continuous test profile. This precision is a definite bonus for routine soils investigations.

Because the Flat Dilatometer testing process is new, we have been very curious to see how well it can predict actual soil behaviour. To satisfy this curiosity, we have compared dilatometer test results to other conventional test data and we have also compared actual measured settlements to those predicted from dilatometer data at three separate projects.

The flat dilatometer equipment and procedures are described by Marchetti (1980) and discussed by Schmertmann (1981). The procedures for determining friction angle in sands using the flat dilatometer are described by Schmertmann (1982).

BACK

2.0 SOIL TYPE

Material index (Id) appears to provide a reasonably accurate indication of soil type. Three typical soil profiles are shown on Figures 1 to 3 along with the significant dilatometer data. In each of these cases, the dilatometer testing was done about 3 to 5 feet away from the borehole. Note that on Figure 1 (Emily Creek Site) the fairly complex soil profile including peat, marl, clay, silty clay, silty sand and sand is reflected quite well by the Id profile. Although the Id for peat and clay soils are similar, the very low compressibility (less than 15 kg/sq. cm) allows us to differentiate the peat zone.

Our experience to date is that the material index is a reasonably accurate indicator for a wide variety of soil types. As Marchetti points out, however, the precision in the "transition zone" between Id=0.6 and Id=1.8 is not always good. We have found that very dense silt soils show up as fine sand. Also, clayey sands may be indicated as silt. Nonetheless, we are satisfied that the material index is a reliable indicator of major soil types.

3.0 SHEAR STRENGTH/COHESION

We have been able to compare vane shear test results with dilatometer undrained cohesion at four different sites in southern Ontario. As shown on Figure 4, the correlation is good over a range of very soft to stiff clays. It should be kept in mind that the vane tests were done to routine investigation standards at sites with considerable variation in strength with depth. (See cohesion results on Figure 1 for example). Therefore, some of the variability in the correlation may be in the vane test results. In any case, our data indicates a tendency for the dilatometer cohesion to be slightly lower than the vane shear strength which is a tendency reported by Marchetti and others.

The comparison of cohesion values in a peat deposit in Victoria County near Lindsay, Ontario were particularly interesting. At this site, the average of 12 vane shear tests (3" x 6" vane) indicated a shear strength

BACK

of 0.15 kg/sq. cm (TSF). Our experience with peat deposits in this part of Ontario is that vane shear values must be reduced by 50% before using them to determine safe embankment heights. It may be significant that the average "cohesion" measured with the dilatometer in this deposit was 0.08 kg/sq. cm (TSF) or about 50% of the average vane shear results. There is some evidence, therefore, that dilatometer cohesion estimates will prove to be more realistic than vane shear strengths for assessing the stability of earth embankments on peat deposits.

4.0 SOIL COMPRESSIBILITY

In order to satisfy our curiosity about the effectiveness of flat dilatometer test results for predicting settlements, we undertook to carry out dilatometer tests at three sites where we have monitored actual settlements. The three sites are:

- Wander Limited Plant, Peterborough
- Citi-Centre Apartment (King Street), Peterborough
- County Road #21 Swamp Crossing, Victoria County

4.1 Settlement Calculation

Settlement Calculation - The settlements estimated from the dilatometer data were calculated by determining the stress increment at each layer due to structure and/or fill loadings which was then used in the following expression to determine the compression (or reduction in thickness) of each layer

$$S = \frac{\Delta\sigma_v}{M} \Delta z \quad \text{where}$$

S = reduction in thickness of designated layer

M = dilatometer compression modules

$\Delta\sigma_v$ = stress increment at centre of layer due to added loads

Δz = original thickness of layer before loading

A check is made for each layer to determine if the preconsolidation loading is exceeded. A computer programme was set up to compute the stress increments and compression for each 20 cm layer of a dilatometer test profile. A typical printout is shown on Figure 5.

BACK

4.2 Wander Limited Site - This is a typical industrial plant structure with conventional spread and strip footings placed on a 4 ft (1.2 m) high sand and gravel fill. The loadings from this structure are reasonably uniformly distributed and the combination of fill loads, structural dead loads and "permanent" live loads (i.e. storage, equipment etc.) produces a loading of approximately 1.0 TSF (kg/sq. cm). The dilatometer and typical borehole profiles for the site are shown on Figure 6. This soil profile is similar to that found in a large portion of the Peterborough area. The fine sand and silt soils were deposited in Glacial Lake Peterborough during the latter stages of the Wisconsin glaciation.

Settlements measured at three locations inside the structure indicated a range from 15 to 20 mm with an average of 18 mm. The computed settlement from the dilatometer data is 21 mm.

4.3 Citi Centre (King Street) Apartment Building - This is a 6-storey apartment building with an underground parking garage. Since significant settlements were expected for this structure, the foundation was designed as a semi-rigid box with heavily reinforced side walls to distribute loads over most of the basement area. The average net loading (after deducting the weight of soil removed for the basement) including dead load and permanent live loads was calculated to be 0.94 TSF (kg/sq. cm).

The soils below the structure are described on Figure 7. As with the previous site, they also consist of fine sands and silts associated with deposits in Glacial Lake Peterborough. The dilatometer test data are also shown on Figure 7.

Several conventional methods were used to estimate the settlement for this structure. As shown on Figure 8, the predicted range of settlement using these methods was 40 to 75 mm. The actual measured settlement, 2 years after construction, was 45 to 50 mm along the length of the building. The estimated settlement based on dilatometer compressibility values is 58 mm.

BACK

4.4 Victoria County Road #21 Swamp Crossing - This site is located about 8 miles north west of Lindsay, Ontario. The swamp is about 900 metres long and consists of fibrous and amorphous-granular peat ranging from 3 to 4 metres thick, approximately. The peat is underlain by silty clay and sand as shown on Figure 9. The original embankment varied from 1.0 to 2.4 metres in thickness and an additional 1.2 metres of fill was placed during the reconstruction to raise the grade. To prevent shear failures, the new embankment was provided with berms and the fill was placed in stages (Figure 10). Elevation profiles of the roadway were measured before, during, and after construction.

The design estimates of settlement were based on a "geometrics" approach using borehole data and the geometry of the original embankment at several locations to build up a relationship between height of fill and compression of the underlying peat expressed as a percentage of the original peat thickness. This relationship (see Figure 11) was confirmed by oedometer tests on several peat samples which tended to bracket the more precise "geometrics" results. Our prediction was that total settlement would amount to 300 to 400 mm over a period of 10 to 30 years. Monitoring of the new embankment indicates that the projected long term post-construction settlement will indeed be about 250 mm to 300 mm. (see Figure 12). The prediction based on the dilatometer compression modulus data ranged from 260 to 340 mm depending on the thickness of peat used.

4.5 Further Comments on Settlement Predictions - In addition to the three cases cited above, we have compared dilatometer settlement predictions with settlement predictions based on conventional techniques at four other sites. In all cases, we have found very close agreement in the magnitude of predicted settlements, which range from 8 to 30 mm. We are now quite confident that the dilatometer test data can be used to produce reasonable and accurate settlement predictions. Our experience also indicates that these predictions tend to be slightly higher than actual.

BACK

5.0 CONCLUDING REMARKS

After using the Flat Dilatometer over a period of 18 months, we are quite satisfied that the method gives us reliable information regarding soil type, strength and compressibility. The equipment is relatively straightforward to use in the field and is rugged enough to be used with conventional SPT drilling rigs. We now use the dilatometer routinely to augment our conventional in-situ testing.

REFERENCES

Marchetti, S., 1980, "In-Situ Tests by Flat Dilatometer," ASCE, Journal of Geotechnical Engrg. Div., Vol. 106, GT 3, pp. 299-321

Schmertmann, J.H., 1980, "In-Situ Test by Flat Dilatometer," Discussion to Marchetti, 1980, ASCE paper, ASCE Journal of Geotechnical Engrg. Div.

Schmertmann, J.H., 1982, "A Method for Determining Friction Angle in Sands from the Marchetti Dilatometer Test (DMT)," Proceedings of the Second European Symposium on Penetration Testing/Amsterdam/ 24-27 May 1982.

BACK

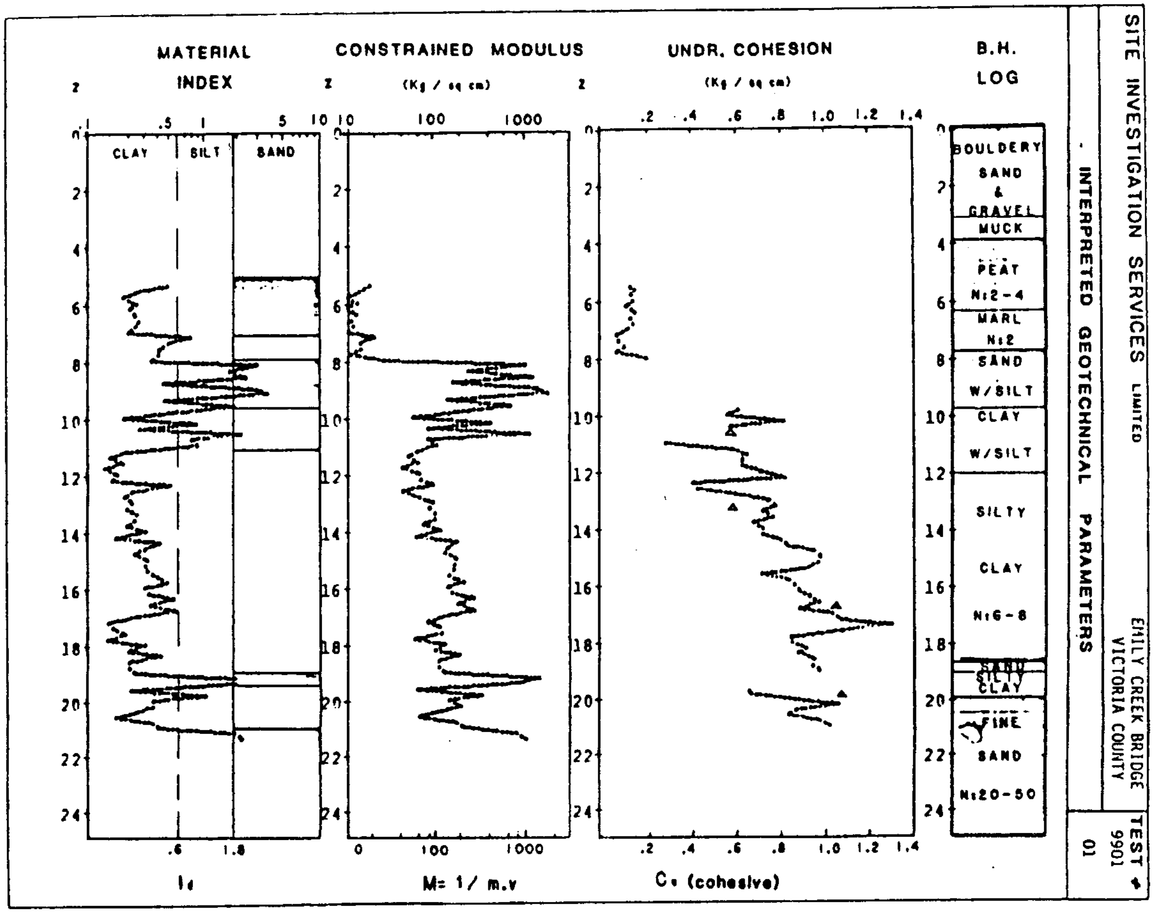


FIGURE 1

BACK

NORTHRAM INDUSTRIAL PARK - 2761

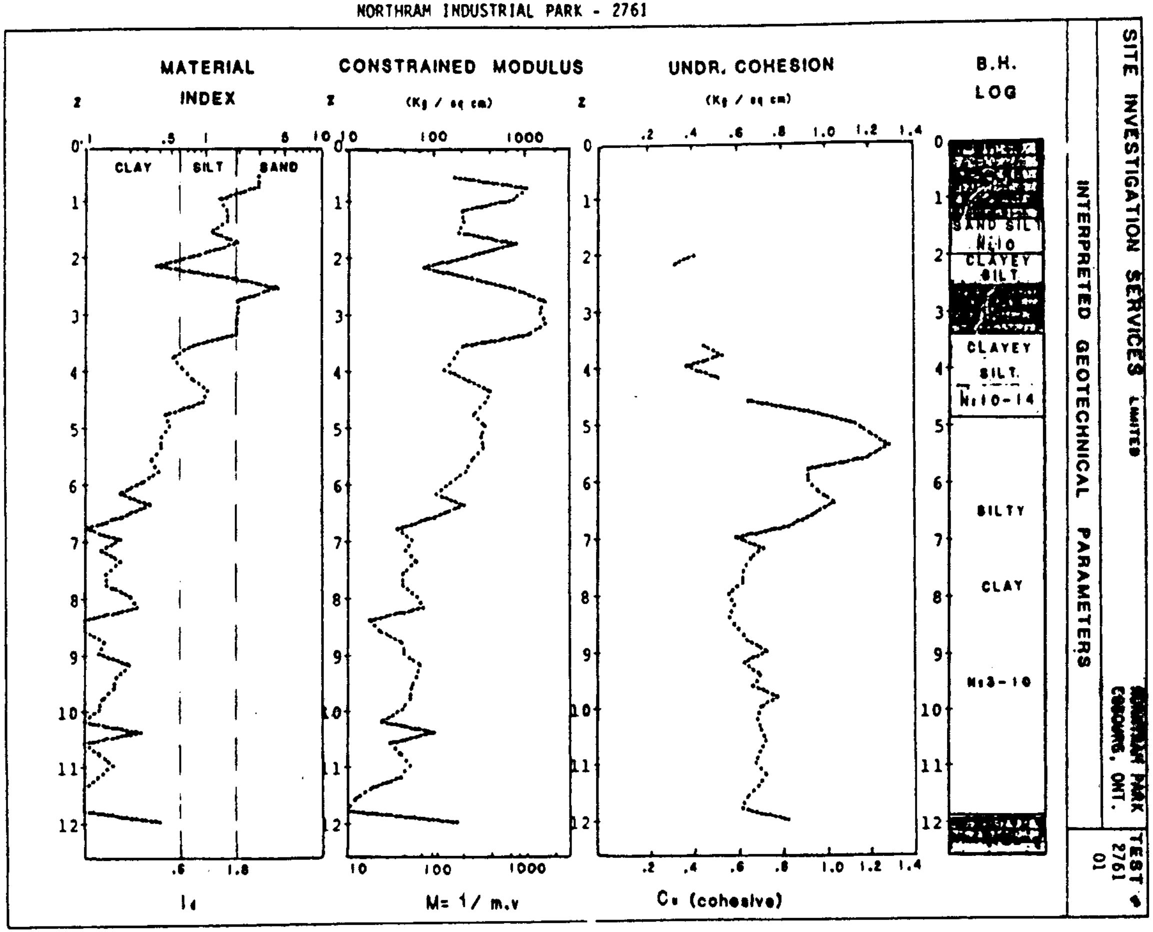
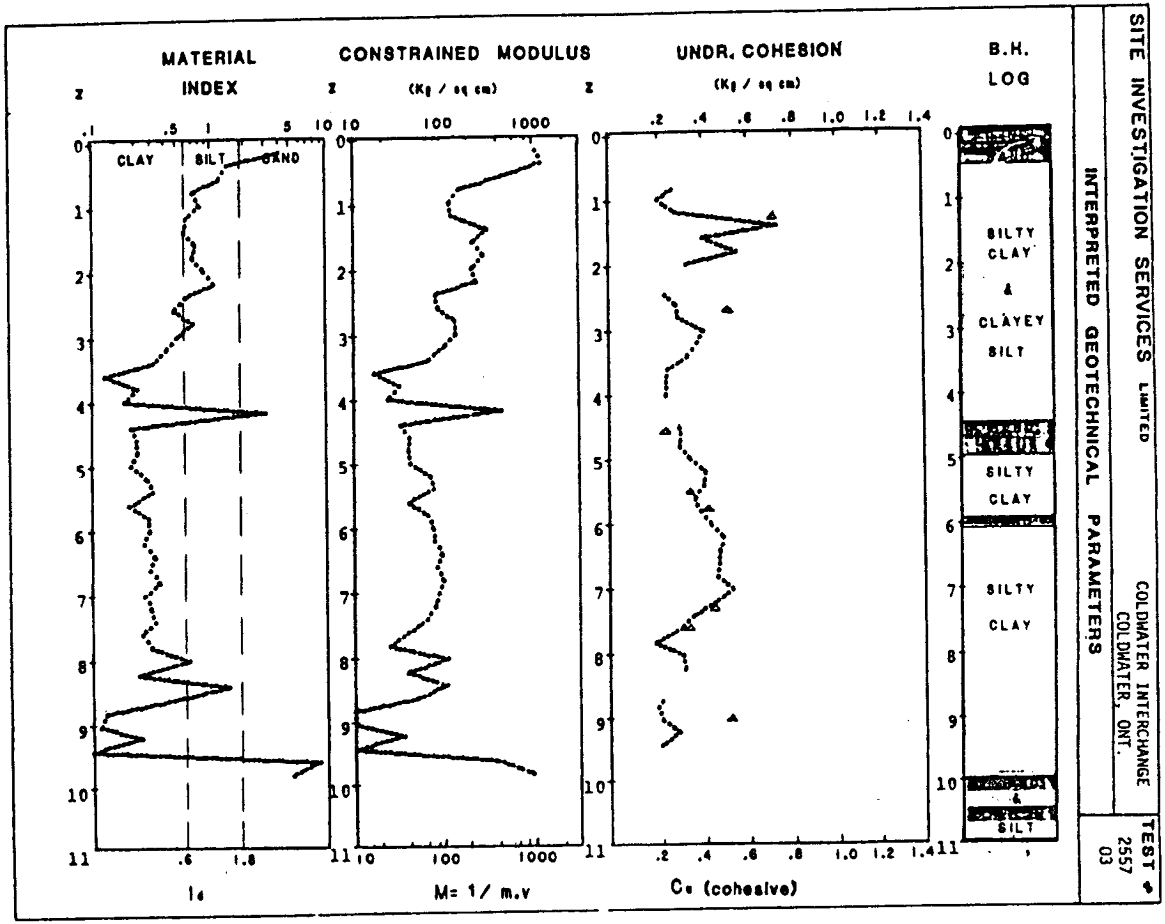


FIGURE 2



MATERIAL INDEX
z
CLAY
SILT
SAND
.1 .5 1 5 10
.6 1.8
I_d
CONSTRAINED MODULUS
(Kg / sq cm)
10 100 1000
M= 1/ m.v
UNDR. COHESION
(Kg / sq cm)
.2 .4 .6 .8 1.0 1.2 1.4
C_u (cohesive)
B.H. LOG
SILTY CLAY & CLAYEY SILT
SILTY CLAY
SILTY CLAY
& SILT
0 1 2 3 4 5 6 7 8 9 10 11
SITE INVESTIGATION SERVICES LIMITED
INTERPRETED GEOTECHNICAL PARAMETERS
COLDWATER INTERCHANGE
COLDWATER, ONT.
TEST #
2557
03

FIGURE 3

Site Investigation Services Limited
677 Crown Drive
PETERBOROUGH, ONTARIO K9J 6W2
(705) 743-6850

JOB
SHEET NO. OF
CALCULATED BY DATE
CHECKED BY DATE
SCALE

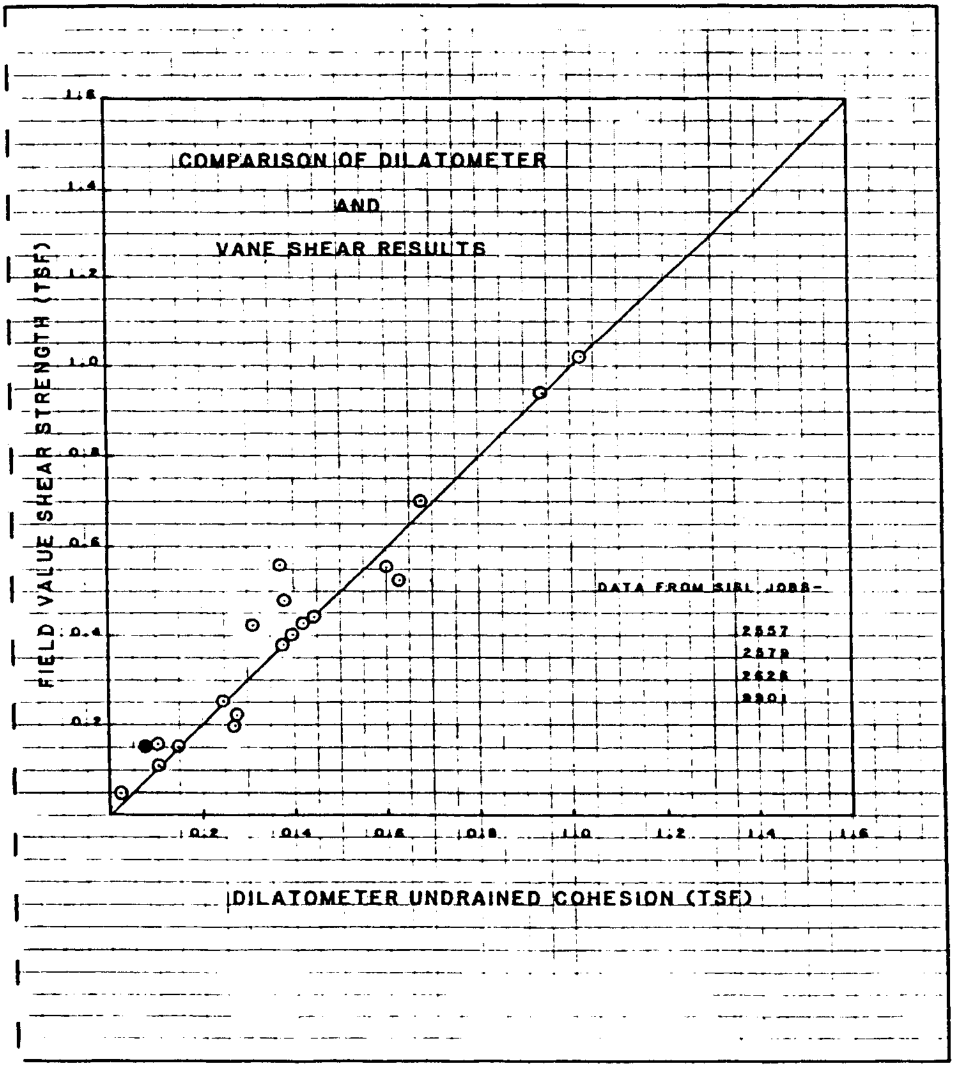


FIGURE 4

SETTLEMENT ANALYSIS FOR JOB#J0791

DILATOMETER TEST#01
FOOTING DATA: WIDTH= 30 , LENGTH= 75 , DEPTH= 0
NET BEARING PRESSURE AT BASE OF FOOTING= 1 Kg/cm2(TSF)

DEPTH (m)	BELOW FOOT	STRESS FACTOR	ADDED STRESS	TOTAL STRESS	OCR EST.	M	SETTLEMENT (mm)	O.C. CHECK
0.80	0.80	0.995	1.00	1.14	138.89	982.5	0.20	
1.00	1.00	0.995	1.00	1.18	14.90	188.6	1.06	
1.20	1.20	0.995	1.00	1.21	13.07	235.8	0.84	
1.40	1.40	0.995	1.00	1.25	2.51	118.7	1.68	*
1.60	1.60	0.995	1.00	1.28	8.99	636.0	0.31	
1.80	1.80	0.995	1.00	1.32	14.83	654.1	0.30	
2.00	2.00	0.995	1.00	1.36	17.24	955.9	0.21	
2.20	2.20	0.995	1.00	1.40	23.07	1006.8	0.20	
2.40	2.40	0.995	1.00	1.42	11.49	638.9	0.31	
2.60	2.60	0.995	1.00	1.44	14.94	874.0	0.23	
2.80	2.80	0.995	1.00	1.45	8.38	145.3	1.37	
3.00	3.00	0.995	1.00	1.47	13.23	733.2	0.27	
3.20	3.20	0.995	1.00	1.49	11.51	531.7	0.37	
3.40	3.40	0.995	1.00	1.51	9.65	544.7	0.37	
3.60	3.60	0.995	1.00	1.53	11.06	696.6	0.29	
3.80	3.80	0.995	1.00	1.55	13.51	810.5	0.25	
4.00	4.00	0.995	1.00	1.57	4.13	400.2	0.50	
4.20	4.20	0.995	1.00	1.59	10.19	648.7	0.31	
4.40	4.40	0.995	1.00	1.60	7.53	646.8	0.31	
4.60	4.60	0.988	0.99	1.62	8.32	604.1	0.33	
4.80	4.80	0.984	0.98	1.63	3.00	361.2	0.54	*
5.00	5.00	0.979	0.98	1.64	1.52	173.0	1.13	*
5.20	5.20	0.975	0.98	1.65	5.43	445.5	0.44	*
5.40	5.40	0.971	0.97	1.67	2.66	125.3	1.55	*
5.60	5.60	0.967	0.97	1.68	1.10	118.9	1.63	*
5.80	5.80	0.963	0.96	1.70	9.76	645.1	0.30	
6.00	6.00	0.958	0.96	1.71	1.28	77.3	2.48	*
6.20	6.20	0.954	0.95	1.72	1.28	77.9	2.45	*
6.40	6.40	0.950	0.95	1.73	0.81	354.5	0.54	*

