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**APPLICATION OF FULL DISPLACEMENT PRESSUREMETER
IN SILTY SOILS**

by

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**A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfilment of the requirements
for the degree of**

DOCTOR OF PHILOSOPHY

in

Engineering*

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September, 1993**

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APPLICATION OF FULL DISPLACEMENT PRESSUREMETER IN SILTY SOILS

submitted by

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in partial fulfilment of the requirements
for the degree of Doctor of Philosophy

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ABSTRACT

Silty soils are widely distributed all over the world. There has been increasing offshore investigations for natural resources and land developments along coastal lines where silty soils are distributed. In testing silty soils, the literature shows that reliable results cannot be obtained from laboratory tests due to difficulties in sampling or severe sample disturbance. An alternative method is needed for obtaining reliable soil information.

The use of a self-boring pressuremeter is time consuming and expensive, despite its effectiveness in reducing disturbance. The full displacement pressuremeter (FDP) has many advantages over the self-boring pressuremeter in terms of practical engineering applications.

In this study, the installation of a FDP was examined in detail. The generalized Lamé's solution was used to solve the cylindrical annulus problem. A disturbance index was proposed in the numerical analysis for obtaining the shear modulus. Values of shear modulus were calculated from FDP tests conducted at three sites based on the analysis. For all the three sites, the new method led to consistent and repeatable results. Excellent agreement was found between the results of shear modulus from self-boring pressuremeter tests and those from the FDP tests. Values of compressibility measured

from laboratory tests were found to be much higher than those from the FDP tests.

Five methods were used to calculate the undrained strength. The results showed that only the limit pressure method leads to repeatable results. It was found as well, that the disturbance effect during probe installation on the undrained strength was very small for the limit pressure method of interpretation. For the three sites, the results of undrained strength from the FDP tests were compared with those from the self-boring pressuremeter, piezocone and field vane tests. The agreement was remarkable. As well, close values of undrained strength were obtained from laboratory tests and from the FDP tests.

Two case studies were carried out to illustrate the applications of the full displacement pressuremeter. The results of settlement based on laboratory consolidation tests grossly overestimated the settlements by a ratio of more than four times. Values of the settlements calculated from the FDP test results closely approximated the actual settlement values.

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Chapter 1

INTRODUCTION

1.1 INTRODUCTION

Compressibility and shear strength are two basic behaviours of soils. Almost all design problems in geotechnical engineering require the knowledge of these behaviours. Compressibility and shear strength conform to two design criteria: settlement and soil capacity. Depending on the type of the structure being used and properties of the subsoil, one of these two criteria will be the dominant factor. Sometimes settlement is the prime controlling factor, sometimes strength is the main criterion.

Laboratory tests have been the traditional way for studying these soil behaviours. Characteristics of compressibility are usually estimated from oedometer tests (also called consolidation tests). Results of oedometer test are expressed in terms of the compression index C_c or the constrained modulus M .

Values of shear strength are normally measured by triaxial tests. For clayey soils, the shear strength is commonly measured under undrained condition and expressed as the undrained strength S_u .

For cohesive soils, the process of applying the parameter of compressibility in settlement estimation is fairly standardised (Sutherland 1974). The compressibility characteristics of the soil are obtained from oedometer tests and then applied in Terzaghi's consolidation theory. This procedure requires good quality and representative soil samples which can be obtained through reasonably good investigation techniques.

Laboratory tests have many advantages. The boundary conditions are usually well defined and the drainage conditions can be strictly controlled. Also, the stress paths can be well defined, preselected and followed during testing processes. The soil nature and physical features can be positively identified in the laboratory.

There are, however, limitations with laboratory tests. The small volume of laboratory specimens cannot reflect the macrofabric of natural soil deposits. For cohesive soils, the effects of sample disturbance are sometimes difficult to assess. For soils such as silty soils, the disturbance effects could be so severe that the laboratory test results cannot represent the actual soil behaviour at all.

In-situ tests have many advantages in comparison with the laboratory tests. A larger volume of soil is tested than is done in most laboratory tests. Hence, the influence of macrofabric can be more accurately reflected by the in-situ tests. Some devices can produce a continuous record of the soil profile. This allows the soil macrofabric and layer boundaries to be determined. Also, the soils are tested in their natural environment, which may not be duplicated in laboratory tests. More importantly, in-situ tests can be carried out in soil deposits where undisturbed sampling is extremely difficult or unreliable.

Many in-situ tests have been developed. The common methods are listed below:

1. standard penetration test
2. field vane test
3. cone penetration test
4. dilatometer test
5. plate loading test
6. pressuremeter test

The standard penetration test has been applied widely in geotechnical engineering practice. Much experience has been accumulated in estimating parameters such as bearing

capacity and compressibility. One apparent limitation is that it is only suitable for sandy soils (Jamiolkowski et al., 1985; Bellotti et al., 1986).

The field vane was developed more than 30 years ago for measuring the undrained strength of soft clays. It was introduced to avoid the soil disturbance that occurs when taking samples for laboratory testing. It was suggested by Campanella and Robertson (1988) that the comparison of undrained strength from in-situ tests should be done with reference to the field vane values. The field vane can be used, as well, to determine the sensitivity of soils.