TOTAL SETTLEMENT FOR LOADING OF 1 Kg/cm2(TSF)= 20.7 mm

FIGURE 5

BACK

WANDER LIMITED PLANT - 0791

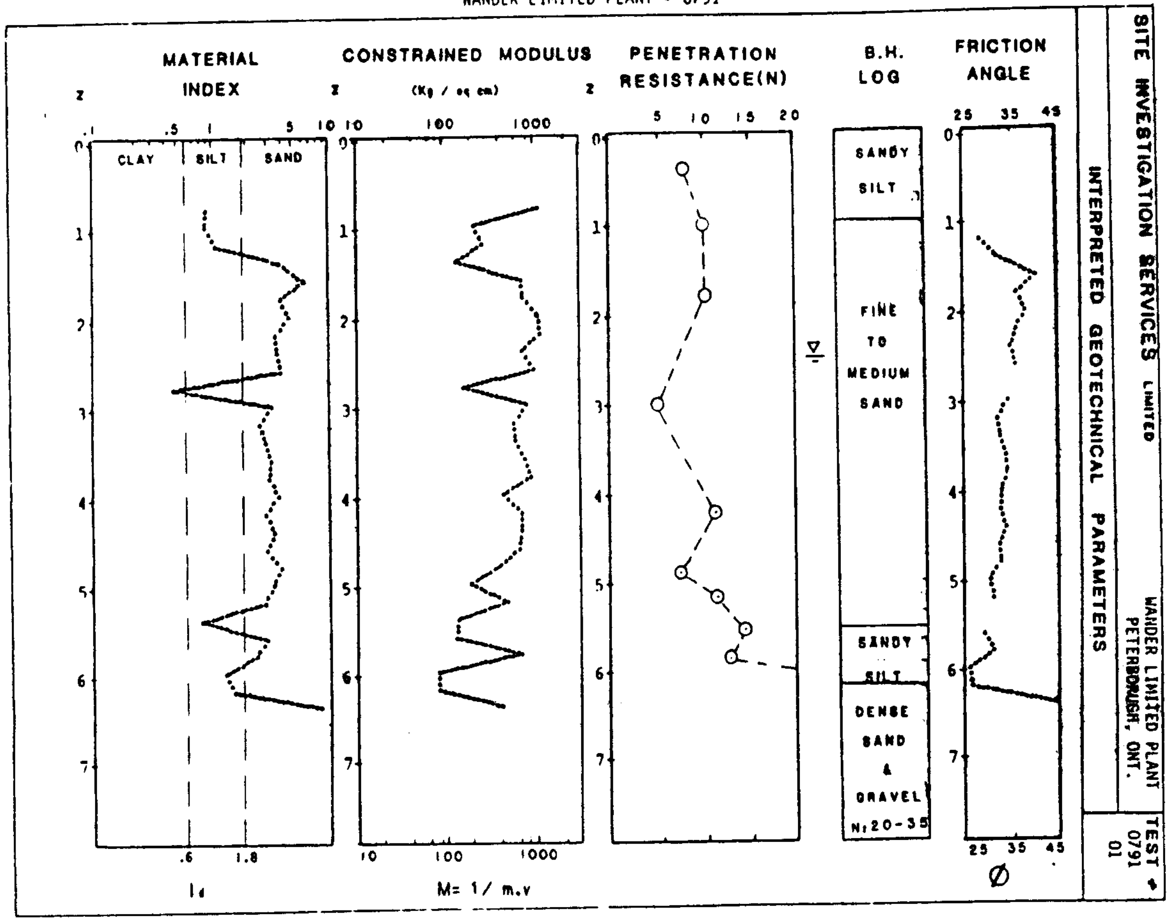


FIGURE 6

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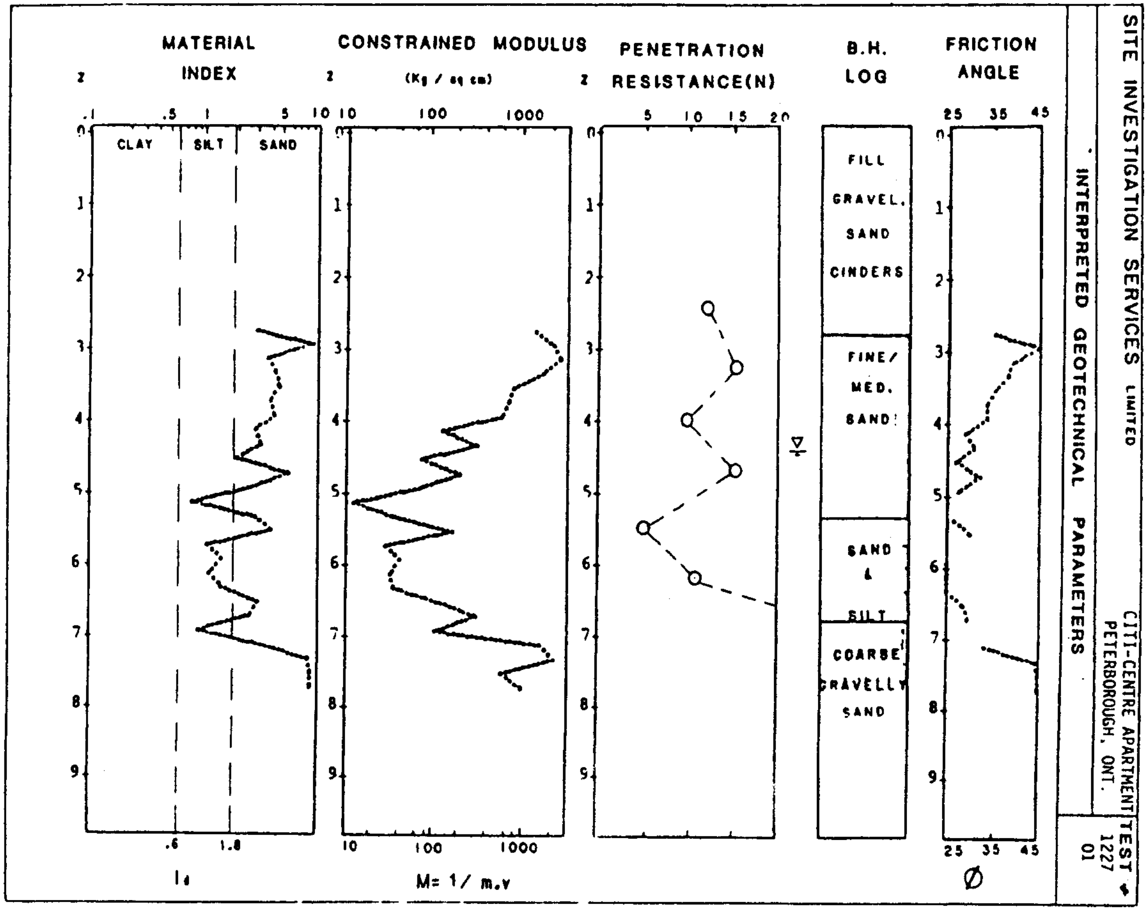

FIGURE 7

BACK

Site Investigation Services Limited
677 Crown Drive
PETERBOROUGH, ONTARIO K9J 6W2
(705) 743-6850

JOB
SHEET NO — OF
CALCULATED BY — DATE
CHECKED BY — DATE
SCALE

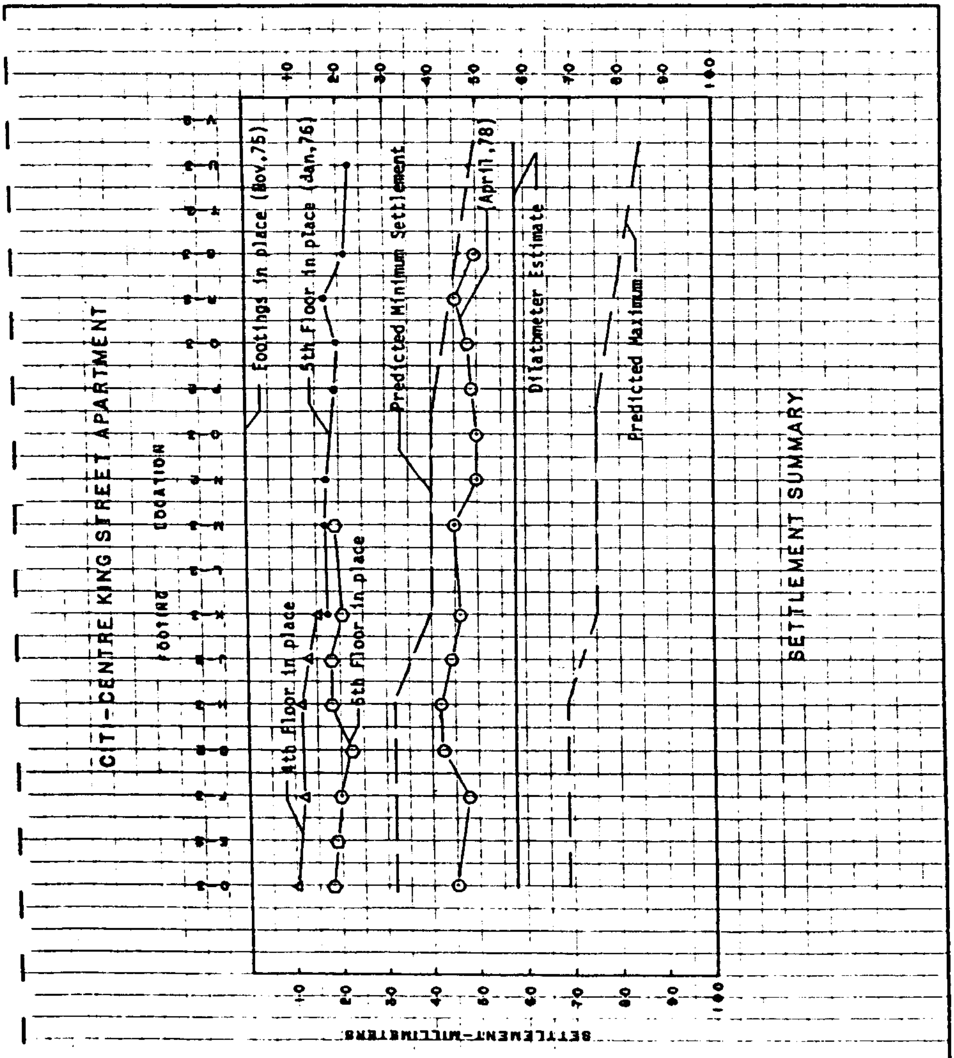

FIGURE 8

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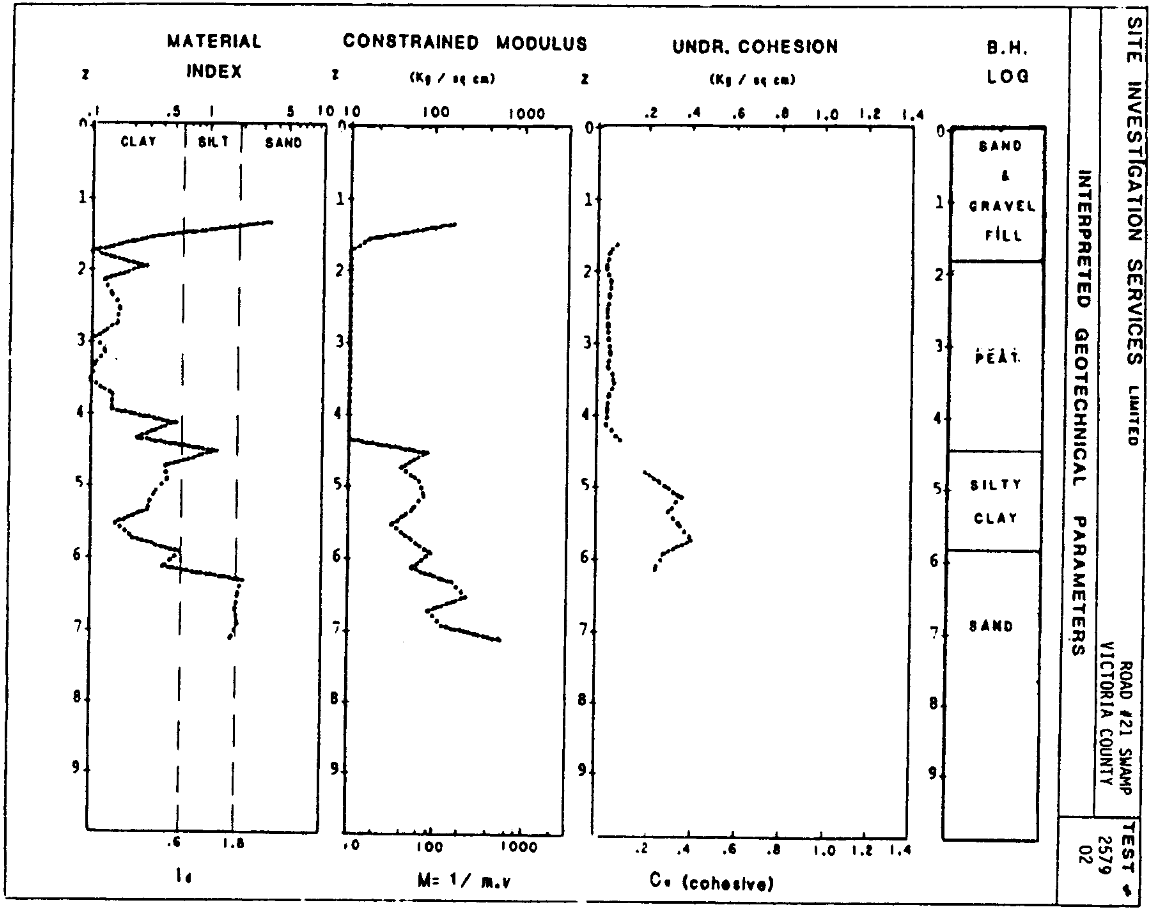
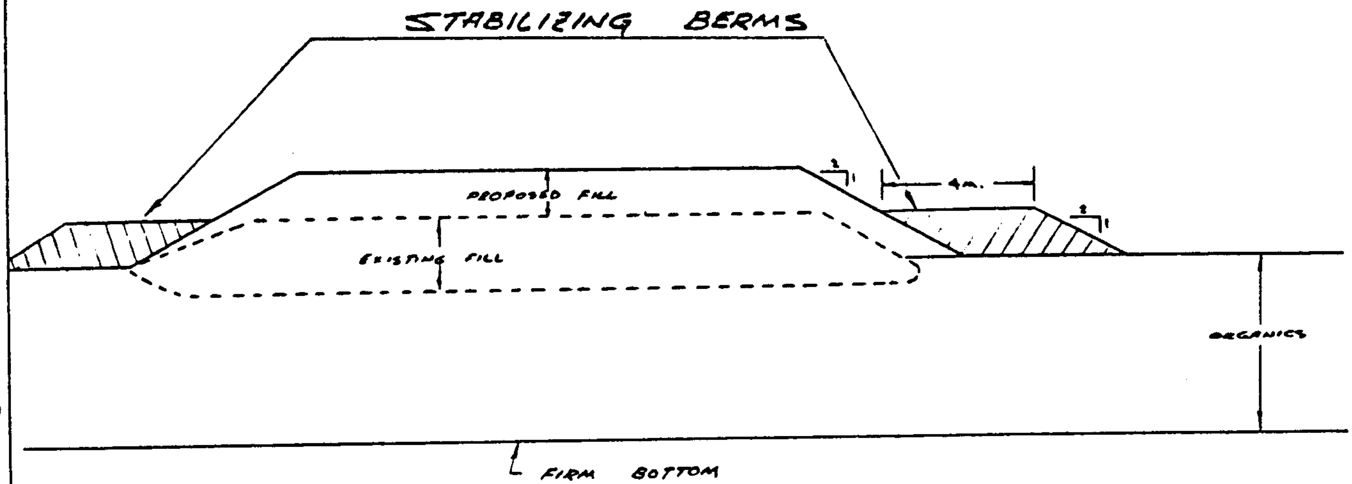

SITE INVESTIGATION SERVICES LIMITED
INTERPRETED GEOTECHNICAL PARAMETERS
ROAD #21 SWAMP
VICTORIA COUNTY
TEST #
2579
02
MATERIAL INDEX
CLAY
SILT
SAND
I_d
CONSTRAINED MODULUS
(Kg / sq cm)
M= 1/ m.v
UNDR. COHESION
(Kg / sq cm)
C_u (cohesive)
B.H. LOG
SAND & GRAVEL FILL
PEAT
SILTY CLAY
SAND
z

FIGURE 9



SITE INVESTIGATION SERVICES LIMITED

VICTORIA COUNTY ROAD No 21
PROPOSED STABILIZING BERMS

SCALE: 1" = 4 METERS
DATE: JUNE 1980
DRAWN: [signature]
FIGURE: 10

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(705) 743-6850

JOB
SHEET NO. OF
CALCULATED BY DATE
CHECKED BY DATE
SCALE

MUSKEG CONSOLIDATION CHARACTERISTICS

Victoria Road #21 Swamp 2579 02

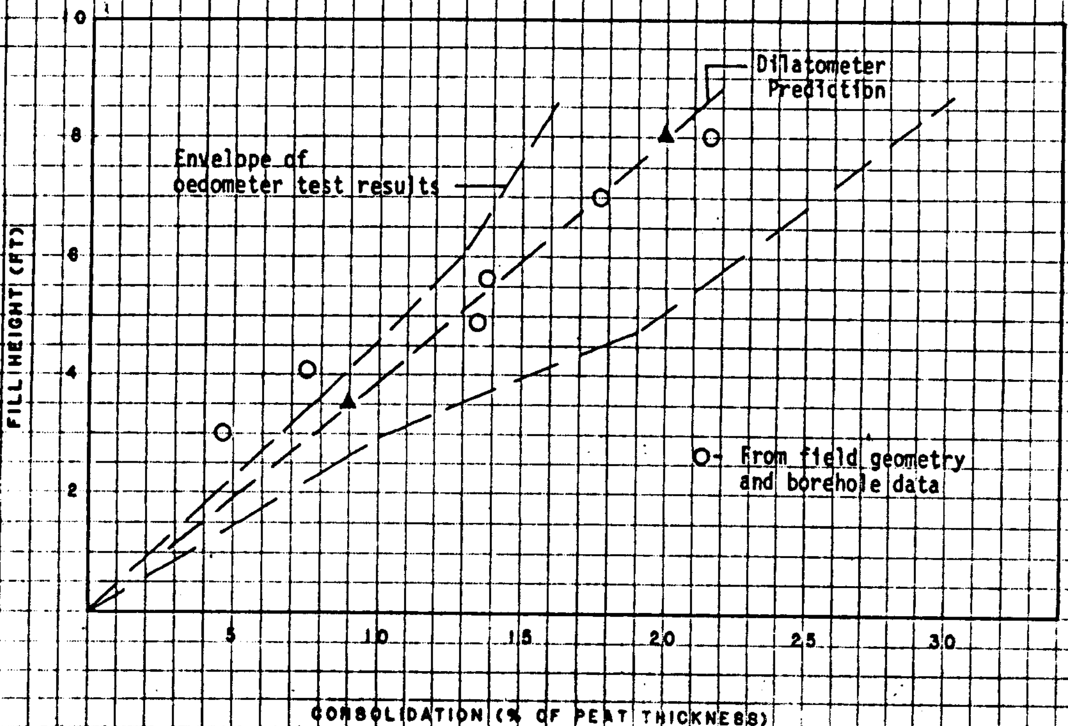

FIGURE 11

BACK

ACTUAL SETTLEMENT RECORD

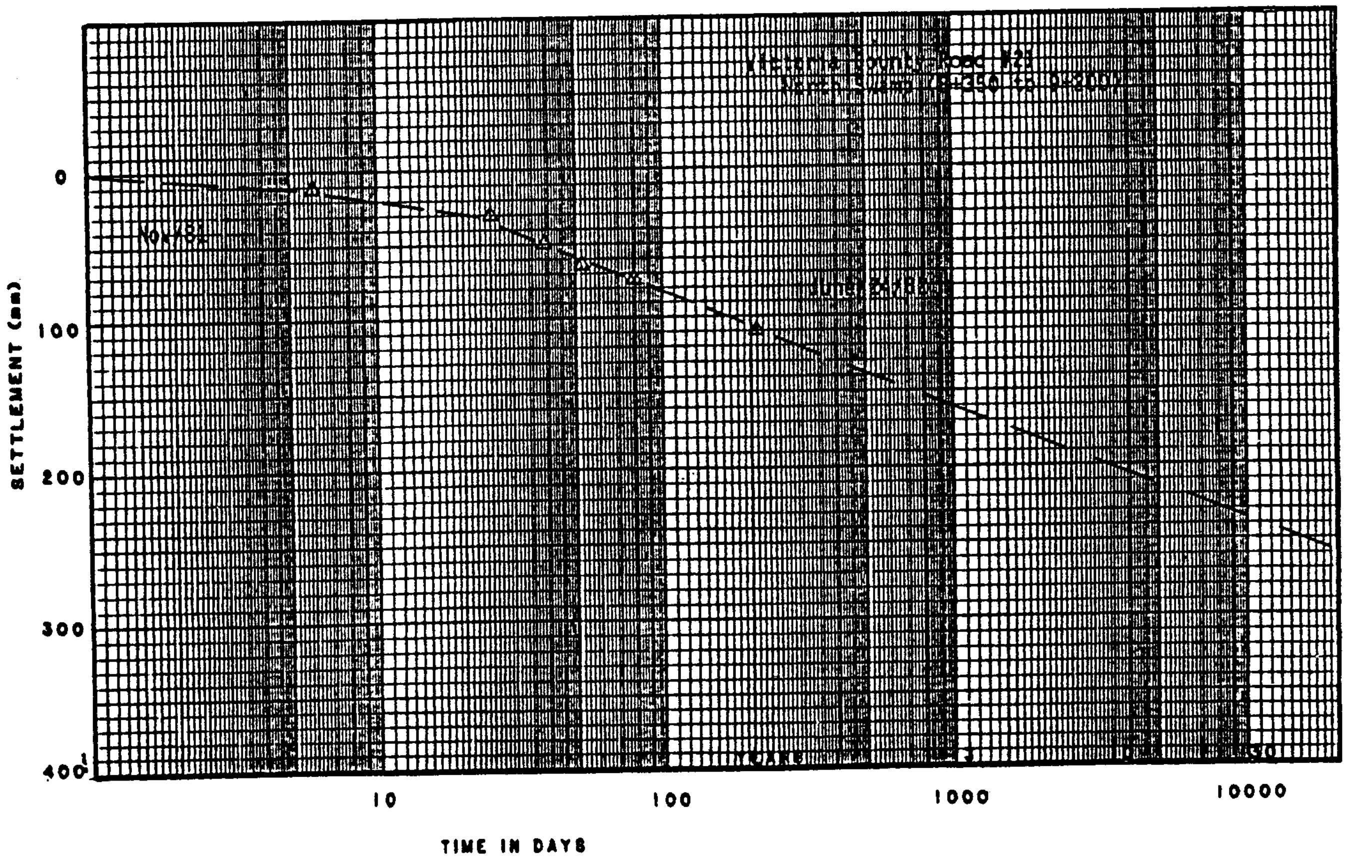

FIGURE 1

BACK

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CALCULATED BY DATE
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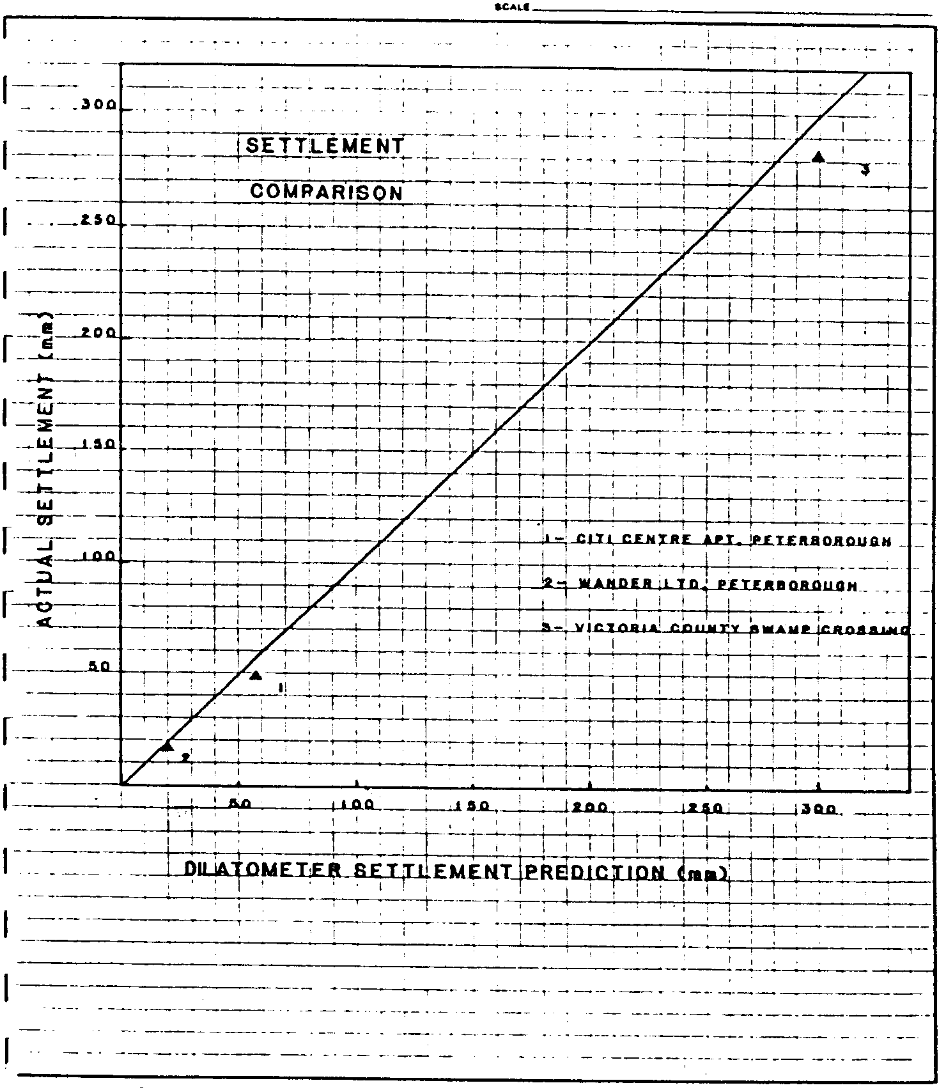


FIGURE 13

MEKECHUK

BACK

FLAT DILATOMETER USE ON C.N. RAIL LINE BRITISH COLUMBIA

John Mekechuk, P.Eng.
Senior Geotechnical Engineer
C.N. Rail
EDMONTON, Alberta

I wish to thank Mobile Augers for the invitation to participate in this conference. On the CNR our experience in the use of the dilatometer was limited, it was not on the research scale, nor elegant. It was perhaps quite basic but nevertheless we are starting to gain experience in the practical use of this insitu test procedure.

On the CN system we have an ongoing program in replacement of timber pile trestles as these reach the end of their service life. On the principal routes and lines which are being up-graded for heavier traffic loads and density, the policy is to replace the timber structures with a permanent type such as steel or concrete. During the original construction and subsequent replacement with timber, there was very limited exploration of sub-soil conditions. In many cases, when it came to replacing these structures, our people would simply refer to the previous pile driving records. Presently, since we are dealing with a different type of structure, with much heavier foundation loads the policy is to conduct a geo-technical exploration. In the exploration program we obtain information on the sub-soil conditions necessary for design of foundations. It is also our practice to bore-hole information to our forces or contractors who are bidding on the project.

In western Canada we have used the Dilatometer at four sites in conjunction with other in-situ sampling methods in the investigation for pile foundations in deep deposits of soft sediments. Three sites investigated were in marine clays near the west coast in British Columbia, and the fourth site was in fresh water glacial lake clays in north-western Ontario. The three sites in B.C. are near Prince Rupert where the railway follows the Skeena River and passes a number of tributaries generally at the confluence with the Skeena

BACK

River. I do not have any slides showing the typical terrain. From Terrace to Prince Rupert, the Skeena River flows in an east to west direction. The river valley is perhaps one half mile wide and the coastal mountains rise quite abruptly from the flood plain level. The transportation corridor is very narrow. Generally it is along the foot of the mountains and paralleling the river shore line. In many cases the B.C. Highway No. 16 and the railway are on a common sub-grade.

Our preliminary information was that the sediments in this area are fairly deep, that is in excess of 30 to 40 meters, and it was unknown whether these might be sands, clays or layered. It was decided to supplement our field investigation with the dilatometer test method. The feature that looked attractive to us was that the situ method would eliminate the need for special sampling equipment, obtaining numerous samples and then transporting them to the nearest laboratory which was perhaps four hundred miles away. The method provides a continuous profile of the subsoils and simplifies the definition of stratification. It also provides a continuous profile of undrained cohesion or the friction angle in the case of sands. By arrangement with Mobile Augers this firm provided the dilatometer test equipment and also trained our ground crew at one site. Our ground crew then carried out the dilatometer tests at the other two sites. The field readings were recorded on the standard DMT data forms which were then forwarded to our friend John Hayes for interpretation. John Hayes then provided us with the computer print out and profiles. In the investigation we also used the Geonor field vane at each site, as a back up procedure and for comparing the undrained cohesion obtained by both test procedures.

I now refer you to the data at the end of this presentation. This first site is at mile 63.4 Skeena Subdivision. (The Skeena subdivision starts with mile zero at Terrace and ends at Prince Rupert at mile 94). We carried out sampling and some filed vanes in the initial borehole then in the second borehole we carried out a continuous vane test. At the third borehole the dilatometer test was carried out. The drilling equipment

BACK

was situated on the bridge deck over the stream. At this site the first nine and a half meters of the bore-hole was cased through sands and gravels. The Dilatometer test probe was then pushed through below the casing depth compared the field vane data with the undrained cohesion obtained by the DMT test and these correspond very closely. The sampling, showed presence of shells in this area and possibly reflected higher vane readings due to obstructions. The sensitivity from the vane test is from 3 to 4 and the C over P ratio, that is the ratio of the undrained cohesion to the effective overburden stress was 0.29 and is quite typical for normally consolidated clays.

The second site is about 28 klm east of Prince Rupert, mile 73.3. From the vane test the sensitivity was 6 to 10. At this site the upper part of the bore-hole was not cased. There is fairly good correspondence between the field vane and the Dilatometer interpretation. The C over Phi ratio works out to 0.28.

The third site is mile 81.5. It is about 14 miles east of Prince Rupert. The upper nine and a half meters were cased though sands and some gravels. The red dots represent the field vane tests. Here there are a couple of vane tests which are higher than the DMT and this is possibly due to obstructions by pebbles in the clay. The sampling identified presence of some shells at this level and yielded slightly higher vane readings. At this level the Dilatometer blade was slightly bent when dense sands and gravels were encountered. The blade was straightened out for further testing. It required about two hours to take each Dilatometer profile, and to test the same depths with a vane at 0.9 meter intervals would have required about 6 to 8 hours. For pile design we used the undrained cohesion profile as obtained from the Dilatometer test method.

The fourth site where we used the Dilatometer test method was at mile 77.0 on the Fort Frances Subdivision. This location is 20 klm. east of Fort Frances and north of the U.S. border. At this site we have a pile timber trestle which is 207 meters in length which crosses a narrow neck on Rainy Lake. The red dots indicate the field vanes which were taken in close proximity to the Dilatometer test site. The lacustrine clay is underlain by about three meters of till, and in turn overlies bedrock. This is quite typical in north-western Ontario where depressions in the shield area are filled with highly plastic lacustrine clays.

BACK

When we look at the comparison of vane tests and undrained cohesion by the DMT you will notice a very close correspondence. From the undrained cohesion and index tests we could interpret that there are two successions in lacustrine clays. The upper clay layer showed a liquid limit of 115% with a water content of 105%. The sensitivity is 6 from the vane test, and the C over P ratio is 1.6 which is very high. The DMT print out results indicated an over consolidation ration 7.6 to 3.8 decreasing with depth. These OCR numbers appear to be fairly high. Our impression was that these clays are normally consolidated or slightly over consolidated. A single one-dimension consolidation test was carried out which gave an over-consolidation of 4.75. A visual examination with the aid of magnification showed that the upper clay was columnar like in structure or might have experienced dessication. This is difficult to believe because the clay is submerged and presumably has been during its history. The lower clay layer, below nine meters, has a liquid limit of 90%, and the water content of 80%. The sensitivity from the vane test was 4, and the C over P ratio is .54. A single one-dimensional consolidation test yielded an OCR of 2.0 and the DMT print-out data for OCR ranged from 2.0 to 1.0, decreasing with depth. A second Dilatometer test was carried out at the site, and the results are very similar.

We foresee that in the future we could be expanding the use of the Dilatometer to problem areas where we are experiencing ongoing settlement of fills on soft ground. Our interest is in the practical use of the DMT and a seminar such as this one is very useful in the discussions of experience and limitations of this test procedure.

DATA FROM CN TEST SITES

1. Test site Mile 63.4 Skeena Subdivision 40 km. east from Prince Rupert.

   Wet density 17.29 KN/m^3
   Liquid limit 39% Plastic limit 23% (Average
   Water content 35% average

   Sensitivity 3 to 4 from vane test
   0 - 27 m   c/p = 0.29

   DMT interpretation

0 - 27 m	O.C.R.	1.5 to 2.0
	K_0	0.7 to 0.8
27 - 40 m	O.C.R.	1.0 to 1.5 decreases with depth
	K_0	0.5 to 0.7 decreases with depth

BACK

2. Test site Mile 73.3 Skeena Subdivision 28 km. east from Prince Rupert.

Wet density 16.97 KN/m^3
Water content 34% average
Visual classification - medium to low plasticity

Sensitivity 6 to 10 from vane test

0 - 50 m c/p = 0.28

DMT interpretation
O.C.R. 1.3 to 2.0 decreases with depth

4 - 10 m K_0 0.75
10- 40 m K_0 0.66 fairly constant with depth

3. Test site Mile 81.5 Skeena Subdivision 14 km. east from Prince Rupert

Wet density 17.17 KN/m^3
Water content 42% to 48%
Visual classification - medium plastic with layers highly plastic

Sensitivity 4 to 10 from vane test.

0 - 30 m c/p = 0.22

DMT interpretation

O.C.R. 0.8 to 1.1 increases with depth
K_0 0.45 to 0.55 increases with depth

4. Test site Mile 77.0 Fort Frances Subdivision.
20 km. east from Fort Frances, Ontario.
207 m. crossing on point of Rainy Lake.

Lake level 0.0. m. DMT datum
Lake bed 3.6 m.

(1) Upper clay layer 3.6 to 9.0 m

Wet density 14.0 KN/m^3
Water content 105% average
Liquid limit 115% Plastic limit 40%

Sensitivity 6 From vane test

c/p = 1.6

One-dimensional consolidation test
Depth 4 m below lake bed level

e = 3.43 C_c = 0.73
O.C.R. = 4.75

DMT interpretation
O.C.R. 3.8 to 7.6, decreases with depth
K_0 1.0 to 1.5, decreases with depth

BACK

(2) Lower clay layer 9.0 to 16.0 m

Wet density 15.1 KN/m^3
Water content 80%
Liquid limit 90% Plastic limit 30%

Sensitivity 4 from vane test

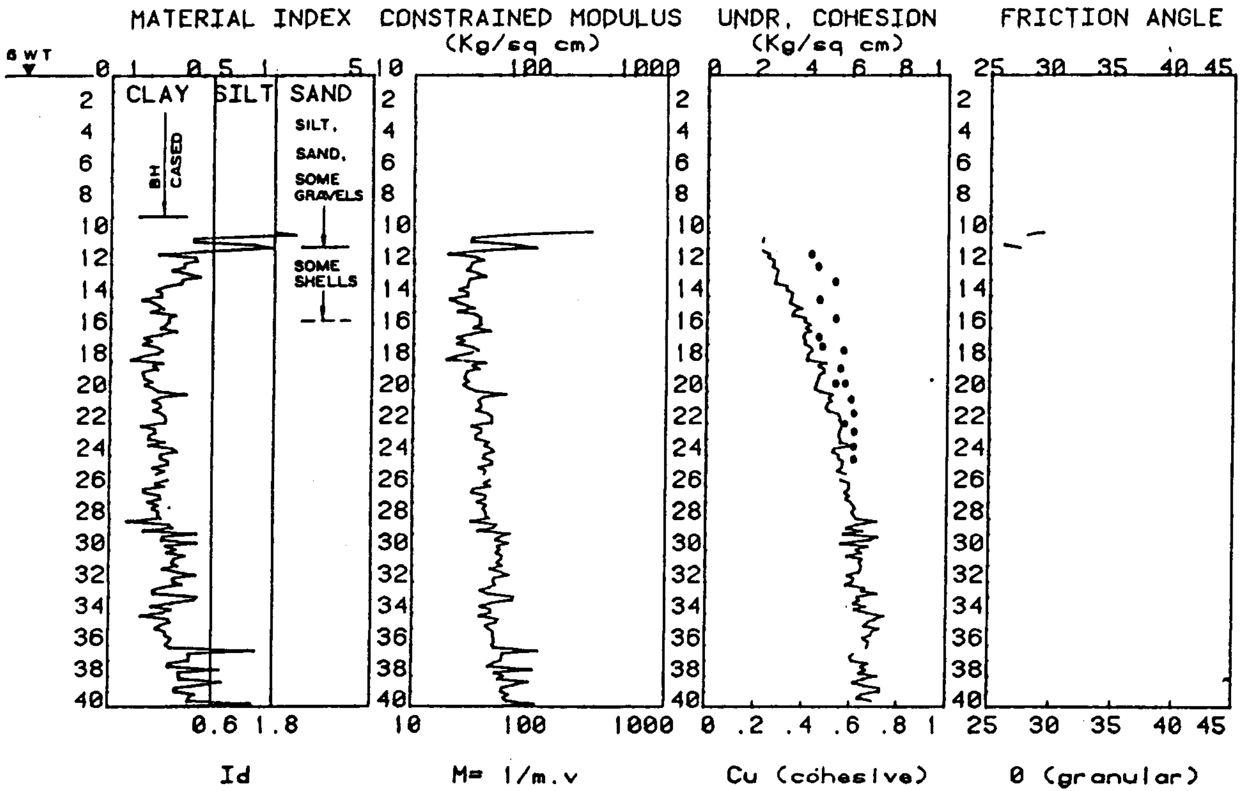
c/p = 0.54

One dimensional consolidation test
Depth 7.5 m. below lake bed level

e = 1.54 C_c = 0.28
O.C.R. = 2.0

DMT interpretation
O.C.R. 2.0 to 1.0, decreases with depth.
K_0 0.8 to .6, decreases with depth.

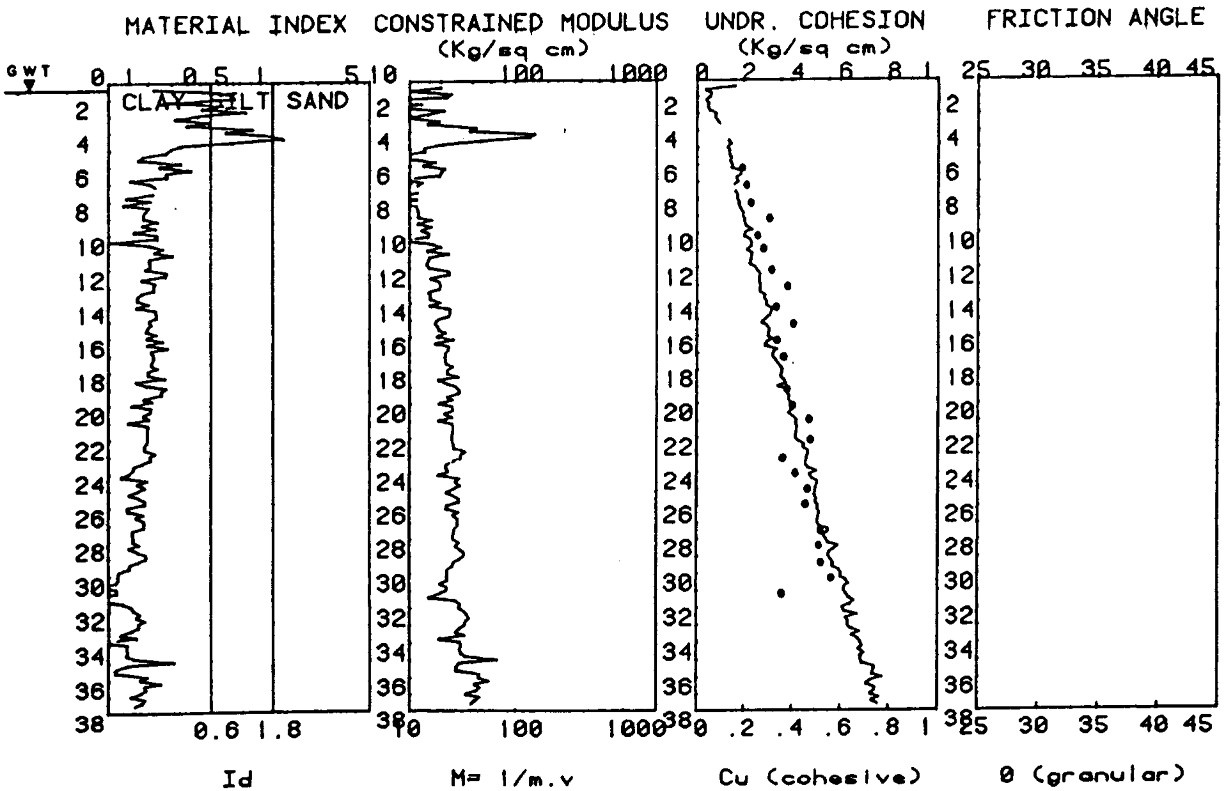
BACK



CN MILE 63.4 SKEENA SUB. BH 63.4-2

• FIELD VANE BH 63.4-1, BH 63.4-5

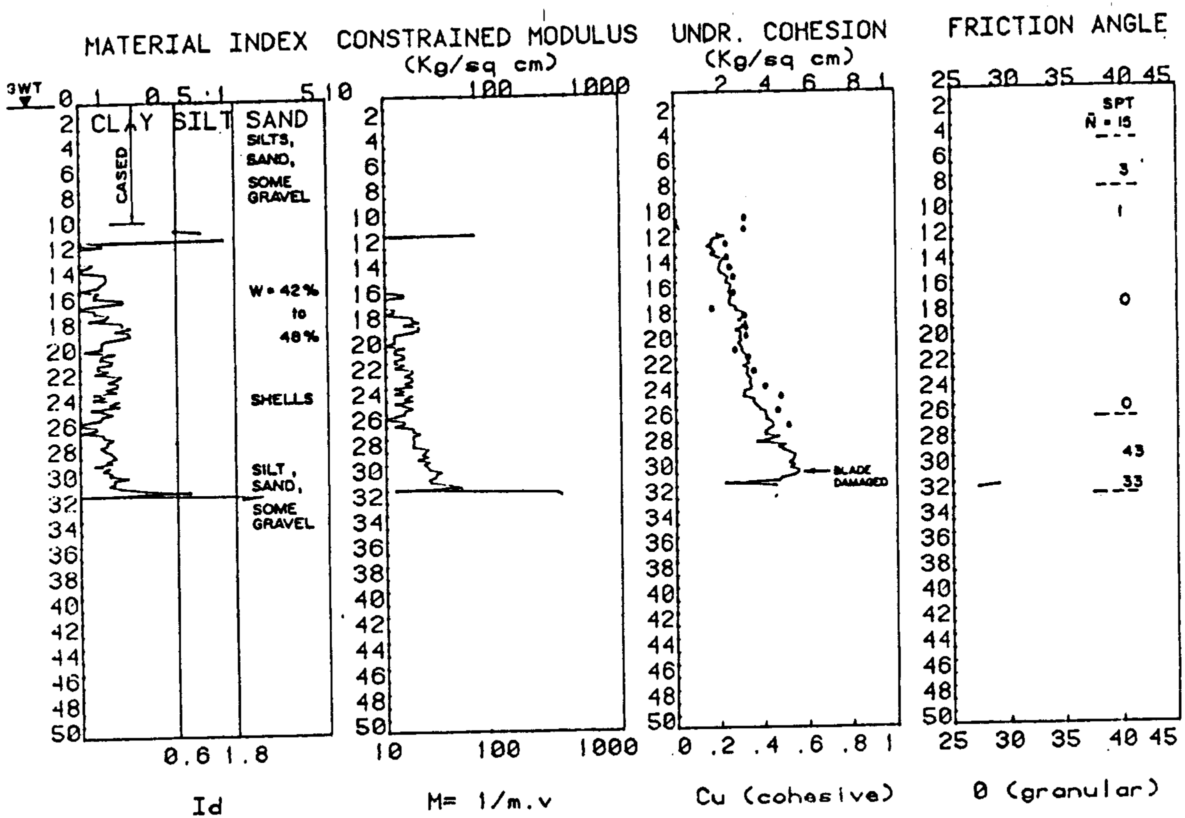
BACK



CN MILE 73.3 SKEENA SUB. BH 73.3 - 3

• FIELD VANE BH 73.3 - 2

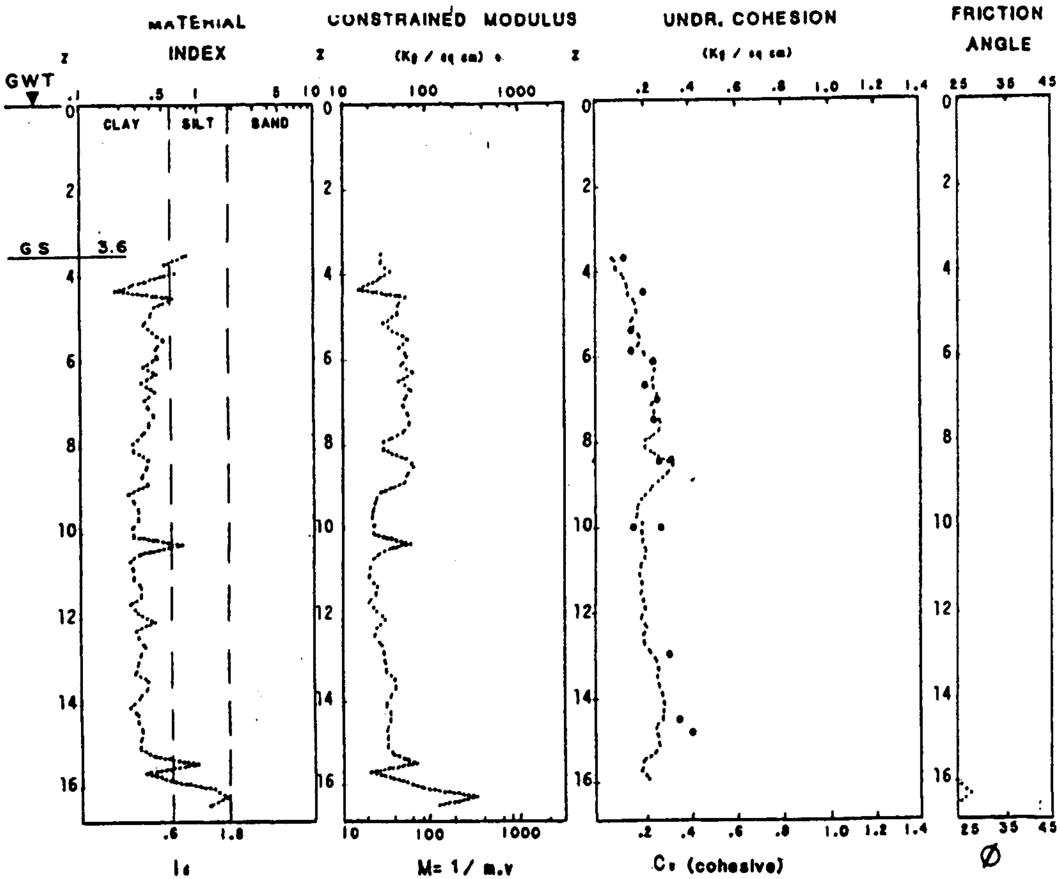
BACK



CN MILE 81.5 SKEENA SUB. BH 81.5 - 3

• FIELD VANE BH 81.5 - 3

BACK



CN MILE 77.0 FT. FRANCES SUB. BH 106
• FIELD VANE

BACK

BURGESS

BACK

USE OF THE FLAT DILATOMETER IN THE BEAUFORT SEA

Neil Burgess, P.Eng.
Hardy Assoc. (1978) Ltd.
Calgary, Alberta

ABSTRACT

This paper describes the design and development of a motion-compensated geotechnical drill rig that was tailored specifically for site investigation work in the Canadian Beaufort Sea. Fieldwork was undertaken during the summers of 1981 and 1982 at potential sites for caisson-retained exploration islands (drilling platforms). The approach to site investigation is described, together with the results of laboratory and insitu testing. The drill rig proved to be reliable and the unique motion compensation system permitted down-hole work to be done in a manner that is comparable to on-shore methods. The value of using a variety of in situ tools is apparent when on-situ decision making is necessary and the test results prove the inadequacy, in certain circumstances, of reliance on laboratory test data. The flat-dilatometer is a relatively new in situ instrument and the results of its first-time application in the Beaufort are encouraging.

INTRODUCTION

The necessity of offshore geotechnical engineering in the Canadian Beaufort Sea has increased in recent years and is likely to continue in the future as commercial production becomes possible. To date, site investigations have been undertaken for temporary exploration structures.

Project work described in this paper was undertaken for Esso Resources Canada Limited in connection with foundation studies for caisson-retained islands. A short open-water season, relatively poor weather and costly support facilities place limitations on the site investigation methods and the speed with which site

BACK

evaluation can be undertaken is of the essence. It is necessary to utilize highly reliable drilling equipment and to resort to in situ test procedures which yield geotechnical information in a minimum amount of time. In this regard, a motion-compensated drill rig has been developed and is described, together with an approach to site investigation, the use of in situ test equipment and a brief comparison of test results from various down-hole tools.

GEOTECHNICAL DRILL RIG

Mobile Augers and Research Ltd. (MARL) completed construction of their DGD-2000 rig in a six week period during 1981. It is a rotary top-drive model that incorporates the draw works only of a Damco 2000. The power train, control system, mast and mud pumping system were designed and fabricated by MARL. Heave compensation was incorporated from designs by Seacore Ltd. of the U.K. and they assisted in the fabrication. Heave compensation was necessary to meet the requirements for soil sampling and testing. The system is a tension-leg concept and is described in principle in Figure 2. The drill rig is illustrated in Figure 3 and its technical specifications are described in Table I.

The rig has a depth capacity of at least 600 m and was fitted with Christensen drill pipe and wire-line systems. In principle, the heave compensation consisted of a 10 tonne, cable-supported clump weight at the sea floor connected by cable to a winch at the base of a "ladder" type frame within the drill mast. In turn, the ladder was retained vertically by cable over a sheave at the top of the mast and connected to an hydraulic ram controlled by a nitrogen-activated accumulator. The ram had a two metre stroke and maintained a constant upper cable force of 4 tonnes, to support the ladder and rotary hydraulic power head. The power head could be locked into the ladder at any elevation or allowed to float. In the locked position, the power head (and drill string) could be maintained at a constant elevation above the sea floor. The ladder moved vertically with respect to the mast, in direct conformance with heave of the drill ship. Flow control within the accumulator was sufficient

BACK

to produce constant-elevation tolerance of plus or minus 3 mm during ship heave of about 1 m.

The optional lock-in of the power head to the ladder permitted rapid drilling between sample depths when the head and drill string were allowed to float or to penetrate under their own weight at an unrestricted rate.

The system has proved reliable with down-time for mechanical reasons being less than two hours in some four months of operations. Of this period down-time for weather amounted to about 25 percent of available time.

Drilling was undertaken from a 55 m anchor-handling tug equipped with a four-point anchoring system. Syledis positioning systems were employed. A photograph of the ship is presented in Figure 1.

SITE INVESTIGATION METHODOLOGY

The Caisson Retailed Island (CRI) concept has been described by others (Mancini et al, (1983). It consists of eight steel caissons that are ballasted onto a sand berm in water depths varying from 15 to 25 m to form an octagonal structure which is infilled with sand, to provide a 91 m diameter drilling platform. Time constraints and the relative uniformity of subsoil conditions at particular sites in the Beaufort were such that the number of borings at a site could be reduced to a minimum of four - one at the island centre and three on the perimeter. Provided that subsoil uniformity became evident from these, the number was limited to four. In principle, the objective was to obtain data within 100 to 150 m below the mudline at the island centre, and to depths of 60 m at the circumference.

Anchoring time varied from 1.5 to 6 hours and was affected mainly by the sea state. Auxiliary tugs were used on occasion to reduce anchoring time and to ensure straight cables. Pronounced anchor cable curvature resulted in gradual movement off position as cable tension produced straightening of the lines where mudline segments were soft and cohesive.

A series of borehole work was undertaken at each of the four principal locations, beginning with conventional sampling with 75 mm shelby tubes (600 and 1500 mm lengths)

BACK

and/or down-hole hammered split-spoon SPT samplers. Stratigraphy was obtained in this manner and decisions could be made with respect to in situ test intervals. Wire line retrieval permitted drilling and sampling to a depth of 150 m within a 24 hour period, the sample intervals varying progressively from 1 m to 6 m with increasing depth.

With an allowance for adverse weather, a 17 day period was budgeted for site work at CRI locations. On occasion, dredge scheduling reduced this allowance substantially and to maintain a construction schedule, all drilling, sampling and testing had to be completed in a four day period at which time the site had to be designated as acceptable or otherwise, with respect to its foundation conditions. A primary concern in the island design centres on the thickness, strength and compressibility of weak sediments at the mudline. Weak, cohesive soil samples that are obtained below ater depths of as little as 15 m are subject to total stress changes that can be sufficient to result in severe disturbance, particularly if the silt content (and permeability) is moderate to high. Miniature laboratory vane shear tests on such samples will yield unrealistically low undrained shear strengths, strengths that can be a small fraction of those obtained with the cone penetrometer, the pressuremeter, the field vane or the flat dilatometer. Reliance on strength and compressibility parameters from in situ tests becomes mandatory in these cases if decisions are to be made on-site with respect to the acceptability of the proposed island foundations.

Drilling, sampling and testing to the above mentioned depths and at four borehole locations was accomplished in as little as seven 12 hour shifts in which time the foundation conditions were defined. If rated acceptable, berm construction commenced immediately.

IN SITU TEST EQUIPMENT

Piezo-cone penetrometers, self-boring pressurmeter and field vane equipment were provided and operated by Situ Technology Inc. (STI). The penetrometers and the pressuremeter were electronic, utilizing solid state circuitry and force/pressure

BACK

sensing components. Data acquisition was accomplished on Apple II micro computers, recorded digitally on floppy disk, displayed on video components as the tests proceeded and recorded on thermal paper printers. The down-hole equipment is of STI's design and manufacture. The pressuremeter and cone penetrometer probes are similar, in principle, to those described by Hughes et al (1977), Clough and Denby (1981), Jones and Van Zyl (1981) and Campanella and Robertson (1981). The field vanes incorporated an electronic torque measuring cell within 600 mm of the vane, to remove the influence of rod friction and to obtain a continuous, time based record of the test. Torque was applied through a conventional Nilcon head.

The flat-dilatometer is described by Marchetti (1980), discussed by Schmertmann (1981) and the components are depicted in Figure 4. The probe can be pushed or driven into soils that vary widely in strength, density, compressibility and composition. The test is relatively new to North America and results in the Beaufort have been encouraging. The distinct advantages with regard to offshore investigation pertain to its ease of use, reliability, borehole independence and the speed with which soil profiling can be accomplished. Test intervals can be as little as 200 mm and the test duration is less than 1 minute.

Relatively thick deposits of cohesive soils are common to the Beaufort, at the sea floor, and the definition of undrained shear strength is important with respect to an assessment of island stability. It is considered necessary to obtain this parameter, in particular, in a variety of ways such that strength envelopes can be defined.

Cone pushing was done at the mudline and at deeper zones of special interest. Pressuremeter testing was undertaken in pre-bored holes and by controlled mud-jetting. A self-boring capacity was incorporated but was not required at all sites. The dilatometer was pushed from the mudline and driven by a down-hole hammer at significant depths.

COMPARISON OF TEST RESULTS

Test results are shown for three sites on Figures 5 to 9 and focus mainly on the

BACK

determination of undrained shear strength and soil stiffness obtained from laboratory and in situ testing. The data were obtained at several locations in water depths varying from 11 to 26 m. The results are presented primarily to demonstrate the variation in soil parameters that is obtained from the various methods.

The following observations are made with respect to the test results:

1. Shear strength obtained in a thick deposit of soft to firm, highly plastic clay from site 1 is shown in figure 5. Cone penetrometer, flat-dilatometer, miniature lab vane and quick triaxial test results are compared. Lab vane results are seen to be substantially lower and reasonably good correlation is obtained for dilatometer and cone tests. An apparent decrease in strength with depths (32 to 39 m depths) is indicated from cone data. A cone factor (Nk) of 11 was used and the tip resistance was reduced by effective overburden pressure. The reduction of tip resistance by overburden pressure can result in an apparent and unrealistic decrease in shear strength.
2. The variation in each strength profile, with depth, is attributed in part to a variation in organic content within the soil stratum. Index and classification properties were obtained from Shelby tube samples at depth intervals varying from 1 m to 3 m. Average values of index properties for the 3 sites are included in Table II.
3. Triaxial strengths correlate reasonably well with Cu/p' ratios of 0.25 to 0.30. The in situ results suggest over consolidation within the upper portion of the profile.
4. Lab vane, field vane and dilatometer test results are compared in Figure 6. Data in Figures 5 and 6 are from two borings at Site 1. The borings are some 100 m apart.
5. Lab vane strengths are about one half of field vane values and about one quarter of strengths predicted by the dilatometer. In relation to the Cu/p' ratios, the lab vane values are unrealistic. Relatively poor correlation was obtained for field vane and dilatometer test results, in this case.

BACK

6. Constrained modulus data from oedometer and dilatometer tests at Site 2 are compared in Figure 7. Site 2 has a 15 m water depth. The tests were obtained for medium plastic silty clay having an average undrained shear strength of 40 kPa. Although the data are limited, the correlation appears reasonable.
7. Deformation modulus values (E) from the Menard pressuremeter and the flat dilatometer are compared in Figure 8 and were obtained at Site 3. These were obtained in soft to firm, medium to highly plastic clay and in clayey silt. In general, the modulus values are comparable.
8. Software is available for the reduction of dilatometer test results and the data are easily reduced, plotted and presented in the format shown in Figure 9. These results were obtained at Site 2. In addition to the data that are plotted, the test yields an estimate of Ko, over-consolidation ratio and the angle of internal friction for cohesionless soils.

CONCLUSIONS

The motion compensated rill rig proved to be exceptionally well suited to offshore geotechnical work and permitted in situ testing to be done with a facility that is comparable to land-based methods. The use of the in situ equipment is considered to have resulted in a much superior assessment of foundation conditions than is possible from laboratory testing alone. The flat-dilatometer in particular is considered to have produced reasonable results and its obvious advantages should result in extensive use of the equipment in offshore investigations.

The approach to site investigation that is described in the paper is considered to have resulted in a good balance between satisfaction of the geotechnical requirements while operating within economic and environmental constraints that are peculiar to the Canadian Beaufort Sea.

BACK

NOMENCLATURE

E	=	Deformation Modulus, analogous to Young's Modulus
Cu	=	undrained shear strength
SPT	=	Standard Penetration Test
Cu/p'	=	ratio of undrained shear strength to effective overburden pressure
kPa	=	Kilopascals (1 kPa = 20.89 lb/ft^2)
Ko	=	the ratio of horizontal to vertical effective stress

ACKNOWLEDGEMENTS

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BACK

TABLE I

MARL DGD-2000 DRILL RIG DATA

Mast:	12 m tubular steel frame, 30 tonne capacity
Main Power:	Detroit Diesel 6V53
Transmission:	Allison 6 Speed
Draw Works:	Double drum winches with air clutches, 8.6 tonne single line pull
Rotary Drive:	Top drive head, max. 125 RPM
Motion Compensation:	Tension-leg, nitrogen activated accumulator
Break-Out Table:	Mechanical drive, motion compensated
Mounting:	12.2 m Hi-Boy tandem trailer with 0.6 m power sub-base slide
Gross Weight:	16 tonnes
Mud Pumps:	Independent Lombardini diesels on FMC single acting triplex high pressure pumps (3 units on manifolded discharge lines)
Clump Weight:	10 tonne gross weight with re-entry guide.

TABLE II

	SITE 1	SITE 2	SITE 3	
SOIL	CLAY	CLAY	CLAY	SILT
Classification	CH	CI	CI-CH	ML-OL
Bulk Density (kg/m^3)	1700	1850	1900	2000
Liquid Limit	58	45	40-60	40
Plastic Limit	22	25	20-30	30
Plasticity Index	36	20	20-30	10
Water Content	55	35	30-55	35
Soil Temperature (C.)	3	5	5	5
Average Cu (kPa)	20	40	20-60	

BACK

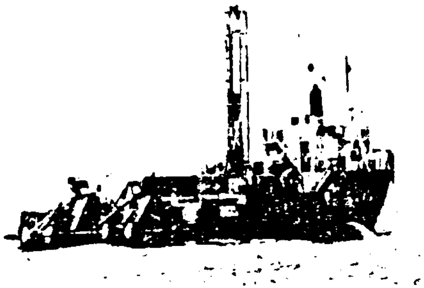

Fig. 1—Drill ship at anchor.

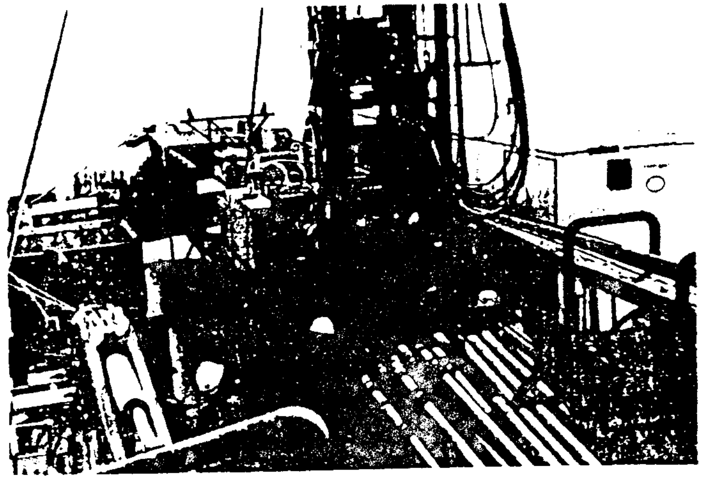

Fig. 3—DGD-2000 drill rig.

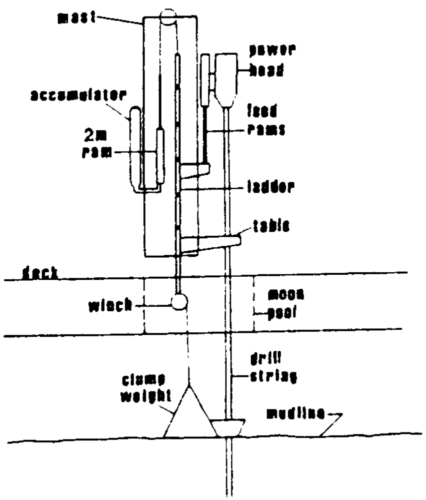

Fig. 2—Heave compensation (schematic)

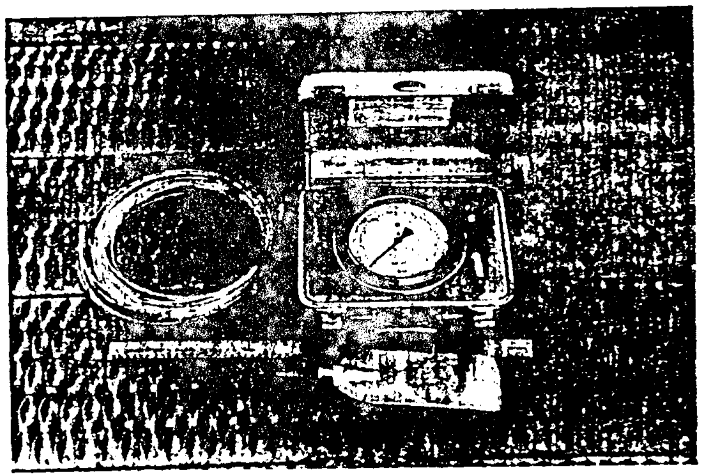

Fig. 4—Flat dilatometer components.

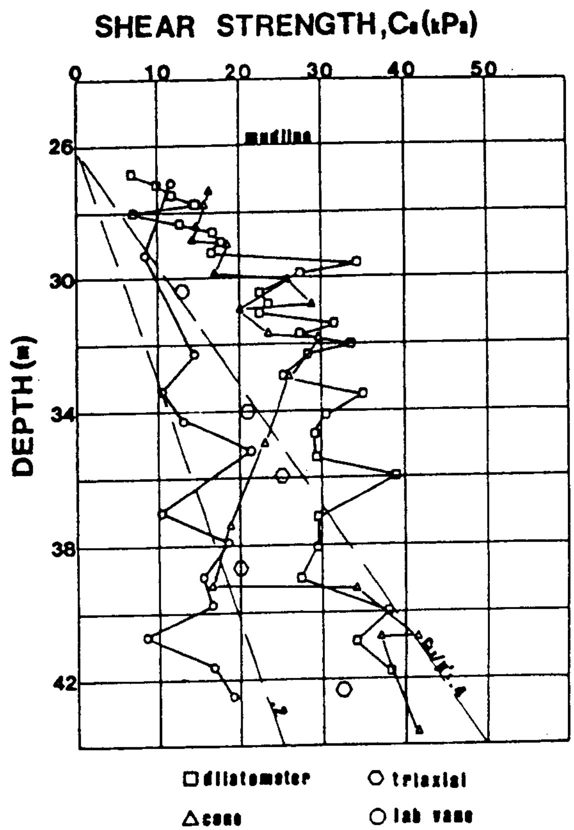

Fig. 5—Site 1—shear strength (Hole 1).

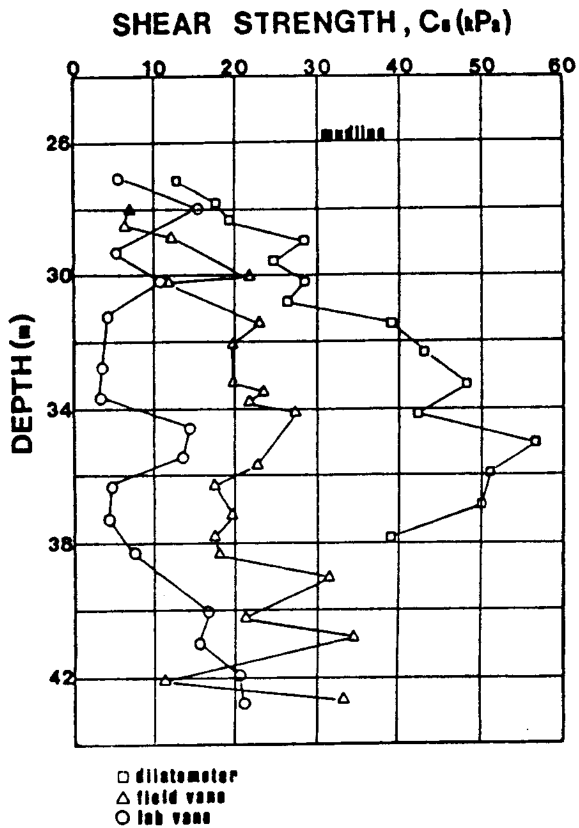

Fig. 6—Site 1—shear strength (Hole 2).

BACK

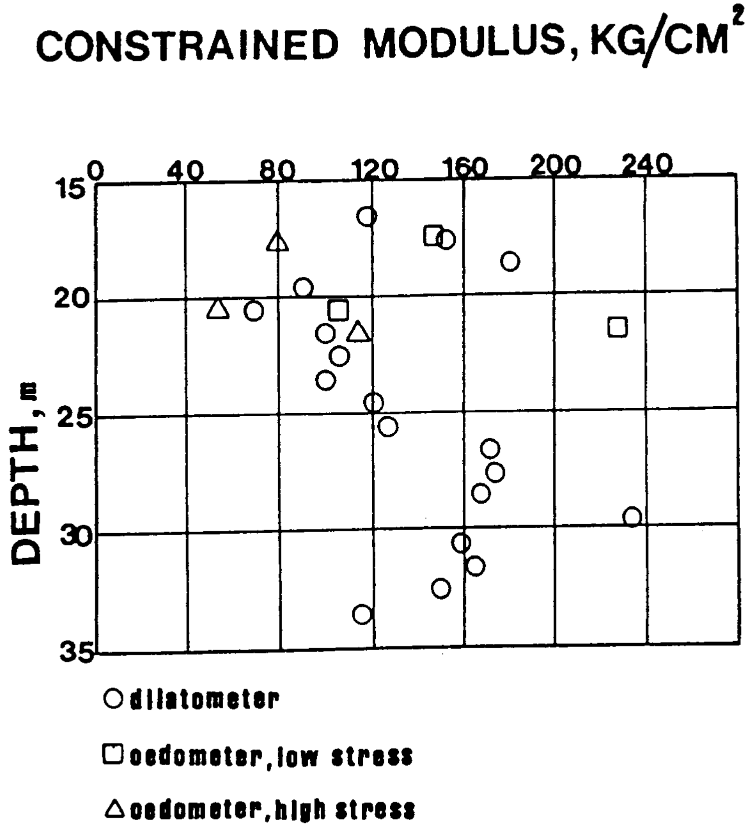

Fig. 7—Site 2—constrained modulus.

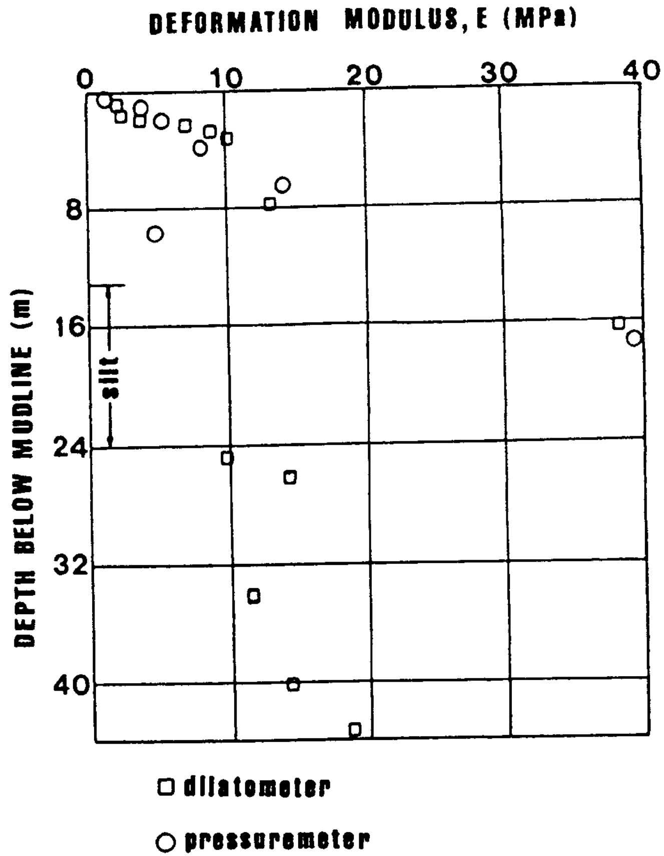

Fig. 8—Site 3—deformation modulus.

CAMPANELLA

BACK

Flat Plate Dilatometer Testing: Research and Development at UBC

R.G. Campanella and P.K. Robertson

Introduction

The measurement of soil properties by in-situ test methods has developed rapidly during the last decade. In-situ testing is a very large field, and the Flat Plat Dilatometer Test (DMT) has a role to play.

In-situ test methods currently available can be divided into two basic groups,

i) Logging methods,

ii) Specific test methods.

The logging methods are usually pentration type tests and are usually fast and economic, and usually provide qualitative estimates, based on empirical correlations, of various geotechnical parameters. Specific test methods are usually more specialized and, therefore, often slower and more expensive to perform than the logging methods. The specific test methods are usually carried out to obtain specific soil parameters, such as shear strength or modulus. The two basic groups are often complimentary in their use. The logging method is best suited for stratigraphic logging and preliminary evaluation of soil parameters. The specific test methods are best suited for use in critical areas, as defined by the logging methods, where more detailed assessments are required of specific soil parameters, which of course may include undisturbed sampling and laboratory testing. The logging method should therefore be fast, economic, continuous and most important, repeatable. Whereas, the specific method should be better suited to fundamental analyses to provide the required parameter. One of the best examples of a combination of logging and specific test methods is the static cone and the pressuremeter.

Table 1 presents an updated version of the table presented by Mitchell et al (1978) of in-situ test methods and their applicability. Each method is listed in approximate order of its cost or complexity and with its suitability for determining various different geotechnical parameters. The suitability of each method for determining various different parameters is indicated by a grade of A, B or C, with A indicating high applicability and a blank indicating litter or no applicability. The grade is based on a qualitative evaluation of the confidence level assessed for each method in determining the various geotechnical parameters. The test methods listed in the upper half of the table tend to be logging methods, whereas the methods in the lower half tend to be specific methods.

The authors consider the DMT as a logging tool since it provides measurements every 20 cm. The DMT data is also interpreted based on empirical correlations.

CURRENT RESEARCH AT UBC

A quick perusal of Table 1 indicates that currently it appears that the selfboring pressuremeter is probably the most applicable specific test and the electric piezometer-friction-bearing cone the most applicable logging method. Following close behind are the dilatometer (logging tool) and the screw plate (specific test). Of course the field vane is still the most common way to find undrained strength and the Standard Penetration Test (SPT) the most used in situ test which has the virtue of usually obtaining a specimen of soil. All of these techniques are under active

BACK

	SOIL TYPE	PROFILE	DENSITY (D_r)	ANGLE OF FRICTION (ϕ)	UNDRAINED SHEAR STRENGTH (c_u)	PORE PRESSURE (u)	STRESS HISTORY (OCR)	MODULUS: YOUNG'S & SHEAR (E&G)	COMPRESSIBILITY (m_v & C_c)	CONSOLIDATION (c_h & c_v)	PERMEABILITY (k)	STRESS STRAIN CURVE	LIQUEFACTION RESISTANCE
Dynamic cone	C	A	B	C	C	-	C	-	-	-	-	-	C
Static cone:													
Mechanical	B	A	B	C	B	-	C	B	C	-	-	-	B
Elec. Friction	B	A	B	C	B	-	C	B	C	-	-	-	B
Elec. Piezo	A	A	B	B	B	A	A	B	B	A	B	B	A
Elec. Piezo/Friction	A	A	A	B	B	A	A	B	B	A	B	B	A
Acoustic probe	C	B	B	C	C	-	C	C	-	-	-	-	C
Dilatometer	B	A	B	C	B	-	B	B	C	-	-	C	B
Vane Shear	B	C	-	-	A	-	B	-	-	-	-	-	-
Standard Penetration Test	B	B	B	C	C	-	-	-	C	-	-	-	A
Seismic CPT downhole	C	C	C	-	-	-	-	A	-	-	-	B	B
K_o Blade	-	-	-	-	-	-	B	-	-	-	-	-	-
Resistivity Probe	B	B	A	B	C	-	C	C	C	-	-	C	A
Borehole Permeability	C	-	-	-	-	A	-	-	-	B	A	-	-
Hydraulic fracture	-	-	-	-	-	B	B	-	-	C	C	-	-
Screw Plate	C	C	B	C	B	-	B	A	B	C	C	B	B
Seismic downhole	C	C	C	-	-	-	-	A	-	-	-	B	B
Impact cone	C	B	C	C	C	-	C	C	C	-	-	-	C
Borehole shear	C	C	-	B	B	-	C	C	-	-	-	C	-
Menard Pressuremeter	B	B	C	B	B	-	C	B	B	-	-	C	C
Selfboring Pressuremeter	B	B	A	A	A	A	A	A	A	A	B	A	A
Selfboring devices:													
K_o meter	-	-	-	-	-	-	A	-	-	-	-	-	-
Lateral penetrometer	C	C	B	B	B	-	B	B	C	-	-	-	-
Shear vane	B	C	-	-	A	-	B	-	-	-	-	-	-
Seismic crosshole	C	C	B	-	-	-	-	A	-	-	-	B	B
Nuclear tests	-	-	A	B	-	-	-	C	-	-	-	-	C
Plate load tests	C	C	B	B	C	-	B	A	B	C	C	B	B

A - High applicability
B - Moderate applicability
C - Limited applicability

TABLE 1 - Perceived Applicability of In-Situ Test Methods - Update 1982

(After CAMPANELLA and ROBERTSON, 1982)

BACK

research at UBC including the newest development, a seismic CPT downhole device.

Cone Penetration Testing -- The electrical cone has been under research at UBC since 1977. A 5-channel cone has been developed at UBC that enables continuous monitoring of bearing, friction, pore pressure, slope and temperature. The dimensions conform to the European Standard for electric cones. Full details of the cone and associated equipment are given by Campanella & Robertson (1981).

The addition of pore pressure measurements during cone penetration testing has added a new dimension to the interpretation of geotechnical parameters particularly in loose or soft, saturated deltaic deposits. The continuous measurement of pore pressures along with bearing and friction has enhanced the electric cone penetrometer as the premier tool for stratification logging of soil deposits. The ability and experience to interpret cone data has also reached a stage such that an impressive array of geotechnical parameters can be estimated from empirical correlations (Robertson & Campanella, 1983). Present cone research at UBC is aimed at improving the state-of-the-art in cone design and interpretation and understanding the pore pressure measurements from piezometer cones.

SPT -- The Standard Penetration Test, with all its problems of repeatability and reliability, is still the most commonly used in-situ test today. However, with a better understanding of the dunamics of the SPT (Schmertmann and Palacios, 1979) and the existence of a fairly inexpensive and easy to use energy calibration unit, it is our belief that all SPT results should be corrected for a given standard energy. Kovacs et al. (1981) and Robertson et al. (1983) have shown that energies can vary from

BACK

about 20% to 90% of the theoretical maximum. Schmertmann has suggested that based on limited data, an efficiency of about 55% appears to be the norm for which it is believed that many current North American correlations were developed. Hopefully, energy corrected N-values will soon become commonplace and correlations may become more meaningful. It is hoped that measured energy correction factors will lead to more repeatability and reliability of N-values in the future. Recent research has been directed toward improving our understanding of the correlation between SPT and CPT data (Robertson et al. 1983).

Selfboring Pressuremeter Test (SBPMT) -- This specific test appears to give the most accurate direct determination of the required soil parameters for computer analyses. These soil parameters include in-situ stress, stress-strain response, effective stresses, volume change characteristics and shearing resistance. The pressuremeter must selfbore itself into position without altering in-situ stresses and disturbing the soil. The greatest problem appears to be the high cost associated with installation with minimum disturbance and the need for highly trained personnel. Still, it is currently being successfully used commercially.

Selfboring pressuremeter research at UBC is performed in cooperation with Dr. John M.O. Hughes and Situ Technology Inc., and the current objective is to provide quality in-situ soil measurements for correlation, comparison and interpretation of similar results from other in-situ tests under development such as the flat plate dilatometer, screw plate, CPT, SPT and the like. The selfboring pressuremeter is, in essence, used as the 'primary reference' test and plays a very important role in all in-situ research at UBC.

BACK

Recent research has been directed toward the development of a cone-pressuremeter.

Screw Plate Test -- This test is merely a small plate load test but carried out at various depths in the ground. One of the reasons it has not been used more is that it is normally installed by hand and thus its effective depth is quite limited especially in sand. Research at UBC has implemented and automated installation and testing procedure making use of the UBC Penetrometer Research Truck. The equipment, procedure, results and their interpretation have been described by Berzins and Campanella (1981) and a Master's Thesis should be completed on the screw plate within the next 4 months.

It has been found that a 500 cm^2 area, double helix screw plate can be installed to depths in excess of 20m through medium dense sands. The torque required reached 17,000 in-lbs and the double helix allowed symmetrical loading on the tip and easier advancement than a single helix. Axial loading through the 10 ton Research Truck can apply a plate bearing as high as 17 kgf/cm^2 or 17 bar (1 bar = 100 kPa). A variety of loading procedures can be applied depending on the soil type and data required. Constant rate of load or deformation can be applied and the load versus deformation plotted to obtain modulus and strength. The load can also be maintained constant or applied in increments to obtain consolidation data. Load tests are often carried out at 1 m intervals in depth. Also, the installation torque is continuously monitored and used as a log of soil types penetrated. Dahlberg (1975) gives an excellent review of the application of screw plate tests in sand.

BACK

Vane Shear -- The field vane is still the most common way to directly determine undrained shear strength of clays. The vane is particularly suited to soft sediments where the sensitivity of cone measurements is often lacking. The field vane undrained shear strength is often used as a reference for correlations with other in-situ test data.

Seismic CPT Downhole -- A new type of device is under development at UBC which combines a bearing-pore pressure cone with a set of miniature 28 Htz. seismometers built into the cone. The bearing and pore pressures are used to log the stratigraphy of the site during penetration and downhole seismic tests performed at appropriate depths in the soil profile when the cone is being removed. This allows the determination of shear modulus (G_{max}) for various soil types. This test is currently in the early stages of development, but results to date are encouraging.

FLAT PLATE DILATOMETER

Equipment and Procedures

The flat plate dilatometer used for the dilatometer testing (DMT) at UBC was developed in Italy by S. Marchetti. The dilatometer is a flat plate 14 mm thick, 95 mm wide by 220 mm in length. A flexible stainless steel membrane 60 mm in diameter is located on one face of the blade. Beneath the membrane is a measuring device which turns a buzzer off in the control box at the surface when the membrane starts to lift off the sensing disc and turns a buzzer on again after a deflection of 1 mm at the centre of the membrane. Readings are made every 20 cm in depth. The membrane is inflated using high pressure nitrogen gas supplied by a tube pre-threaded through the rods. As the membrane is inflated, the pressures required to

BACK

just lift the membrane off the sensing disc (reading A), and to cause 1 mm deflection at the centre of the membrane (reading B), are recorded. Readings are made from a pressure gauge in the control box and entered on a standard data form. Full details of the test procedure are given in the Dilatometer Users Manual (Marchetti and Crapps, 1981).

The dilatometer is pushed into the ground using the UBC in-situ testing vehicle at a rate of penetration of 2 cm/sec. Before and after each sounding the dilatometer is calibrated for membrane stiffness.

The dilatometer data (readings A and B) are corrected for offset in the measuring gauge and for membrane stiffness. Another small correction is required because of the configuration of the measuring system. A full discussion on corrections is given by Marchetti and Crapps, 1981. Simplified expressions for the corrected data are:

$$P_o = A + A$$

$$P_1 = B - B$$

A is the vacuum required to keep the membrane in contact with its seating, since after several readings the membrane acquires a permanent outward curvature. B is the air pressure required to cause a 1 mm deflection in free air.

Using the P_o and P_1 the following three index parameters were proposed by Marchetti:

$$I_d = \frac{(P_1 - P_o)}{P_o - u_o} = \text{Material Index}$$

$$K_d = \frac{P_o - u_o}{\sigma'_{vo}} = \text{Horizontal Stress Index}$$

$E_d = 34.6(P_1 - P_o)$ = Dilatometer Modulus.

where u_o is the assumed in-situ hydrostatic water pressure and σ'_{vo} is the in-situ vertical effective stress. The data is reduced using a computer program supplied with the instrument and adapted at U.B.C. Computer graphics facilities are used to generate the completed plots.

The dilatometer equipment is extremely simple to operate and maintain. The simplicity and low initial cost of the equipment is one of the main advantages of the flat plate dilatometer as an in-situ test method. However, the simplicity of the equipment does generate some difficulty with interpretation of the results. Details of these problems will be discussed in later sections.

FACTORS AFFECTING RESULTS FROM DMT

Before using any data from flat plate dilatometer testing it is important to realize and account for potential errors that the data may contain. During the use of the flat plate dilatometer at U.B.C. several significant aspects concerning data collection and interpretation have been observed. Some of these points are summarized in the next sections.

Non-Verticality

It is almost impossible to push an instrument into the ground without some non-verticality, especially for deep holes. This problem is particularly important if the instrument measures a lateral stress, such as the dilatometer. The initial lift-off pressure for the dilatometer (P_o) can be significantly influenced by non-verticality.

A simple slope sensor similar to those incorporate din to many cone

BACK

penetration devices could also be included into the flat plate dilatometer. However, it is not clear how the data could be adjusted to allow for non-verticality.

The problem can be reduced, somewhat, by paying careful attention to the initial verticality at ground surface and by restricting the maximum depth of penetration. Work by Van de Graaf and Jekel (1982) using the CPT has shown that negligible error in recorded depth can be assumed for a maximum penetration depth of 15 m, provided no obstructions exist. Experience at UBC would suggest that good verticality can be maintained in soft uniform deposits for penetration depths in excess of 15 m. However, for less uniform deposits the suggested maximum depth of 15 m by Van de Graaf and Jekel (1982) would appear reasonable.

The incorporation of additional sensors to the existing dilatometer would significantly complicate the equipment and thus detract from its main advantage, i.e. simplicity.

Pore Pressure Effects

The dilatometer records total stress measurements (P_o and P_1). This is an important aspect regarding the test procedure and data interpretation. If the dilatometer were submerged in water, the lift-off pressure P_o should equal the hydrostatic water pressure (U_o). Marchetti has attempted to take this into consideration by normalizing the horizontal stress index, K_d. However, these are still several problems with the existing approach. The data analysis assumes the existing static water pressure to be hydrostatic. However, the in-situ static water pressure is rarely hydrostatic. The assumption of hydrostatic water pressure (U_o) can

BACK

have some influence on the index parameters especially in soft deposits where P_0 and P_1 are small in relation to the assumed u_0. The existing procedure assumes the membrane inflation is performed "without delay" when pushing is stopped. The rate of pressure increase is set so that expansion occurs in 15 to 30 seconds. It is not always possible to maintain a constant rate of testing since the rate of expansion is generally carried out at a constant rate but P_0 and P_1 may vary considerably, thus the time to reach P_0 and P_1 will vary. Also, the time between stopping penetration and full inflation is not always constant.

Results from cone penetration testing with piezometer measurements have shown that penetration into soft, saturated, cohesive deposits can generate very large pore pressures. The radial distribution of these large excess pore pressures gives rise to substantial hydraulic gradients in the radial direction. Dissipation of the excess pore pressures commences immediately after stopping penetration. The value of the high pore pressures around the dilatometer when testing in soft, saturated cohesive deposits will have a significant influence on the measured total stress values of P_0 and P_1.

Research at UBC has shown that if the rate of testing in a saturated cohesive deposit is varied, the index parameters I_d, K_d and E_d will also vary. Dilatometer testing at the UBC research site (McDonald's Farm) in a uniform clayey silt deposit from a depth of 15 m to 33 m was carried out at a variety of rates. The rate of testing was progressively decreased to allow pore pressure dissipation. As the excess pore pressure decreased the measured valued P_0 and P_1 also decreased. The resulting decrease in P_0 and P_1 caused an increase in the index parameters I_d and E_d but a decrease in K_d. The decrease in K_d is a direct result of the decreasing pore pressures around the dilatometer membrane. The increase in I_d and

E_d indicates that the drop in P_o is greater than the drop in P_1, since I_d and E_d both depend on $(P_1 - P_o)$.

In many low permeability cohesive (clay) deposits, the generally accepted rate of testing will have little influence on the measured values. This has been confirmed by the remarkably consistent dilatometer test results obtained in the Norwegian clays (Lacasse and Lunne, 1982) and recent research at the Univ. of Florida. However, when testing in relatively high permeability deposits such as silt or silty fine sand where significant pore pressures can still be generated during penetration, the existing testing procedure may not produce such consistent results due to variations in pore pressure dissipation.

EXISTING INTERPRETATION METHODS

Marchetti performed DMT at about 10 well documented sites in Italy and developed empirical correlations based on these results. Correlations were developed between the three index parameters, I_d, K_d and E_d and soil type, soil unit weight, K_o, OCR, undrained shear strength, constrained modulus and friction angle. All of the soil parameters were obtained from laboratory test results. The majority of the sites consisted of clay deposits with only two sites involving sand. At both sand sites the sand was very loose with relative densities around 30 to 40%. Details of the sites and the empirical correlations are given by Marchetti (1980).

The interpretation of the DMT results centers around the three index parameters, I_d, K_d and E_d. The parameters, I_d and K_d require a knowledge of the in-situ water pressure (u_o) before penetration and the in-situ vertical effective stress (σ'_{vo}). The in-situ water pressure is assumed to be hydrostatic and the only data required is the depth of the

BACK

ground water level. The significance of this assumption was discussed in the previous section. The in-situ vertical effective stress (σ'_{vo}) is calculated using soil unit weights obtained from an empirical correlation using I_d and E_d and using the assumed hydrostatic water pressure. The index parameter K_d can be significantly influenced by the assumed values of u_o and σ'_{vo} since,

$$K_d = \frac{P_o - u_o}{\sigma'_{vo}}$$

especially in soft saturated cohesive soil deposits where P_o is small.

The purchase of the dilatometer equipment in North America includes a computer program that contains the empirical correlations for interpretation and data presentation. An example of DMT results analysed and displayed by the computer is shown in Fig. 1 and 2.

The correlations proposed by Marchetti (1980) were based on a limited amount of data. In his closure to his 1980 ASCE paper Marchetti suggested that "the data base for all the correlations discussed in the paper will expand with the expanding use of the dilatometer test". Unfortunately, the writers believe that the development of the computer program to analyse and interpret the DMT results has tended to restruct the user and discourage improvements or modifications to the existing correlations as more experience is gained with the test. However, this problem will likely be minimized in the future with the recent addition of Dilatometer Digests by GPE Inc. which includes program updates.

BACK

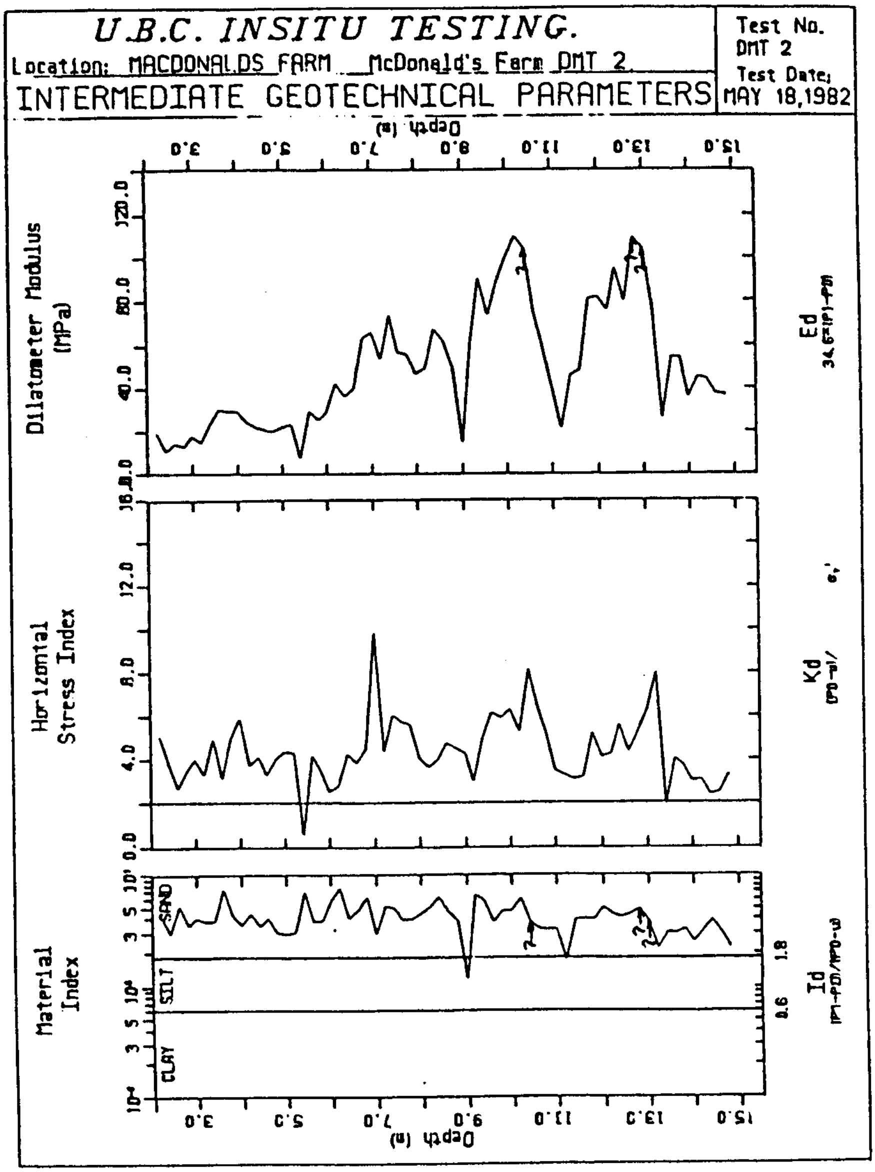

Fig. 1. Example of Dilatometer Data Presentation (McDonald's Farm Site).

U.B.C.INSITU TESTING RESEARCH GROUP.

File Name:dmt2 | Record of Dilatometer test No:DMT 2
Location:MACDONALDS FARM | Date:MAY 18,1982

Calibration Information:DA= 0.18 Bars   DB= 0.60 Bars   ZM= 0.10 Bars   ZW= 1.00 metres

Gamma=Bulk unit weight
Sv =Effective over.stress
Uo =Pore pressure
Id =Material index
Ed =Dilatometer modulus
Kd =Horizontal stress index

INTERPRETED GEOTECHNICAL PARAMETERS
Ko =Insitu earth press.coeff.
OCR=Overconsolidation Ratio
M =Constrained modulus
Cu =Undrained cohesion(cohesive)
PHI=Friction Angle(cohesionless)

Z (m)	PO (Bar)	P1 (Bar)	Ed (Bar)	Uo (Bar)	Id	Gamma (t/CM)	Sv (Bar)	Kd	OCR	Pc (Bar)	KO	Cu (Bar)	PHI (Deg)	M (Bar)	Soil Type	Description	Z (m)
0.60	0.61	1.50	31.	0.0	1.47	1.60	0.114	5.3	8.21	0.71	1.21		28.3	98.	SANDY SILT	COMPRESSIBLE	0.60
0.80	0.78	2.30	53.	0.0	1.96	1.70	0.148	5.3	10.54	1.56	1.20		29.6	100.	SILTY SAND	LOOSE	0.80
1.00	1.03	3.50	85.	0.0	2.39	1.70	0.182	5.7	12.81	2.33	1.27		31.1	169.	SILTY SAND	LOOSE	1.00
1.20	0.85	3.00	74.	0.02	2.60	1.70	0.196	4.2	7.29	1.43	1.03		30.7	129.	SILTY SAND	LOOSE	1.20
1.40	0.92	3.70	96.	0.04	3.17	1.70	0.210	4.2	7.14	1.50	1.02		32.0	168.	SILTY SAND	LOOSE	1.40
1.60	0.80	1.90	38.	0.06	1.49	1.60	0.222	3.3	2.88	0.64	0.85		27.4	55.	SANDY SILT	COMPRESSIBLE	1.60
1.80	0.78	2.30	53.	0.08	2.18	1.70	0.236	3.0	3.69	0.87	0.78		28.7	73.	SILTY SAND	LOOSE	1.80
2.00	1.03	3.60	89.	0.10	2.77	1.70	0.250	3.7	5.69	1.42	0.93		30.7	145.	SILTY SAND	LOOSE	2.00
2.20	1.48	7.10	194.	0.12	4.12	1.80	0.266	5.1	10.54	2.80	1.18		35.0	373.	SAND	LOW RIGIDITY	2.20
2.40	1.20	4.30	107.	0.14	2.92	1.80	0.282	3.8	5.86	1.65	0.94		31.0	176.	SILTY SAND	LOW RIGIDITY	2.40
2.60	0.95	5.10	144.	0.16	5.23	1.70	0.296	2.7	3.05	0.90	0.71		34.4	195.	SAND	LOOSE	2.60
2.80	1.27	5.00	129.	0.18	3.41	1.80	0.312	3.5	5.10	1.59	0.89		31.9	205.	SAND	LOW RIGIDITY	2.80
3.00	1.50	6.70	180.	0.20	3.89	1.80	0.328	4.0	6.48	2.13	0.98		33.7	305.	SAND	LOW RIGIDITY	3.00
3.20	1.35	5.60	147.	0.22	3.77	1.80	0.344	3.3	4.49	1.55	0.84		32.4	225.	SAND	LOW RIGIDITY	3.20
3.40	2.03	8.80	234.	0.24	3.79	1.80	0.360	5.0	9.93	2:58	1.16		34.1	443.	SAND	LOW RIGIDITY	3.40
3.60	1.43	10.20	303.	0.26	7.48	1.80	0.376	3.1	4.09	1.54	0.81		39.1	451.	SAND	LOW RIGIDITY	3.60
3.80	2.24	10.60	296.	0.28	4.38	1.90	0.394	5.0	9.89	3.94	1.16		35.4	561.	SAND	MEDIUM RIGIDITY	3.80
4.00	2.74	11.30	296.	0.30	3.50	1.90	0.412	5.9	13.94	5.74	1.31		34.2	606.	SAND	MEDIUM RIGIDITY	4.00
4.20	1.91	9.00	245.	0.32	4.45	1.80	0.428	3.7	5.72	2.45	0.93		34.4	402.	SAND	LOW RIGIDITY	4.20
4.40	2.15	8.40	216.	0.34	3.45	1.80	0.444	4.1	6.83	3.03	1.00		32.8	372.	SAND	LOW RIGIDITY	4.40
4.60	1.86	7.90	209.	0.36	4.02	1.80	0.460	3.3	4.46	2.05	0.84		32.9	319.	SAND	LOW RIGIDITY	4.60
4.80	2.28	8.00	198.	0.38	3.02	1.80	0.476	4.0	6.53	3.11	0.98		31.5	337.	SILTY SAND	LOW RIGIDITY	4.80
5.00	2.55	8.80	216.	0.40	2.90	1.90	0.494	4.4	7.74	3.82	1.05		31.9	383.	SILTY SAND	MEDIUM RIGIDITY	5.00
5.20	2.63	9.40	234.	0.42	3.07	1.90	0.512	4.3	7.58	3.88	1.04		31.9	415.	SILTY SAND	MEDIUM RIGIDITY	5.20
5.40	0.75	2.90	74.	0.44	7.00	1.70	0.526	0.6	0.17	0.09	0.04		30.7	63.	SAND	LOOSE	5.40
5.60	2.74	11.30	296.	0.46	3.75	1.90	0.544	4.2	7.20	3.92	1.02		33.4	817.	SAND	MEDIUM RIGIDITY	5.60
5.80	2.40	9.80	256.	0.48	3.86	1.90	0.562	3.4	4.85	2.73	0.87		32.7	401.	SAND	MEDIUM RIGIDITY	5.80
6.00	1.95	10.30	289.	0.50	5.75	1.80	0.578	2.5	2.70	1.56	0.67		35.0	376.	SAND	LOW RIGIDITY	6.00
6.20	2.16	14.50	427.	0.52	7.51	1.90	0.596	2.8	3.23	1.92	0.73		38.8	589.	SAND	MEDIUM RIGIDITY	6.20
6.40	3.15	13.70	365.	0.54	4.05	1.90	0.614	4.2	7.37	4.52	1.03		34.1	641.	SAND	MEDIUM RIGIDITY	6.40
6.60	3.00	14.60	401.	0.56	4.76	1.90	0.632	3.9	6.13	3.87	0.96		35.3	671.	SAND	MEDIUM RIGIDITY	6.60
6.80	3.48	21.80	634.	0.58	6.32	1.90	0.650	4.5	8.08	5.25	1.07		39.8	1140.	SAND	MEDIUM RIGIDITY	6.80
Z (m)	PO (Bar)	P1 (Bar)	Ed (Bar)	Uo (Bar)	Id	Gamma (t/CM)	Sv (Bar)	Kd	OCR	Pc (Bar)	KO	Cu (Bar)	PHI (Deg)	M (Bar)	Soil Type	Description	Z (m)

Fig. 2. Example of Dilatometer Data Tabulation
(McDonald's Farm Site).

BACK

Theoretical Considerations

General -- The flat plate dilatometer is a penetration test that includes a lateral expansion after penetration. The test (DMT) therefore combines many of the features contained in the cone penetration test (CPT) and the pressuremeter test (PMT). It seems reasonable that many of the observations and theories developed for the CPT and PMT relate to the interpretation and understanding of the DMT results.

Experience with CPT results has shown that very large stresses are generated during cone penetration. Although the dilatometer is wedge shaped and only 14 mm thick it can be expected that large changes in stresses will also occur around the blade during penetration. It can therefore be assumed that the penetration process will have a significant influence on the measured values P_o and P_1 during dilatometer membrane expansion.

The expansion of the dilatometer membrane is similar to the expansion of a pressuremeter. Thus, many of the observations made from pressuremeter testing may apply to the expansion phase of dilatometer testing.

The membrane on the dilatometer is located in the center of one side of the flat plate, a short distance behind the sharpened tip. Observations and cavity expansion theories have shown that there is some total stress relief behind the tip of any penetration tool. This is because the total stresses required to open the cavity at the tip are larger than those required to maintain the cavity. In the case of the electric cone, the theories of spherical cavity expansion relate approximately to the tip and cylindrical cavity expansion to the cone shaft. It seems reasonable that a similar process exists for penetration of the flat plate dilatometer. However, the level of stresses and strains developed around the dilatometer

BACK

may be smaller than those around a cone, when penetrating the same material, because of the thinner (14 mm) wedge shape (Davidson, 1983). The element of soil that is in contact with the dilatometer membrane, however, has undergone some stress relief (i.e. unloading).

Observations from pressuremeter resting have shown that the elastic modulus can be measured by performing an unload-reload cycle during a pressure expansion test. The membrane expansion of a dilatometer appears to be a reloading and therefore may be associated with an elastic modulus. However, the expansion of 1 mm at the center of the membrane may exceed the previous unloading and further shearing may take place, resulting in a modulus softer than the elastic modulus.

The writers believe that the penetration process and the phenomena of stress relief can be expected to have a significant influence on the measured values P_o and P_1 and the difference (P_1-P_o).

<u>Sand</u> -- Observations made during CPT and SBPMT at UBC would indicate that DMT penetration and membrane inflation in clean sands usually takes place under drained conditions. Experience has shown that the values of P_o, P_1 and (P_1-P_o) are usually relatively large in sands, especially dense sands. Thus, errors in assumed values of u_o and σ'_{vo} have a less significant influence on the index parameters than in soft clays where the values of P_o, P_1 and (P_1-P_o) are usually small.

The membrane of the dilatometer is located in a similar position relative to the tip as the friction sleeve on the cone tip. Thus, the changes in in-situ stresses adjacent to the dilatometer membrane due to penetration can be expected to vary in a similar manner to those around the friction sleeve of a cone. Research at UBC has shown that the changes in horizontal stresses due to cone penetration are related to the dilatancy of

BACK

the sand, as shown in Fig. 3. Thus, the measurement, $P_o - u_o$, can be expected to increase with increasing dilatancy of the sand deposit. Data indicates that the maximum dilation angle for a sand decreases linearly with the logarithm of increasing confining stress for a sand at constant relative density. However, for a limited stress range it can be assumed that the maximum dilation angle (ν_{max}) decreases linearly with increasing confining pressure. Thus, it can be expected that the horizontal stress index parameter, K_d, should be related to relative density for normally consolidated, uncemented sand. Recent chamber test results in sand using the DMT suggest this to be true (Marchetti, 1982). Results presented by Marchetti (1982) are shown on Fib. 4. Results from the two sand sites presented by Marchetti in his ASCE 1980 paper are also included in Fig. 4. The in-situ vertical effective stress (σ'_{vo}) for the data presented in Fig. 4 was in the range 0.5 to 2.2 kgf/cm^2, with a K_o of about 0.45. The in-situ relative density values of the sand deposits presented by Marchetti (1980) were estimated by the writers from CPT data. However, Marchetti (1980 & 1982) suggests that the sand at the two sites have relative densities around 60 to 70%. It seems unlikely, however, based on the calibration test results, that a sand with a relative density of 60 - 70%, and at a low confining pressure, would have K_d values of as low as 1.5. Experience gained at UBC appears to confirm this view.

Clay -- Observations made during CPT and PMT would indicate that DMT penetration and membrane expansion in clays takes place under undrained conditions. Observations and cavity expansion theories would also suggest that the penetration and membrane expansion generate very large excess pore pressures during DMT in soft, normally consolidated cohesive soils.

BACK

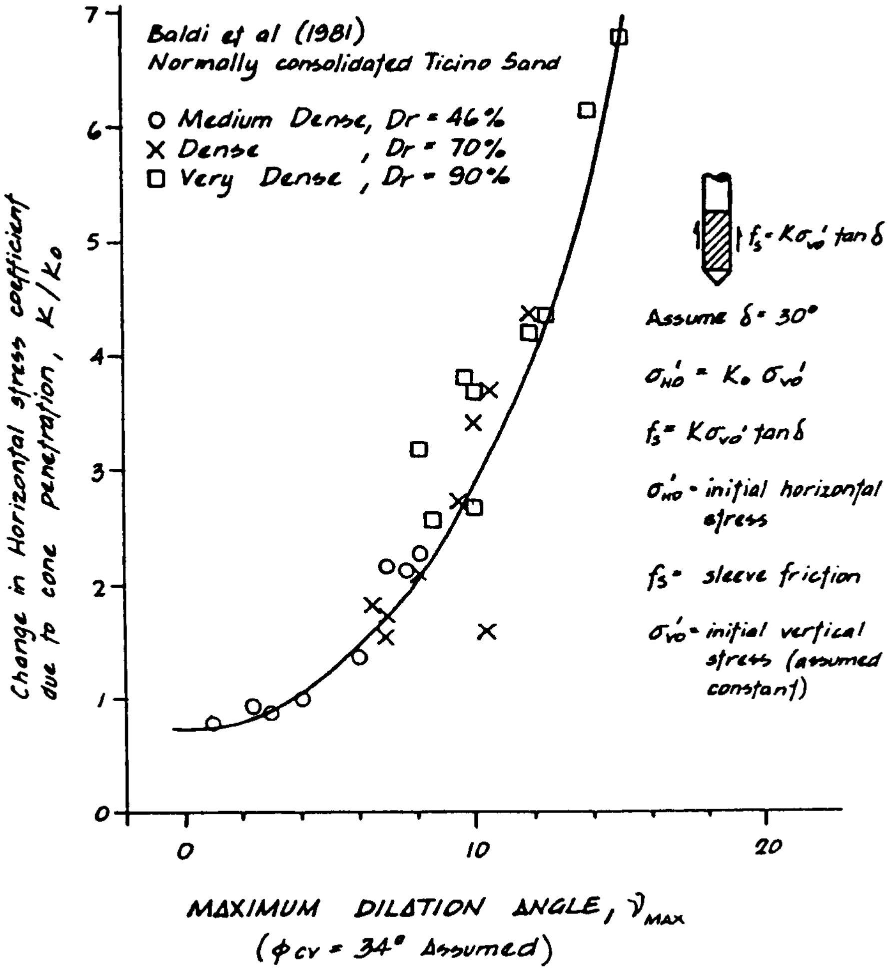

Fig. 3. Change in Horizontal Stress Coefficient Due to Cone Penetration in Sand.

BACK

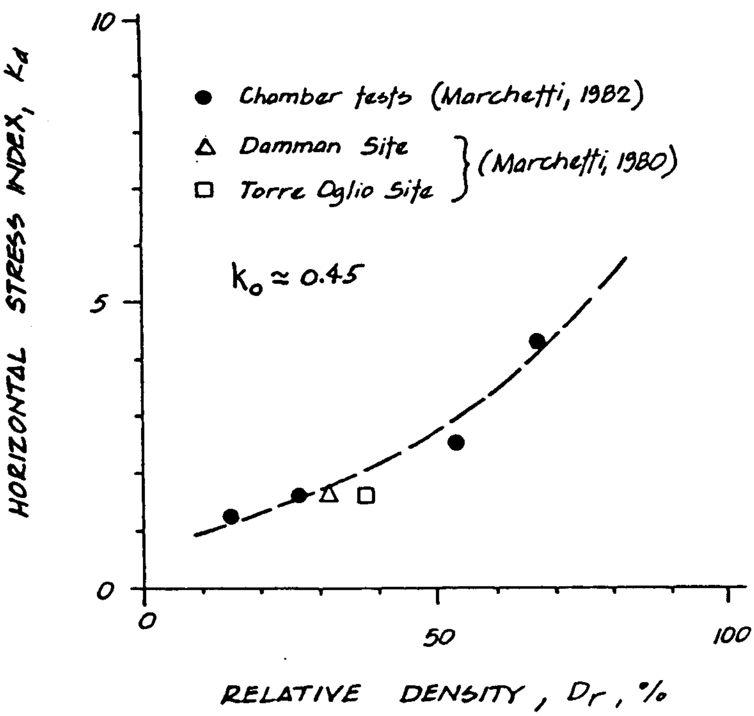

Fig. 4. Correlation Between Horizontal Stress Index from DMT and Relative Density for Normally Consolidated, Uncemented Sand. (K_o = 0.45).

BACK

The clay sites used by Marchetti (1980) to develop the empirical correlations were mostly composed of soft saturated deposits where large positive pore pressures could be expected during DMT.

Experience at UBC has shown that the values of P_o, P_1 and (P_1-P_o) are usually small in soft clay deposits. Thus, errors in assumed values of u_o and σ'_{vo} may have some influence on the derived indix parameters and subsequent inferred geotechnical parameters.

Cavity expansion theories have shown that a limit pressure exists for undrained cavity expansion in soft clays. It seems reasonable to assume that the penetration process in a DMT is sufficient to induce pressures equivalent to the limit pressure. Because of the stress relief phenomena, creep and pore pressure dissipation, the lift-off pressure P_o is less than the limit pressure. However, the expansion of 1 mm is probably sufficient to re-establish a limit pressure. Thus the value P_1 is probably related to the limit pressure for some form of cavity expansion.

The cavity expansion theories have shown that the limit pressures are related to the E/c_u ratio. Ladd et al. (1977) have shown that the E/c_u ratio varies approximately with plasticity index (PI). Thus, the index parameters I_d and E_d from DMT results in cohesive soils may relate to the PI of the soil. Since experience has shown that geotechnical parameters such as undrained shear strength and compressibility can be related in some manner to PI, it seems reasonable that the index parameters I_d and E_d can similarly be related to these geotechnical parameters. Marchetti (1980), Schmertmann (1980) and Lacasse and Lunne (1982) have reported good correlations in soft clay deposits using DMT results.

For DMT results in overconsolidated cohesive soils, the correlations

BACK

may not be so successful. This may be due to the pore pressure effects. During penetration in overconsolidated cohesive soils, small positive or possibly negative (below hydrostatic) excess water pressures may be generated. These smaller pore pressures influence the measured P_0 and P_1 values, since both values are total stress measurements. An example of this phenomenon will be presented in a later section and its implications on the interpretation discussed.

Experience Gained at UBC with the DMT

Table 2 shows some of the important parameters that can presently be interpreted from the DMT and summarizes the experience at UBC regarding the relative success of the interpretations. The following sections will describe some of these experiences.

McDonald's Farm Site -- A research site for in-situ testing is located on an abondoned farm (McDonald's Farm) at the Vancouver International Airport. The site is located on the north side of Sea Island on Ministry of Transport, Canada land near the Municipality of Richmond. Sea Island is located between the North Arm and Middle Arm of the Fraser River on the north side of the main Fraser River Delta. The site is approximately level with the natural ground at elevation +1.6 m. Sea Island is contained by a system of dykes to protect against flooding from the Fraser River.

A summary of the soil profile based on sampling, laboratory and cone penetration testing (CPT) is shown in Fig. 5. The upper 2 m of soil consists of soft, compressible clays and silts. The sand from 2m to 13m was deposited in a turbulent environment and is therefore relatively non-uniform in density. In general, however, the sand increases in density

BACK

Table 2

Summari of
Experience at UBC with DMT

Interpretation	Sand	Clay
Soil Type	Slightly toward too fine a grain size	
OCR	Questionable	Generally good
K_o	Questionable	Generally good
c_u	--	Generally good
ϕ	Low	--
M	Unsure	Variable
E	Quite good	Very low

Research Sites	
McDonald's Farm	Sand and Clay
New Westminster	Sand and Silt
Langley	Clay (O.C. & sensitive)

BACK

with depth as indicated by the constant relative density relationship by Baldi et al., 1982. The sand has a medium to coarse grain size with thin layers of medium to fine sand. A thin transition layer of fine sand with some silt exists from 13 m to 15 m.

The sand is underlain by a deep deposit of soft, normally consolidated clayey silt. The clayey silt is estimated to extend to a depth of more than 300 m. (Blunden, 1973)

Groundwater is approximately 1 m below existing ground surface and groundwater pressures are approximately hydrostatic for the depth shown in Fib. 5.

Fig. 6 shows the three intermediate parameters, I_d, K_d and E_d from the DMT. The material index, I_d, has identified the sand and clay layer very well. However, the classification is slightly on the fine side for both soil types.

The horizontal stress index, K_d, is very variable and quite high in the sand. It is the K_d that is used to estimate the in-situ K_o and OCR. The writers feel that K_d is not a good parameter for estimating K_o or OCR in sand. The chamber test work in Italy has shown that K_d is affected by both in-situ stress level and density. Thus, it is not possible to distinguish between the two influences of in-situ stress and density. Fig. 4 showed data that related K_d to relative density for one level of in-situ stress (K_o = 0.45). Thus, at present, it is not possible to estimate K_o from the DMT K_d data in sand.

It is interesting to note that the K_d value is constant at about 2.0 in the underlying clayey silt of McDonald's Farm. The DMT interpretation estimates the in-situ K_o in the silt to be about 0.5 with an OCR of about 1.0. All the in-situ testing has shown the silt to be normally consolidated.

BACK

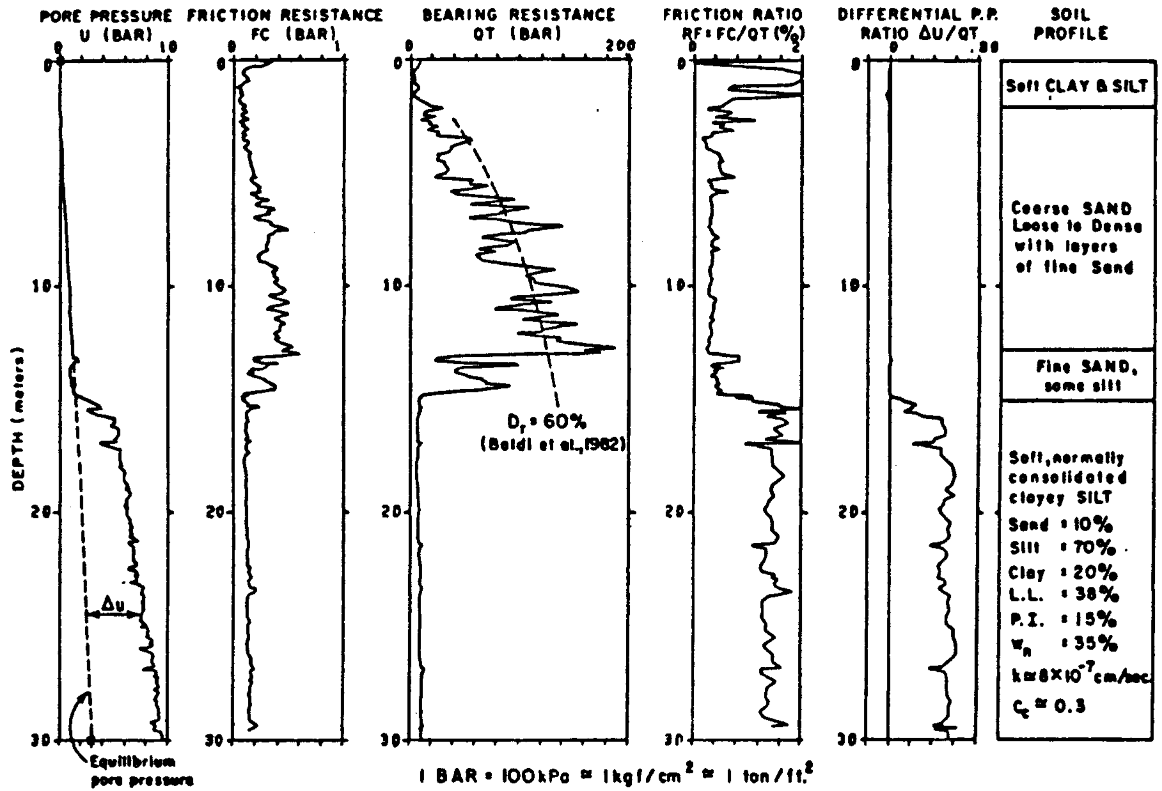

Fig. 5. Soil Profile for Research Site at McDonald's Farm. (After Campanella, Robertson and Gillespie, 1983).

The dilatometer modulus, E_d, is also shown on Fig. 6. The E_d is very variable in the sand with values very much higher than the underlying silt. Included on Fig. 6 are the results of laboratory triaxial test measurements of the initial tangent Young's Modulus on "undisturbed" samples of sand. It appears that the DMT E_d compared reasonably well with the measured laboratory values. However, it is important to remember the sensitivity of laboratory measured moduli values to disturbance as well as the anisotropic nature of most sand deposits in terms of moduli.

Fig. 7 shows the plot of interpreted geotechnical parameters from the DMT. The DMT has predicted an approximately linearly increasing undrained shear strength, c_u, in the silt. These values agree very well with the predicted values from the cone ($N_k = 15$) and the measured values from the self-boring pressuremeter tests. The c_u/σ'_{vo} ratio is about 0.2 from all the in-situ tests, which is reasonable for a normally consolidated low PI silt.

Also included on Fig. 7 is the DMT predicted friction angle, ϕ, values for the sand. The average ϕ angle predicted from the DMT, using Marchetti's interpretation, is about 34^o. The measured values from the laboratory testing and from the self-boring pressuremeter testing indicated an average ϕ value of about 41^o.

<u>Langley</u> -- Another research site for in-situ testing is located near Langley, B.C., which is approximately 40 km east of Vancouver. The site is underlain by a thick deposit of a sensitive glacial marine silty clay. The clay is overconsolidated near the surface due to dessication and load removal but is approximately normally consolidated below a depth of about 10 m. The clay in the upper 10 m has a sensitivity in excess of 10, whereas, the clay below a depth of 10 m has a much lower sensitivity.

BACK

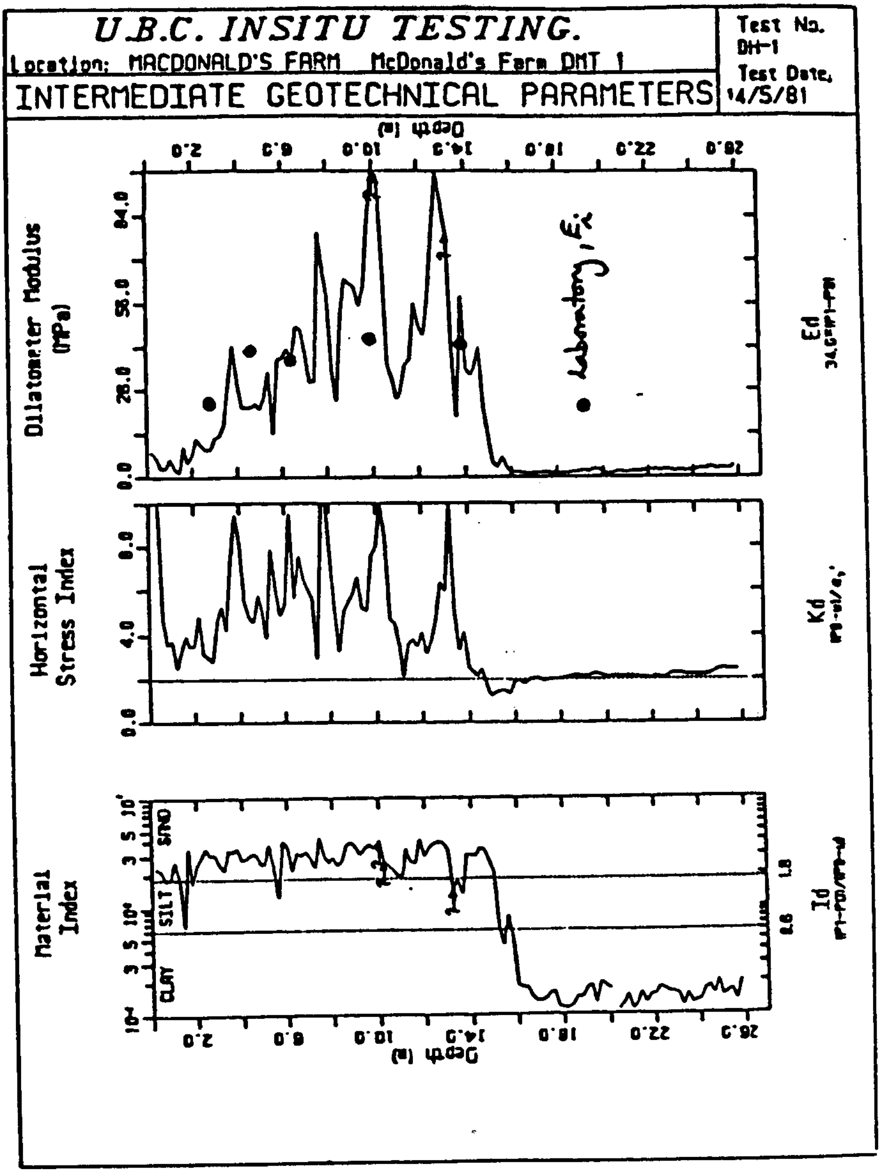

Fig. 6. Intermediate Parameters from DMT, McDonald's Farm.

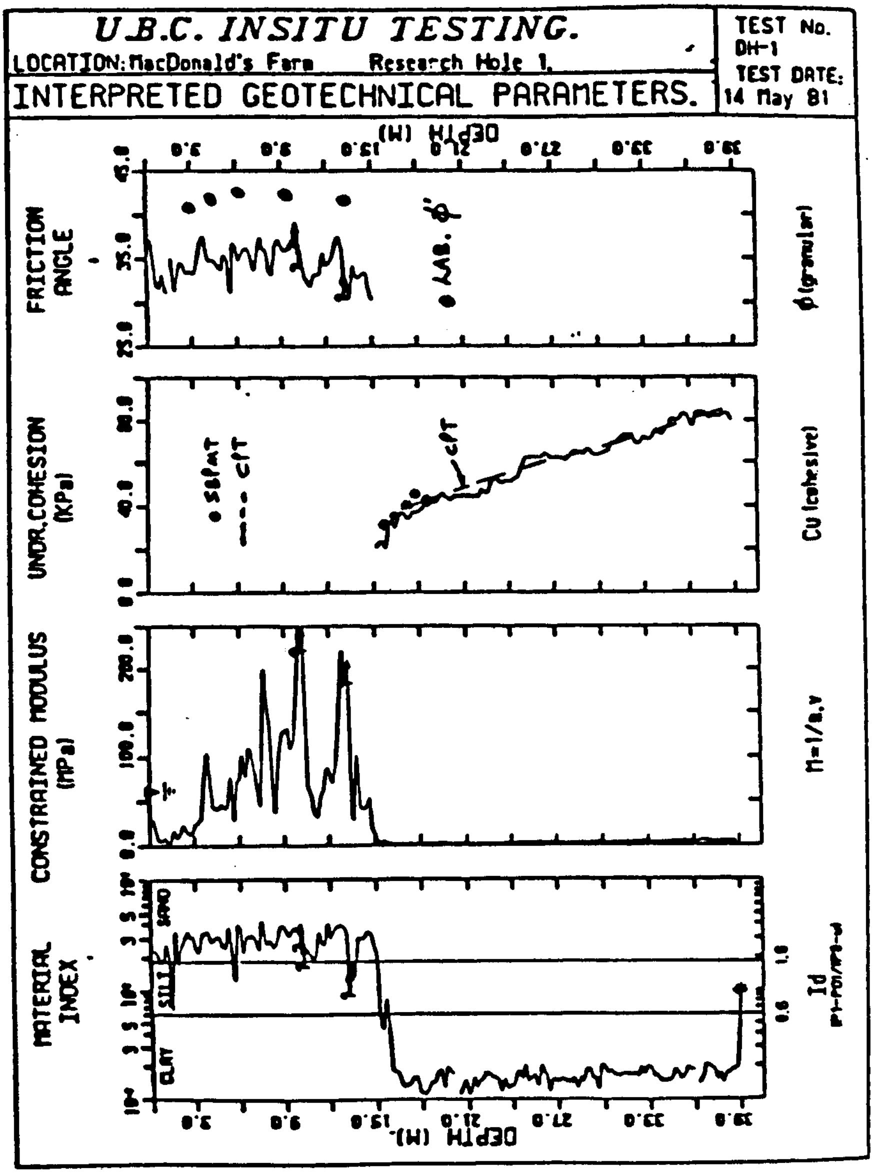

Fig. 7. Interpreted Geotechnical Parameters from DMT, McDonald's Farm.

BACK

Fig. 8 shows the interpreted geotechnical parameters from the DMT. The material index, I_d, has clearly identified the clay deposit, including a sand layer at a depth of about 9.5 m.

The undrained shear strength, c_u, from the DMT, however, is considerably larger than the measured field vane values. The c_u values from the cone penetration test and screw plate test data agree with the field vane values. The reason for this descrepancy is unclear but may stem from the predicted stress hostory (OCR) from the DMT. Fig. 9 shows the intermediate parameters K_d and E_d. The horizontal stress index, K_d, is high which leads to a high predicted OCR for the deposit. This high OCR in turn leads to a high predicted c_u value using the DMT corelations.

The dilatometer modulus, E_d, is considerably smaller than the Young's modulus measured from screw plate tests. In general, the screw plate Young's Modulus at 25% of the failure stress is 5 times larger than the dilatometer modulus. In general, the DMT did not predict very realistic geotechnical parameters for the Langley site. This may be related to the high sensitivity of the clay deposit.

<u>New Westminster</u> - Another research site for in-situ testing is located on a former dock area in New Westminster, B.C. The site is located on the north bank of the main channel of the Fraser River just at the entrance to the North Arm. The entire site was gradually reclaimed between the early 1900's and 1945 for dock facilities. The reclamation was carried out in several stages. The river adjacent to the dock was dredged and the sand and silt spoil was used to fill the site behind timber bulkheads.

A summary of the soil profile in the research area at the site, based on sampling, laboratory and cone penetration testing is shown in Fig. 10.

BACK

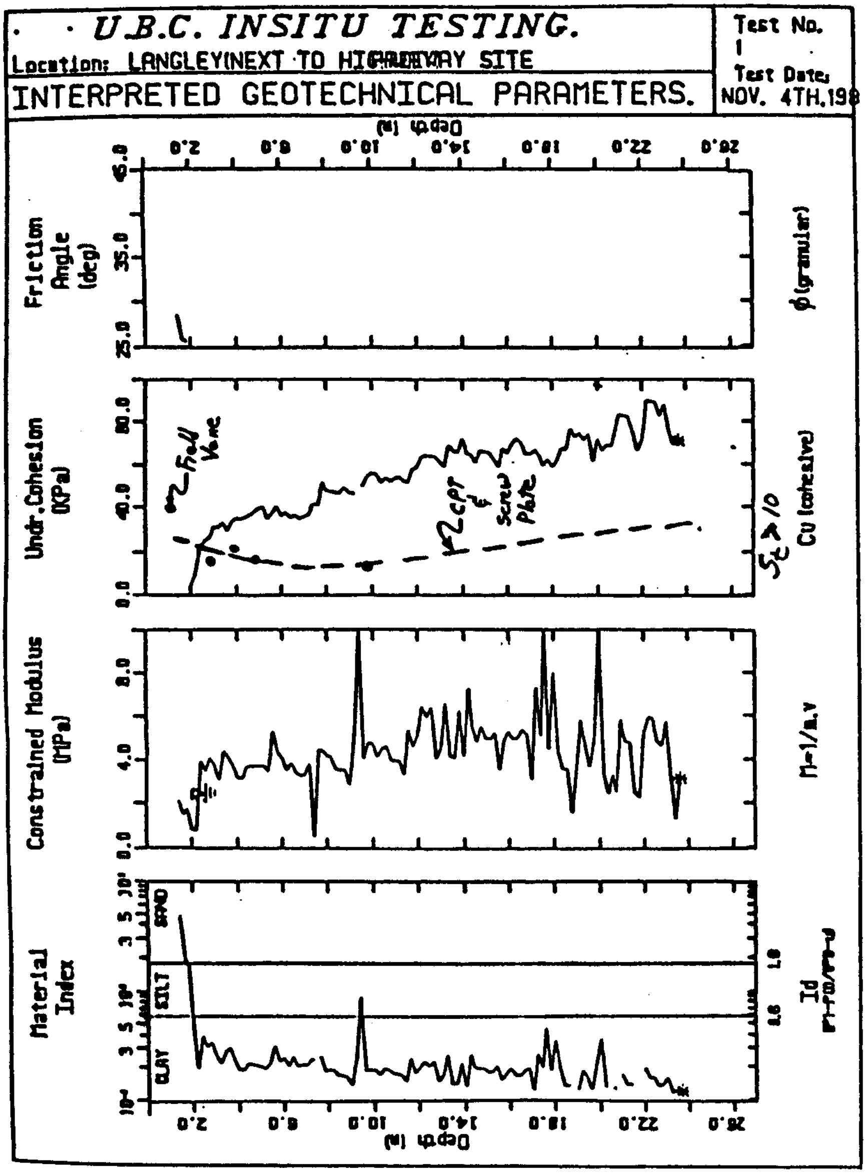

Fig. 8. Interpreted Geotechnical Parameters from DMT, Langley Site.

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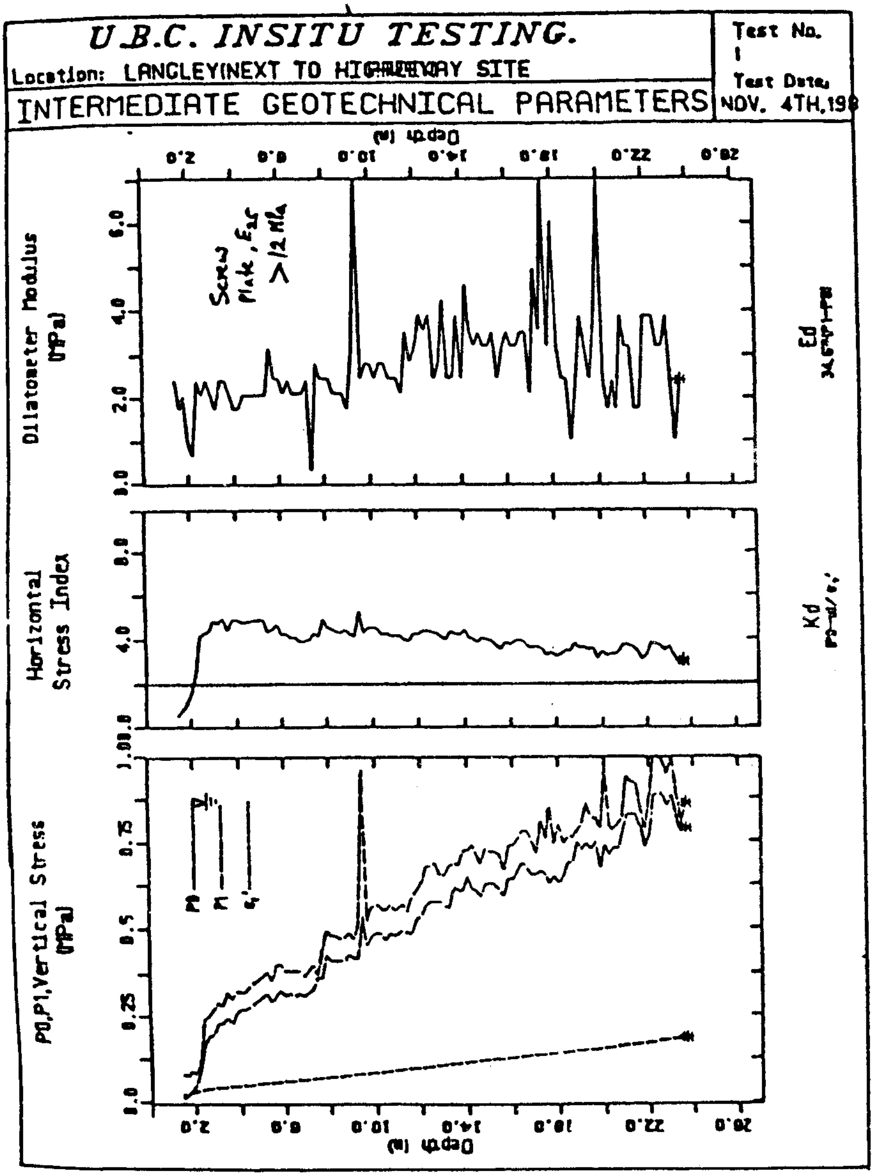

Fig. 9. Intermediate Geotechnical Parameters from DMT, Langley Site.

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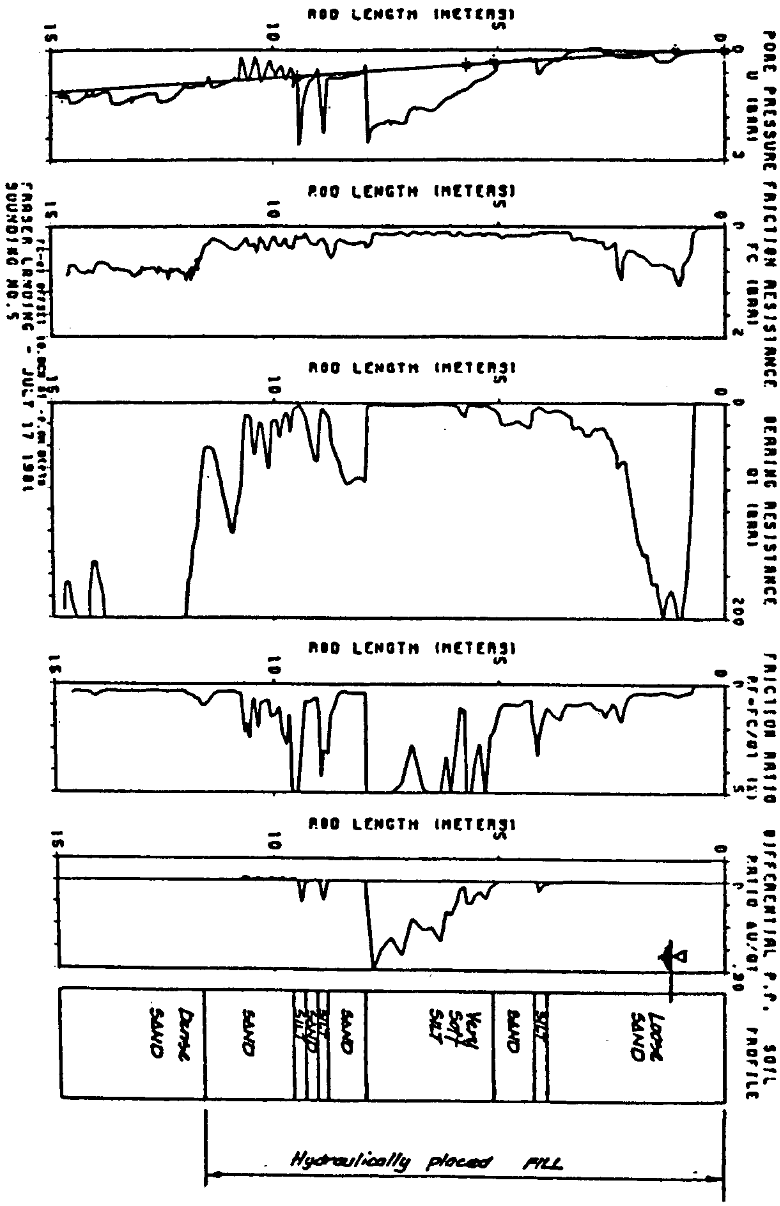


Fig. 10. Typical Soil Profile at New Westminster Site.

BACK

The upper 12 m consists of a loose hydraulically placed sand and silty soil. The sandy soil exists to a depth of about 5 m and a low plastic silt layer lies between 5 and 8 m depth. The silt layer varies in thickness and elevation across the area due to depositional history. The silt has a plasticity index (PI) of about 8% and a liquidity index of about 1.0. The site is approximately level at elevation +3.65 m. Groundwater fluctuates with tidal movements and varies from about 1 m to 4 m below ground surface.

The site is the proposed location of the Fraser Landing complex, a major condominium structure. The geotechnical consultants, Macleod Geotechnical Ltd., were concerned about the stability of the site under earthquake loading. Two soil stabilization methods were studies by the consultants, Vibro-compaction and Dynamic Compaction. A test program was undertaken by the consultants to evaluate the effectiveness of each method. Access to the site was made available to the authors before and after stabilization treatment. A site plan showing the treatment areas and test locations is shown in Fig. 11.

Fig. 12 shows a summary of the piezometer cone data from the test section before and after treatment by dynamic compaction. It is interesting to note that before treatment, the silt layer generated very high excess pore pressures during penetration as indicated by the large differential pore pressure ratio values. However, after treatment, there was a remarkable change in the pore pressure behaviour with significant pore presures less than hydrostatic. Similar pore pressure behaviour was observed during cone penetration for the test section where treatment was by vibro-compaction. The very different pore pressure behaviour suggests that volume change characteristics of the silty layer have been altered

BACK

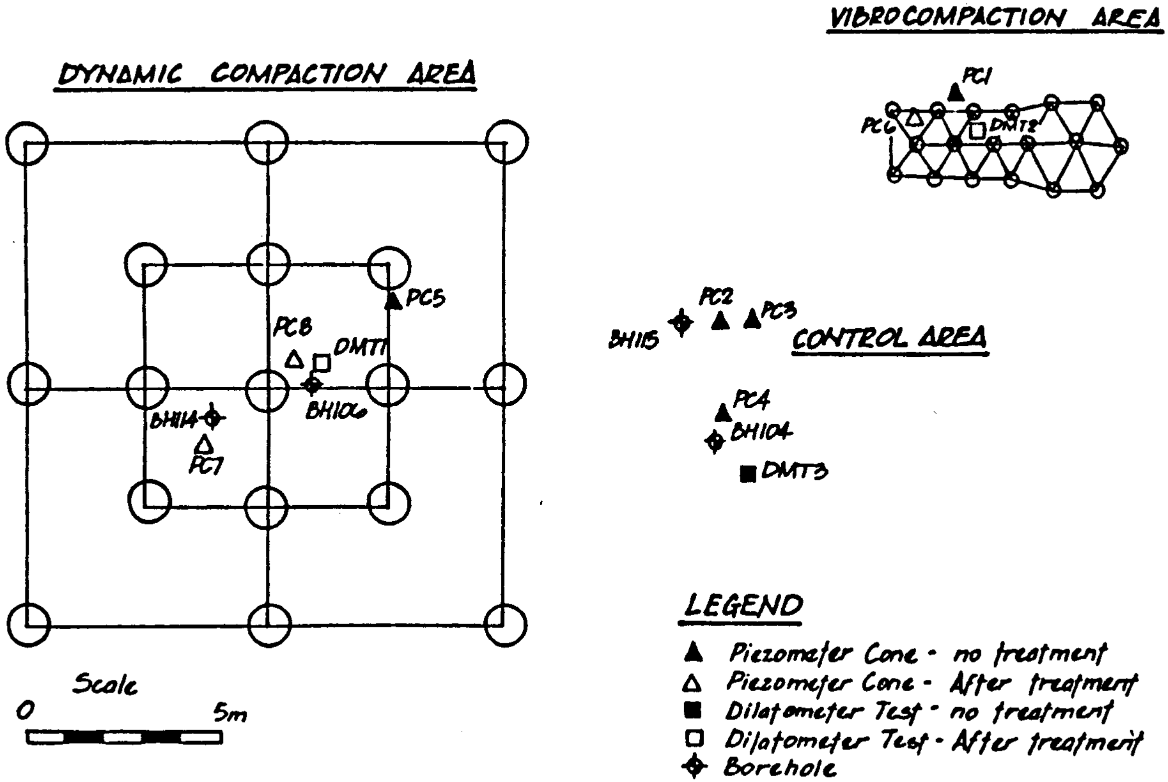
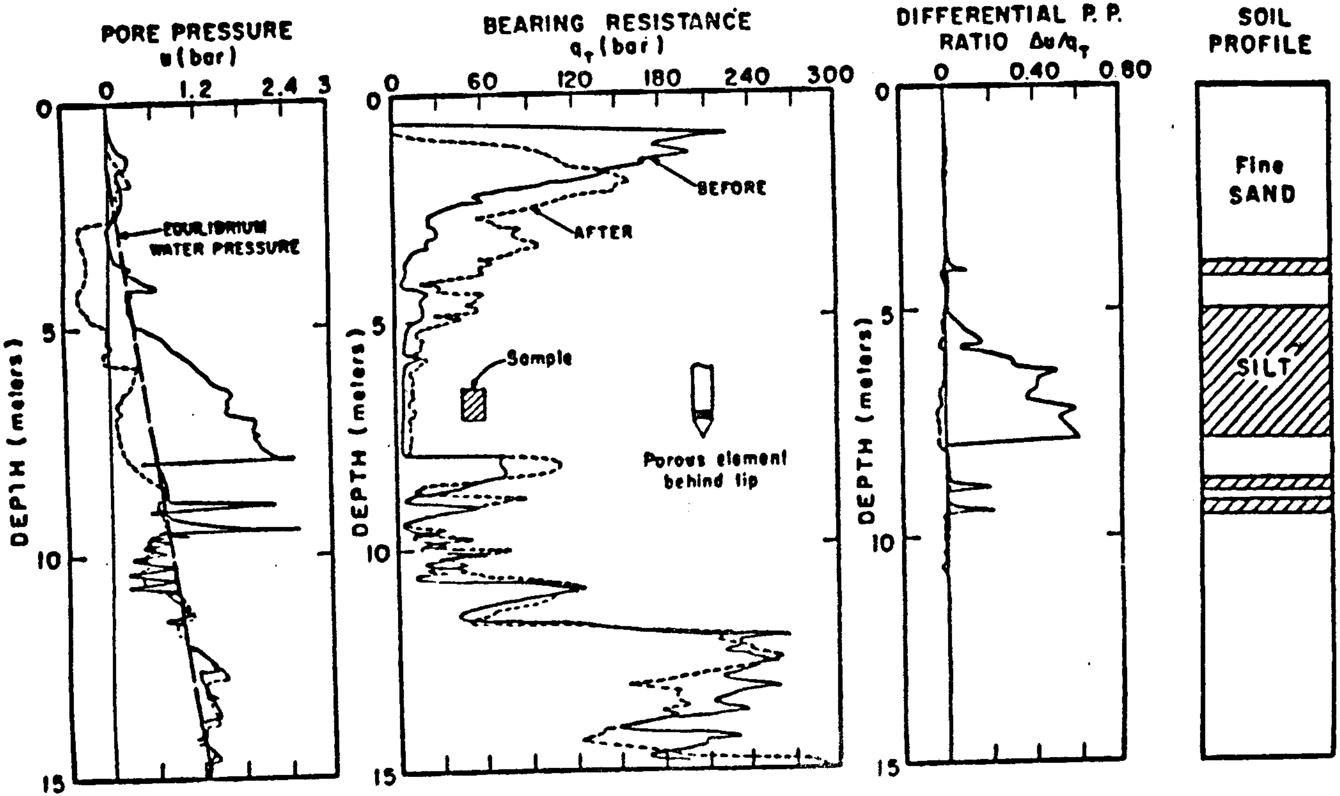


Fig. 11. Site Plan, New Westminster Site.

BACK

Fig. 12. Piezometer Cone Logging Before and After Dynamic Compaction, New Westminster.



PIEZOMETER CONE LOGGING BEFORE AND AFTER
DYNAMIC COMPACTION (NEW WESTMINSTER, B.C.)

BACK

dramatically. From the standpoint of liquefaction resistance, volume change characteristics are very important.

A summary of the field and laboratory results for the silt layer before and after compaction is shown in Table 3. The laboratory testing clearly showed that the silt was soft and contractive before treatment but dilative after treatment.

Figure 13 shows a summary of the DMT results from the control area where no treatment was carried out. The material index, I_d, has clearly identified the silt layer from 7 to 9.5 m and the horizontal index, K_d, is constant at about 1.8. Figure 14 shows a summary of the DMT results after dynamic compaction. The silt layer exists from a depth of about 5 to 7 m and can be identified from the basic DMT data (P_0 and P_1). However, the I_d is barely able to identify the silt. The K_d within the silt has now dropped to about 1.2. In the overlying sand the K_d has increased due to the increase in density and horizontal stresses. However, in the silt the K_d has decreased. This decrease is more marked in the DMT results after vibrocompaction (Figure 15). In the vibrocompaction area the silt exists from about 7 to 8.5 m and can again be identified from the basic DMT data (P_0 and P_1). However, the silt layer is not identified from the material index, I_d. The K_d has now decreased in the silt to about 0.6. However, in the overlying sand the K_d has increased significantly due to the large increase in horizontal stresses caused by the vibrocompaction treatment.

Marchetti has suggested in his recent 1982 paper that K_d can be used to estimate liquefaction resistance. Table 3 summarizes the measured K_d values in the silt and the predicted cyclic stress ratio to cause liquefaction using Marchetti's proposed correlation. Also included in

BACK

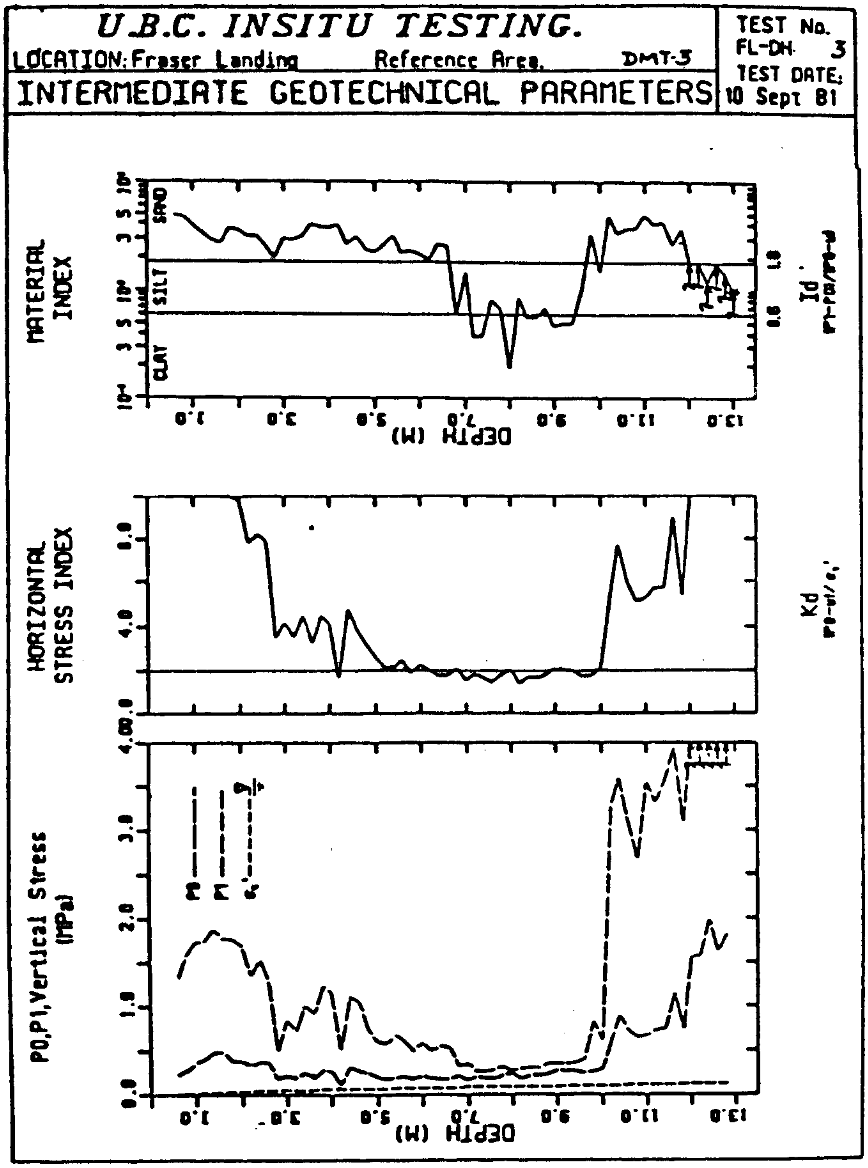


Fig. 13. Intermediate Geotechnical Parameters from DMT in Control Area, New Westminster Site.

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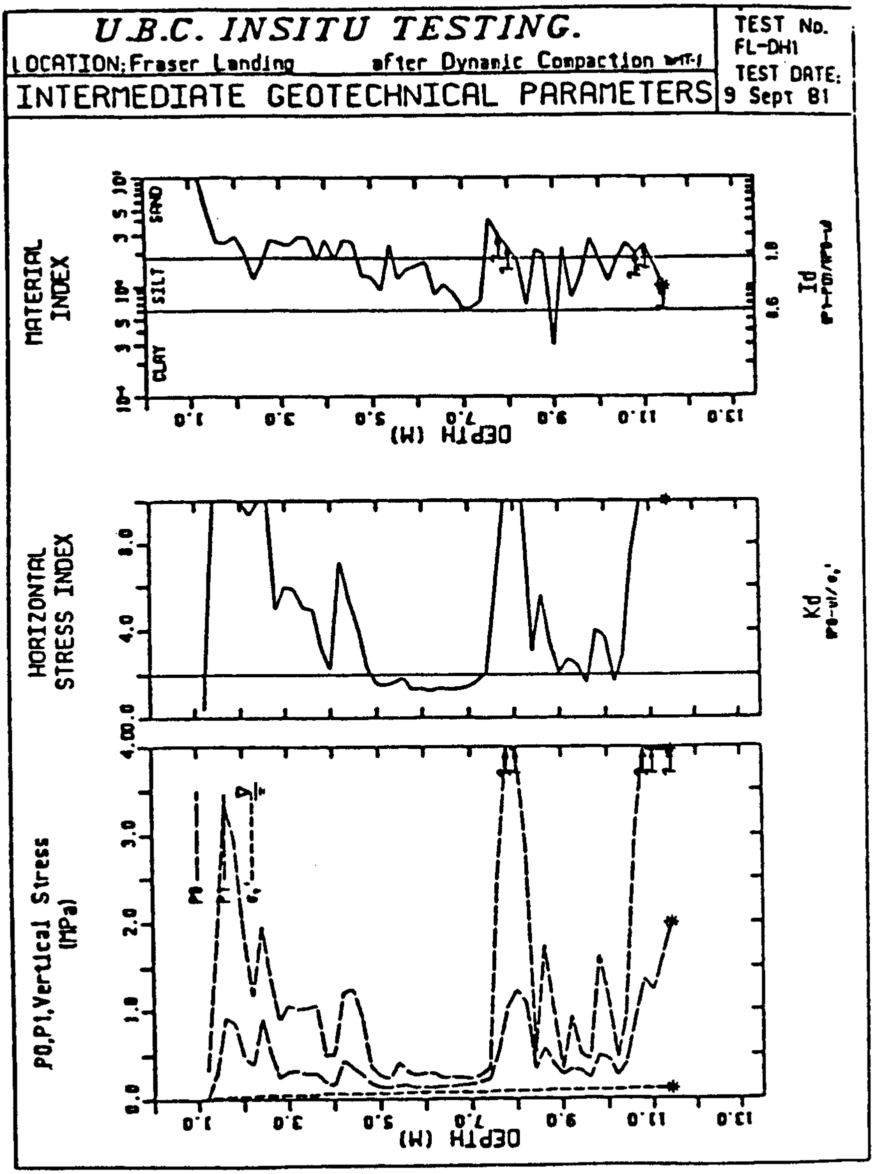

Fig. 14. Intermediate Geotechnical Parameters from DMT after Dynamic Compaction, New Westminster Site.

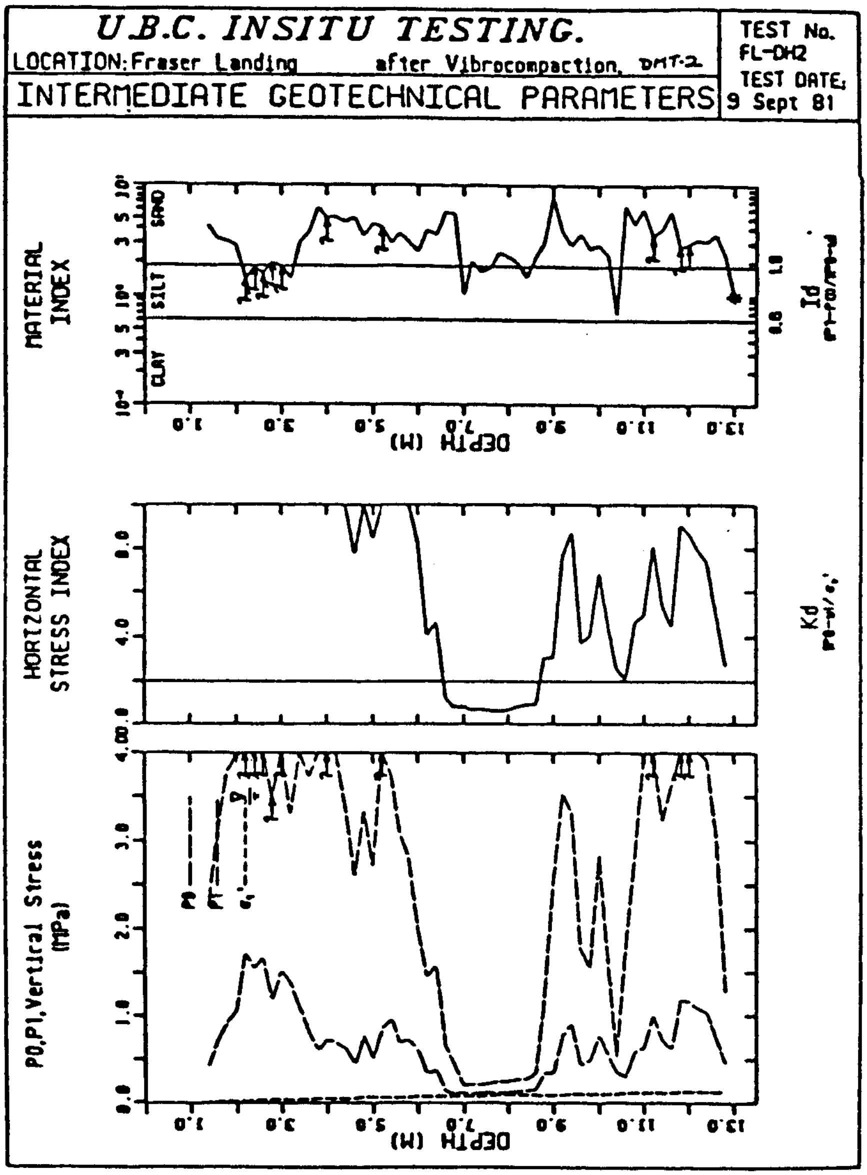

Fig. 15. Intermediate Geotechnical Parameters from DMT after Vibrocompaction, New Westminster Site.

BACK

Table 3

Comparison of Silt Parameters Before and After Compaction, New Westminster Site

	Control Area (No Compaction)	Dynamic Compaction Area (After Compaction)	Vibroreplacement (After Compaction)
SPT N-value, blows/ft.	5	7	-
CPT cone bearing, q_T, bar	4.5	10	-
Undrained shear strength, c_u	14 kPa	82 kPa	-
Cyclic stress ratio τ/σ' to cause liquefaction in 10 cycles	0.10	0.20	-
Dilatometer			
Horizontal Stress Index, K_D	1.8	1.2	0.6
Marchetti's cyclic stress ratio	0.18	0.12	0.06

$\tau/\sigma' = K_D/10$

BACK

Table 3 are the results of some cyclic laboratory tests that clearly show that the silt showed a marked increase in liquefaction resistance after treatment. Thus, for some reason, the DMT results predict completely the wrong behaviour of the silt after treatment.

The reason for this response probably results from the pore pressure behaviour of the silt during penetration. Before treatment, very large pore pressures exist around the dilatometer during testing. However, after treatment, the piezometer cone data indicates that very small pore pressures exist around the dilatometer during testing. Since the dilatometer records total stresses, it is very sensitive to the pore pressures around the instrument during the test.

Research Dilatometer

Much of the Dilatometer research at UBC is currently comparing predicted parameters with those measured by other in-situ tests and laboratory tests at various sites in the lower mainland of British Columbia. We are also predicting and monitoring field performance at three preload sites, at an axial and lateral pile load tests site, and at field compaction sites by dynamic compaction and vibroreplacement. Prototype performance evaluation is the best way to develop correlations.

The previous examples of our experience with the DMT and UBC indicates the usefulness of this simple device. However, the interpretation is empirical and needs further evaluation based on a better understanding of the measurements and the factors affecting them. For this purpose, a research dilatometer has been under development at UBC for the past year.

The research dilatometer at UBC is identical in size, shape and operation as the Marchetti design except for the passive measurement of several paramters which include:

1. pore water pressure at the center of the moving diaphram,
2. deflection at the center of the diaphram,
3. total pressure activating the diaphram,
4. verticality of the dilatometer during penetration, and
5 the penetration force for the dilatometer.

Measurement of pore water pressure during penetration and during inflation of the diaphram will help us to understand rate effects, compression characteristics of the soil and apply an effective stress interpretation to observed behaviour. Simultaneous measurement of pressure and deflection of the diaphram will give a better assessment of the stress-strain character of the soil in comparison to the standard two point approach of measuring P_0 and P_1 for lift-off and 1 mm deflection. Any measured deflection of the blade from vertical will indicate either a stress increase or stress relief on the diaphram, therefore, affecting the measurements. Finally, a direct measure of pushing force will allow a direct calculation of ϕ as suggested by Schertmann, 1982. The research dilatometer should be operational very soon now as it appears that most of the severe development problems have now been overcome.

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BACK

PANEL

BACK

PANEL QUESTION PERIOD

Q. My question is, why do you use a gas, I think its nitrogen, to inflate the diaphrahm. It would seem to me that you'd have problems if you are drilling very deep and you have got your gas tank up on the surface, and a couple of thousand feet below you've got your diaphrahm. Why, for instance not use a hyraulic system?

A. I will just answer that gas doesn't require a compensation for hydrostatic pressure, so its a simplication to use gas, and that's about it in a nutshell.

Q. Well, I'm really being a devil's advocate but I would say what can't we do with the SPT whcih we can do with the Dilatometer? If we look at all the correlations that we've got, if we take an SPT we can get friction angle, we can get the shear strength, we can get some idea of its modulus. Now when you are doing an SPT all your doing is taking a series of numbers which are got from the ground, and correlating them with past exeperience. Isn't the Dilatometer exactly the same? We've got a tool which gives a series of numbers, than can be got in a very consistent way which is the problem of the SPT. But we have got a series of numbers and all we're saying is we've got a correlation between those numbers and the shear strength, the friction angle and stiffness.

A. Several things: First of all, and I know he's being a deveil's advocate, the correlation with friction angle is not an empirical correlation but a theoretical calculation. So they differ in that respect very dramatically. And secondly there is a very important philosophy to the Dilatometer that this gives me a chance to emphasize once again, and that is if you shorten the extrapolation distance between that you do in test that produces less soil disturbance and correlate to the

BACK

undisturbed condition compared to what you do in a more-disturbance test and correlate, you have a much better chance for getting a more accurate extrapolation to the insitu condition. I think the disturbance distance, if you will, for the standard penetration test is very much greater than the disturbance distance for the Dilatometer. Therefore the potential for an accurate correlation is much greater for a tool like the Dilatometer. And one further item: With a Dilatometer test you make two measurements. With the SPT you make one measurement. There is a lot more potential in interpreting two measurements than one by a factor of three. I guess I can say one other thing, as well. That is the Dilatometer test attempts to directly measure a pressure and a modulus and then it extrapolates from these direct measures to pressures and moduli. With something like the standard penetration test you are measuring a blow count to something that happens in the ground in the way of a modulus. So there is some difference in kind, and I think there i a great difference in degree.

Q. Dr. Schmertmann, I would like to follow on from John Hughes here. You mentioned a factor of 3 improvement on SPT. I think it is a factor of 2. If you look at the three formuli you gave Kd, Ed, Id, and you do a spot of algebra you find Id is simply Ed/Kd projected by a constant, approximately. You only have two numbers, not three, and those follow through from mathematics generally, if you measure two independent things you only learn two fundamental properties. Care to take it?

A. That's true in algebra but not in statistics. In statistics you also have the possibility of correlating on the interraction of the two. That is your third possibility.

BACK

Q. Basically the rebuttal would be relations, it depends on your assumption between relations for the two variables. If I wish to combine two numbers there are a simple infinity of combinations I can dream up, and basically the first thing I learn depends on my funcational relationship between the two variables. Now I may have other data which I am introducing in that assumption and that is why the combination between those two numbers works, but it doesn't actually tell me any more about the soil, and the follow on from this is for example I think I counted seven correlations on the last paper, to two measurements, what they are really telling me is that of those seven correlations five of the material paramaters are intrinsically inter related. If you go to say the simplest model soil which consistently works which is kind of very, the (inaudible) modified critical state model, we have basically a five parameter model. You would never on an absolute basis with a Dilatometer hope to predict the fundamental behaviour of clay, for example. It is not physically possible from the number of measurements you are making. Now that is not to say the gadget is not useful, but I think we have to be cautious not to overstate how useful it might be.

A. I agree completely. Could I just say something, as a practising engineer. For routine examinations of soil conditions the Dilatometer test is much superior to some of the so-called standard procedures that we have been using. Where I come from, there is not much available in the way of static cone testing equipment. Certainly, compared with the standard techniques that are available, the Dilatometer (from my experience) is giving us much more data and therefore more confidence in what we're doing. It really reduces the area where judgement has to come into play and as you know, if you are trying to design anything from standard penetration tests there is a lot of judgement require. I really don't look on the dilatometer as an instrument that is giving us absolute information.

BACK

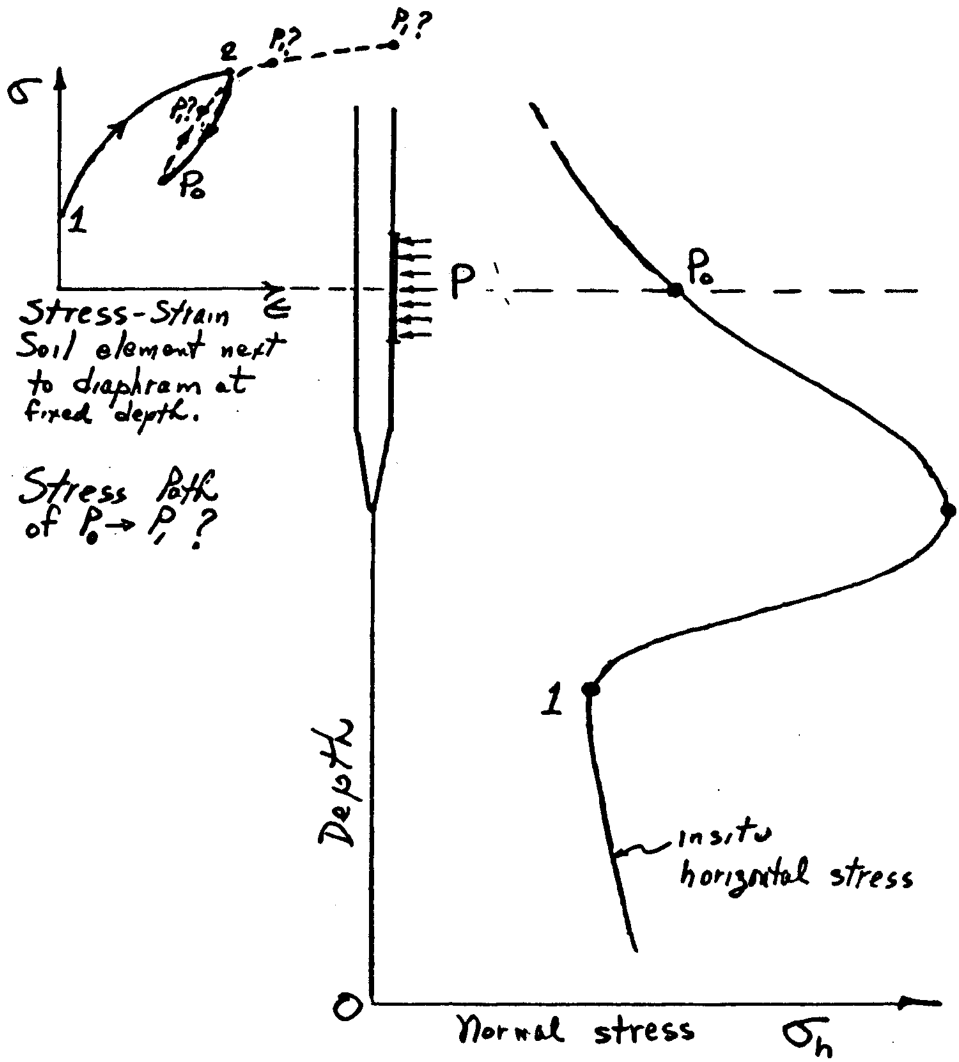
I just think it's giving us a good, accurante sensitive feel for the soil conditions.

Q. This is (inaudible) from Pipelines, Calgary. Any of these measurements where we measure so called insitu properties in fact we are disturbing the soil by inserting this instrument. Based on your experience would you say the Dilatometer has the least impact on the soil properties, so we are disturbing the soil least with the Dilatometer than with say the cone penetrometer and other. Your comments please.

A. (Dr. Schmertmann) Well, I think there is one exception that we must immediately mention, and that is the self-boring pressure meter, because the whole objective of the self-boring pressure meter is to get it into the soil with very very little disturbance. The degree of success is another matter, but that is the objective. The Dilatometer goes in with far less volume strain and shear strain disturbance than the cone penetration test does and this has been the subject of study at the University of Floria. In dry sands the level of volume and shear strain is considerably less. Furthermore, it is much more uniform around the Dilatometer membrane that it is around the penetrating cone. The area around the penetrating cone is an obvious area of very non uniform stress conditions, while the conditions that occur immediately opposite the Dilatometer membrane are relatively quite uniform. The DMT blade is intended to disturb the soil as little as possible, and I think it seems to me to be a pretty good compromise between that objective and the fact that you have to have an instrument to be used in the field. Marchetti's objective was as short an extrapolation as feasible to the insitu state.

A. Dr. Campanella: I wonder how clear it is to everyone when we talk in terms of insertion stresses. This is just a crude sketch (See sketch over)

BACK



Crude sketch for Camparella's Answer pg. 99

BACK

What I am trying to do is demonstrate in a very simple manner the stress state that we think exists around the tip of the blade, and its very similar to the concept that you can apply to the cone inserting into the soil as well. When the blade is pushed into the ground the total stress condition is increased and there is a greater total stress needed to enter the ground and open the cavity (Point 2) than there is to keep the cavity open as it passes along. That means that if you were to look at this point in the ground right here, when this was up at this point this high stress condition existed here, and so you might think of the stress increasing like this, it reaches a point and then as the blade passes that point there is this stress relief and so this point travels back in that direction. I think this is an important thing to understand when the diaphragm is then loaded this point is assumed to go up like that and be on a sort of elastic type behaviour, a reload curve. One of the reasons why we wanted to actually instrument the diaphrahm was to measure what happens between the points P_o and P_1. What happens if in fact the point doesn't do that but actually winds up out here for the 1 mm of movement. You only have two points, P_o and P_1 in the DMT and of course you are assuming a straight line between them so you actually are thinking of a path like that. Now if it does this it would be just perfect, and that's what we would like to try and verify. Our initial result seems to indicate that in sands it does stay on a stright reload path, but we think, and again it is just our thought, that in clay soil its already at the limit here when its unloaded and then when the diaphragm is expanded it very quickly comes back to the limit and goes on straining at constant stress. So we'd

BACK

like to instrument the diaphragm and actually see what the stress-strain path of the diaphragm is. I am not sure that everyone is clear that in fact the stress state that exists on that diaphragm when you stop to perform the test is one of stress release and that there has been an unloading of stress on the side of the blade and that's an important point.

A. Peter Robertson, I would like to add something to that. Dr. Schmertmann has mentioned the "short Extrapolation" routes, and I think that what Dick has just said is a better way of explaining that simplistic approach of short extrapolation.

I think the other point I would like to make which relates to all three of the previous questions is related to the meaning of empirical correlation.

All of the original Marchetti correlations are empirical. Marchetti measured the two parameters Po and P_1 and calculated the three index parameters. He then plotted graphs of index parameters against some other parameter like undrained shear strength, or over-consolidation ratio, and by going through that process developed correlations which are empirical. It's important to remember that process in terms of the meaning of empirical correlations when one talks about disturbance. After all, for years we have had correlations with the Standard Penetration Test, and even before that correlations to things like plasticity index (PI) of a clay, which of course involves total disturbance. But for years we have managed to get quite good correlations of certain parameters from the PI or the liquid limit, or all the atterber limits of clay, and get quite good empirical estimates of certain soil parameters. Some of Marchetti's parameters are, maybe, on the same level as some of those empirical correlations. Some may be a lot better, and I think that experience and maybe some research will open some of the doors to our understanding of those correlations and on what basis they have from a theoretical point of view, if any. My own feeling is that some do have some sort of theoretical basis. One could go through a logical theoretical discussion and logically say this index parameter should relate to a certain sort of parameter in some manner. Yet, I think there is still some way to go. Chamber test research and other types of research combined with just practical experience may open some of those doors.

Q. The question I want to ask is addressed to John. Really what is the strain along here, at the time the probe is pushed into there, because I think it is in the other of several thousand per cent, isn't it? If I look at the behaviour of the (inaudible) on that boundary. When you have pushed it into the soil you have observed the strain that has gone on, and so in what sort of order is it?

A. Well, if you want the boundary layer exactly at the boundary we don't know. But a short distance away like half the blade thickness the volume strain is anywhere from two to ten per cent depending on whether you started with a loose sand or a dense sand. At the moment I don't know because I haven't seen the results. Professor Davidson told me that they were about the same magnitude as the volumetric strain. Just shooting from the hip my own impression is that there is something like an initial 5% strain, and then an unloading and then a restrain. But, just to put this into perspective: you don't have to know all the details of this behaviour in order to use it. A good example is aspirin. We have been using aspirin for a long time and it works very, very well, and I don't think the medical people still know what aspirin does.

Q. Chris, Gold Associates I am a little confused now. I thought I had reasonable understanding until Dr. Roberston was explaining things. From what you said I understand that the basic correlation are between the three properties, or the three indicis (what you have developed from A & B) and behavioural properties, normal engineering behavioural properties. Is that the case? Because

BACK

my impression up to this point was that what you were doing was for example for undrained strength you were measuring a lateral stress and assuming that its an insitu stress, and poor water pressure of course comes in there then you know what the over consolidation ratio is and you know what the pre-consolidation ratio is and therefore you can also use a empirical correlation to get to an undrained strength, but is that the form of correlation, or is it a directly correlation between those three indices properties or those three indices and measured behavioral properties. Which is it?

A. Its not that simple to answer in a few sentences. If you buy the DMT equipment you get a manual with it, and the manual has quite a bit of detail about what the program does, but its quite a thick manual and not all that many people read it in detail. The correlations are quite complex and some involve two of the material index parameters. Marchetti plotted lots of graphs of index parameters against soil parameter and some gave quite good relationships and some gave very bad ones. It was the good ones he chose. I think it turned out that he got quite good relationships in clays for OCR, so he estimated OCR for a clay and then used laboratory results that correlated OCR to CU over P' ratio and then he estimated the vertical effective stress and then estimated CU. So we are left with a long string of correlations. Does that explain it? But what is actually my concern is the technique of processing the data through a computer program. A lot of people that use the data and interpretation don't know what is happening between the two stages. They collect the data and get the interpretation but they don't know what's happened in between. In some respects this is good because you don't have to think about it and this massive amount of data is being manipulated for you, but in other ways, I feel personally, it can be a little worrying because the majority of people that use the program don't know what has happened and therefore just blindly

BACK

take the results. And as we have seen today people have presented data and said "well it worked good here, it didn't work good there". And they have no way of knowing why because they didn't know how the data got from one stage to the other. Maybe if they were more aware of that it might be easier.

Q. Mike Jefferies, Gulf Canada I would like to throw two observations into the can here, which you guys can't resolve now but its something you might like to think about when you are doing your research. No 1. Beaufort Sea. We have been finding geostatic stress fails, horizontally something like twice to three times what we would predict from the measured OCR based on audometer tests, this will obviously impact your local correlations. No. 2. while doing this work we are using our pressure meter, John Hughes' gadget, and we monitored pressure both at the surface and down the hole in the pressure meter. We were running these tests very rapidly and we were in 30 meters of water, maybe 15 meters into the dirt. What we found when you run the test very fast is that you can easily have a 20 psi differential in pressure between what you measure at the surface and what you see with a proper pressure down the hole. You might like to see, when you have got your well instrumented device, how the standard unit in influenced by the length of the lead you have on it, because obviously this is a function of pressure where it travelled down thin bore plastic pipe.

A. There have been a number of comments about the pressure at the top versus the pressure at the bottom. There is a quick field check that you can make to see whether you are pressure loading too fast. - - you simply close the valve. Its a flow control valve that you are using and it control the rate at which you let gas into the system to pressurize it. If you just close the valve you can see if the needle drops when it reaches equilibrium. If it drops a lot you are loading too fast because it took a drop in pressure at the top to reach

BACK

equilibrium. If there's a big difference in pressure between the top and the bottom, and the equilibrium was half way in between, that creates the drop that you can see. If you close and see very little movement in the guage needle then number one, you know you don't have a leak, and number two, you know that you haven't been loading it too fast. This is something that worries everyone that first uses the DMT. As a practical matter though the pressure stress wave in a gas tube travels very quickly and if you're only working with 20, 30 or 40 meters of cable its not all that much of a problem in the normal testing times that are used. Typically you get your A and B readings in a 15 to 30 second interval. The technique is to increase the gas pressure rapidly till you come close to where you think you are going to get the reading, and then you increase it very slowly as you approach where you expect the reading. That works 80% of the time. Perhaps 20% of the time you get fooled because the reading is not exactly where you expected.

Q. Erik Funegard of Geosystems. I would like to ask one question which I think is based mostly on my experience in pressure meter testing previously. Somebody said that the measurements are independent of the operator, now you are saying that you have to check the speed, or the rate of your testing. Now wouldn't the rate definitely influence your results and wouldn't it, if you are allowed to vary the speed, be dependent on the operator?

A. Well, I think the term 'operator independent' has been used in the sense that if you set this thing up in the field and take ten guys and one after another have them run the test you get very similar results. As long as the guy can hear there's no "technique" required to perform the test, except that the recommended rate of applying the pressure is that you achieve the pressures in a 30-second time interval, and that's really not that hard to do. Marchetti has done some tests to determine whether there is much difference in the readings at various rates, and I think he has simply, as a standardized procedure,

BACK

said, get each reading within an interval of 15 seconds. You really don't have much problem doing that in the field with different operators.

What Dr. Schmertmann was talking about in terms of checking the two pressures was that you may want to check the rate of pressure applicant, especially with the B reading. Maybe if I back up a little bit to describe the procedure.

At the start of the test, the buzzer is on. You then pressure up to get an A reading when the buzzer goes off. You apply pressure for another 15 to 30 seconds and the buzzer comes on again and that's your second (or B) reading. If you're going too fast, or if there is some concern that you're applying the gas pressure too fast, or that there is a difference between the top and the bottom pressures, rather than exhaust the gas at the B reading (which is what you normally do), you can simply shut it (the gas) off and then see what happens to your pressure at that point. That would give you some feel for whether or not it's being pressured up too quickly.

The other comment I would like to make is that there are a lot of things that happen, I guess with any kind of testing. One thing I have noticed with the Dilatometer is that it does seem to tell you when there is something funny going on. For instance you always make the assumption that you have a uniform pore water pressure distribution. We had a site where we were testing through sands then into clay and then through another sand layer. At one part of this site, this lower sand layer was actually slightly artesian. We were a little bit higher up the hill on one of the test holes, never thinking while we were doing the test that there was anything different from that assumed pore pressure distribution. Yet when we came to calculate the results we got some very funny negative readings because the ID is calculated from P_0 (the A reading) minus the pore pressure. When you start getting negative readings like that nothing works very well in the calculations, and what we found out was that in fact there was a golf course adjacent to this area that we were testing and they were actually pumping water from this lower aquifer so that in the area that we were testing, the ground water had disappeared due to the pumping and we actually broke through the clay into a relatively dry sand with a lowered water table. Of course, the usual assumption about the pore water pressure was wrong, but the point I am trying to make is that the results told us that it was subsequently confirmed, of course, with the piezometer installations. There have been a number of instances like that where the results we got did not seem to be just right I have gained some confidence in the test to not fool you.

A. The key to the rate effects is that is will be soil type dependent. What we find is that in a clean sand that's fairly well drained you do not develop any excess pure pressures during the test so there are no rate effects due to pore pressures, other than possibly the air wave going down the line. In a very low permeability clay there is also little rate effect. Dr. Schmertmann has been involved in some tests by one of his students who confirmed that for sands there was no problem because the pore pressures were dissipating or not even generated. In clays there is a low enough permeability that during that 30 second time period the pore pressures are not dissipating very much. But it's in that intermediate soil that Dr. Campanella was highlighting, such as a silt, that might have a relatively large permeability buy yet still low enough that it will develop pore pressures when you push in the instrument. You could get operator inconsistencies there due to pore pressure effects. I should also like to add that, Dr. Schmertmann mentioned a simple test of getting to the B valve and shutting the valve, and if there is no stress wave it should stay constant. Well that may not happen in a clay because if a clay has developed large pore pressure and you maintain the 1 mm displacement and hold that constant, the high pore pressures outside the diaphragm immediately start to dissipate and so your pressure will drop off.

BACK

Well ok let's say that some dissipation does occur and the membrane expands a little bit while you are sitting there looking at the needle, the total volume of gas that's in the system far exceeds the additional volume that might be created by this pretty small movement of the membrane, and I don't think that this is really a valid comment. Perhaps I misunderstood it, but in any event it is a very practical test just to shut the valve and see what happens to the needle. Once you use the test a little you'll see that for yourself. I don't imagine everybody in the room has operated the Dilatometer and we do have it over there, and you can stick the Dilatometer in that barrel full of clay and there's water in it. There's a water table, and there's sand in it and you can play with the thing and hear it beap, hear it stop beaping and hear it beap again, if you wish, if your a hands-on-type. If you are not a hands on type when this thing's is over we'll turn on the video of the Skyway bridge in Tampa and you can hear the beap and see the instrument, so whichever you like there. Dr. Campanella did you have a comment at all?

Q. Mike Jefferies, Gulf Canada Resources again. I would like to aim one at Neil here. What was your cycle time for the Beaufort test between starting one test with the Dilatometer, completing the test, pushing or driving the instrument down into say the next test interval which from what I remember of your graph is typically a meter below it. In minutes, were you looking at a fifteen cycle time or thereabouts, perhaps?

A. Well as long as we could drive it may have been one or two blows, to go a foot or two. There was a very small amount of time involved in between tests. Where the tests were widely spaced of course we were drilling and maybe doing other things between tests.

Q. Now I was just thinking in comparison with the self-boring pressure meter where you just keep pushing the unit ahead, I take it you mean about a five minute cycle time.

BACK

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involve the self-boring pressure meter.

\- If I may come back to you on that I would suggest the cone because that does it faster and better, and there are as many correlations with the cone as there are with the Dilatometer. (Question from the audience - inaudible) I would agree but you then get into problems of equipment availability, cost and those tend to be very area dependent, but if I had the choice I'd go with an electric piezometer cone, as a logging tool, based on my experience as of now.

A. Well I think I can say I've had a fair amount of experience with the Dutch cone test, and now I've had a fair amount of experience with the Dilatometer, and I've also had a fair amount of experience with the pressuremeter test. Now, when I have a choice at a job, the first tool I go out with is usually the Dilatometer because I find it gives me the most useful information the quickest. This is assuming I have some information about the soil conditions. Of course if I don't have any information about the soil conditions I will have an SPT boring made first, because that's the cheapest thing to do. Then, if I'm looking for more specific data as Peter said, I will take a look at the other tools and the piezometer cone is one of them, because it tells you something about pore pressures. That comes at a price, of course. The self-boring pressure meter tells us something very special, more accurate we hope, about certain other parameters. If its worth it we would use that, or we would do sampling and laboratory testing. But we have found the Dilatometer, or at least I find it, a very efficient tool to get quickly some good information from the site. This often turns out to be enough information. That's all we need. One disadvantage of the cone test is that it doesn't tell us anything about lateral stresses, and the Dilatometer does, and I consider that a tremendous advantage. I also think that the Dilatometer gives us better information about drained compressibility than the correlations with the cone. But the question of lateral stresses is extremely important and that's something not to be overlooked, it should not be played down, it should be

BACK

A. Perhaps.

A. I don't know what the cycle time was on the ship but in a land operation, where you are pushing it, the machine sitting there ready to push, and the minute you've finished your B reading and the operator hears the exhaust he begins to push, and usually you're at the next testing level not in a matter of minutes but in a matter of seconds, and even when you're driving usually you are sitting there with a hammer ready to operate and I'd be very surprised, unless its really hard driving, that you are talking about more than a minute to get down to the next test level, but in an off shore operation maybe it is different.

Q. I would like to come back a bit here. The reason I threw it out is for comparison with the self-boring pressure meter. Actual things tend to go a bit slower because you are running twenty-four hours a day, and you have various technicians and coffee breaks and things. For comparison we have got a cycle time figure of about twelve minutes, once. We have understood how to use the self-boring pressure meter (inaudible) about two and a half times more valuable. Personally I think it is, but that is the question.

A. Personally I think it isn't and the test time in similar sites that I wasn't on site for where the test intervals are very close and the clay was weak we were probably pushing it, and like Jack says you're looking at one or two minutes per test interval. The self-boring pressure meter is a relatively quick test too but with the bore-hole data you can get what the pressure meter is. The number of tests you can do in a given period probably exceeds anything else they can do down the hole. Peter Robertson - I would like to make a comment regarding comparison of tests. Dr. Campanella tried to make the point of the difference between the logging test and a specific test, and we view the Dilatometer as alogging test, so I would use it as a quick logging tool where I can get quick approximate interpretations of soil parameters that may then identify critical areas that I might want to go back and do specific tests on, where that specific test may

BACK

played up. In sand, for example: Say you do a ground improvement job using vibroflotation and after thr probe incertions you find, at least we have found a number of times in the centre of the vibroflot pattern, no difference between the before-after CPT gc-logs. You wonder what in the world happened? After all you saw them putting in sand. You know that they have put in such and such a number of cubic yards of sand over a certain area, so you know the density increased. What's the explanation? We think, at least in the sands that we have been working with, there is only one and that is that the lateral stress is somehow decreased as a result of the operation. The Dilatometer tells us that sort of thing. The cone provides no information at all on that point, and that is just one example. Now Peter would like to respond with something else.

- Both tests are still actually at quite an early stage of their interpretation I think. The cone is maybe a little further ahead, simply because its been available longer, so its interpretation is maybe a few years ahead of the Dilatometer. But lateral stresses, particularly with the piezometer cone there are potentials that the data could be interrepreted to understand better the in situ stress and stress history of the soil from the electric piezometer cone, particularly for clays. John is correct I think that the Dilatometer as it is now appears to correlate quite well with stress history and in situ stress for clays. However I feel that in sand when looking at available chamber test data, it seems to confirm that the Dilatometer cannot distinguish between the effects of density and stress and stress history in sand. So for the measurement in situ stress, I don't think it works out well in sands, but it does work out well in clays. Better than the cone does. Except John's true in that last example which sounds quite difficult to explain.

A. OK I would just add in a quickie here. I certainly agree with the importance of horizontal stress fields. I suspect that something like the (inaudible) step taper blade (?) modified to run on a cone rod on a continuous rapid basis might

BACK

be run in parallel to the cones. We do a cone push and a step taper blade push. This might be a better way of proceeding, than going to the Dilatometer. It seems to me particularly at the Beaufort where we know we have a geostatic stress field, as well as an over consolidation problem, the step taper blade the cone combination would be able to differentiate there which we don't get from the Dilatometer.

This brings up one of the case histories I came equipped to show if I had time. I'll just hit the high points because its come up. This (transparency) is DMT data from a dunamic compaction job in New Mexico. Here there was a particular sand layer, underneath some gravel, that was treated by dynamic compaction and we had before and after Dilatometer tests. I_d is plotted here, and there is very little change in I_d essentially the same. The red represents the conditions before compaction, the green represents the conditions after. K_d increased slightly and the prediction for a pre-consolidation stress increased slightly. Those are stress dependent parameters, they both increased so you can surmise that the lateral stress has increased. The prediction of friction angle is a density dependent parameter and the friction angle increased by something like 4^o. The molulus changes result from combination of the two effects. The modulus increased rather significantly. I think there is some reason to be optimistic that in practice you can have some means of being able to distinguish stress and density effects, as a result of something like dynamic compaction.

A. I think you are right in the sense that if you look at all the parameters then, with judgement, you could infer certain things. But based on chamber tests done by Bellotti and Jamiolkowski in Italy where they had a large chamber filled with sand under controlled density and controlled stress conditions, the tests quite clearly showed that there was a relationship between the index KD and the density providing the KO stayed constant. If K_0 changed then the KD changed, and likewise

BACK

if you kept the density constant then there was a relationship between KD and K_o but that if you didn't know how the K_o or the density was changing then you wouldn't know which constituted either part. In Marchetti's esopt II paper he clearly states that he agrees with that. As you just stated, John, there was a case where you are probably exactly right in your interpretation but you don't have any concrete measured proof other than the inferred proof on the Dilatometer of the effect each of those factors have; The effect of changing density and changing stress level.

- We always have the problem of what is the truth. I have been involved with a lot of chamber tests. I don't want to bore you with the details, but I'm involved in all the tests that you have described, and there is a lot of interpretation that goes into those tests. We are really at a very early stage with the Dilatometer tests. There are about another dozen tests, I think Dr. Campanella mentioned that, another dozen tests scheduled for the Italian chambers. We have another couple of dozen scheduled for the University of Florida, and we are really at the early stages of knowing what's going on when we put the Dilatometer in. There are lots of details about chamber tests that effect the interpretations and they have not all been sorted out yet.
- This might be the moment when we are talking about Dilatometers and piezometer cones and self-boring pressure meters to pass on a piece of information. And this will be available for you near the door as you leave. From March 16th to 18th at the Sheraton World Hotel in Orlando, florida there is a conference on new methods in in situ testing, Dilatometer, piezometer cone, self-boring pressure meter. It's Tuesday, Wednesday, Thursday and Friday. The (inaudible) will be Dr. John Davidson, Associate professor of Civil Engineering at the University of Florida. Dr. John Schmertmann, principal of Schmertmann and Crapps, and Dr. (inaubible) Whistler who is a consulting engineer of Orlando, Florida.

BACK

So I'm quite certain if you want the latest on those instruments, and a good holiday to boot, that's the place to go.

\- Could I just make one comment regarding Dr. Schmertmann's statement that of course you would first do an SPT test because that's cheapest. I would say today we can do a cone penetration hole, piezometer cone at least if its to any significant depth a lot cheaper than SPT counted per foot or meter. We have done continuous cone testing piezometer cone to 100 meters, going through sand, with SPT blow counts of up to 75 completed in and out of the hole in less than 3½ hours. (I am Erik Funegard of Geosystems).

Q. What sort of time would it take, Dr. Campanella, at your test site at McDonald's farm to do a hole 200 feet, say a test of a meter in the hole. Equipment set up how long would it take with the Dilatometer.

A. We went out with Marchetti and the hole we put down went to 40 meters, and we tested at 20 cm. intervals, 200 tests. It takes about 30 seconds a meter to pull out the way we do it. I remember it took about 4 to 5 hours to do. It was a long half day.

BACK