The cone penetration test has been used very successfully in determining the soil profile and identifying very thin soil layers. It can be used, as well, to interpolate ground condition between control boreholes. The most recently developed piezocones have been used for estimating pore pressure, overconsolidation ratio and shear strength. In evaluation of compressibility, however, the result from piezocone test is essentially preliminary in nature (De Ruiter, 1982; Robertson et al., 1982; Wroth, 1984).

The dilatometer is a semi-empirical method of estimating soil parameters. The probe is pushed into the soil to a desired depth and expanded laterally. Recordings are made of the corresponding lift off pressure for the plane of the blade, and the pressure after about 1 mm expansion of the blade plane. After corrections, three indices are obtained from these pressure values. These indices are correlated to soil parameters like modulus, shear strength, horizontal stress, overconsolidation ratio, etc. The popularity of this method has been increasing (Lutenegger, 1988; Schmertmann, 1988).

The plate loading test is a traditional method. It is not frequently used for common projects, as it is costly to run and time consuming. Also, the test depth of plate loading is very limited.

The pressuremeter test was introduced by Louis Menard many years ago. The test consists of placing a cylindrical probe in the ground and expanding the probe to

pressurize the soil horizontally. The pressure on the soil and the relative increase in cavity radius are obtained¹, which gives an in-situ stress-strain curve of the soil tested.

The pressuremeter test is one of the most promising in-situ test. The testing process can be modelled by cylindrical expansion theory. In theory, the boundary conditions are controlled and well defined. Direct measurements can be made of particular fundamental soil parameters. It has the potential for measuring in-situ both deformation and strength parameters at the same time.

Results from pressuremeter tests have many applications. They can be used in the designs of shallow foundations, vertically loaded or horizontally loaded piles, retaining walls, pressure grouted anchors and pavements.

Several types of pressuremeter device have been developed. They are the Menard pressuremeter, the self-boring pressuremeter (SBP), the push-in pressuremeter (PIP), and the full displacement pressuremeter (FDP).

The Menard type of pressuremeter test needs a pre-formed hole, into which the probe is lowered and the test performed at a certain depth. There are two ways to do this: one way is to complete drilling the entire hole at one time and then lower the probe into the borehole at several desired depths to perform a series of tests; the other way is drilling the hole to a certain depth, perform a test, then more drilling carried out for another test. Since the hole has to be formed ahead of the test, it is essential that a good quality borehole be formed. Nevertheless, the preformed hole will cause stress release in the soil, which will inevitably affect the test results.

The self-boring pressuremeter was developed to reduce the disturbance effect of the drilling process used in the Menard type pressuremeter test. It combines a self-boring device with a pressuremeter probe. With care in operation, disturbance of the soil during advancing can be greatly reduced. Also, the problem of stress release associated with pre-drilling is eliminated. The quality of this type of test has been greatly improved.

However, the amount of time and cost needed for the operation are very high. It has been used mostly for research purpose or on very important projects.

The push-in pressuremeter was developed originally for the purposes of off-shore investigation (Henderson et al., 1979). Currently, it is being used as well for on-shore investigation. The device is inserted by pushing either into an undersized pre-cored hole or into the bottom of a borehole without any pre-coring. Most of the experience with PIP has been in offshore applications.

A full-displacement pressuremeter (FDP) device is a pressuremeter fitted with a conical tip to ease the pushing of the device to the desired depth.

FDP is the most practical device among the pressuremeters. No pre-coring is needed before the test. The probe is inserted into the ground just as for a cone penetration test, except that it stops at desired depths for carrying out the pressuremeter test. The operation is easier than that of the Menard type or the self-boring pressuremeter. The amount of time needed for a whole profile tests is much less than the other two types. For the amount of time for one self-boring pressuremeter test, ten FDP tests may be performed. The cost of the equipment is also lower for FDP.

FDP can be combined with a cone penetrometer, which is commonly called "cone pressuremeter". This feature permits that a much wider range of soil information can be obtained with one test. The distinguished ability to identify soil stratigraphy by cone penetration test is well appreciated.

FDP also has great potential in offshore geotechnical investigations. The probe of the FDP can be pushed into the ground continuously, while other pressuremeters do not have this ability.

As a relatively new device, more research work is needed for the FDP test. Its installation method (insertion of the probe) is very different from other types of pressuremeters. Soil around the probe is deformed because of the installation. This raises

a number of questions. How does the deformation affect the test results? Is the current interpretation methods for other pressuremeters applicable to FDP test? How do the test results correlate with other in-situ test results? All these questions need to be answered before the FDP device can be used in practical applications.

As mentioned earlier, some soils are sensitive to sampling disturbance. Silty clay or clayey silts are such soils.

There is an increasing need for understanding the properties of silty soils. Silty soils are widely distributed along the world's sea coasts. There has been increasing offshore investigations for natural resources. Land developments along sea coasts have also been increasing because of land shortages. These soils are sensitive to any disturbance during drilling, which makes it difficult to obtain representative and relatively undisturbed soil samples. As a consequence, many cases have shown that settlements calculated with oedometer consolidation test results are much higher than the observed values (Landva, 1987; D'Elia, et al., 1974; Keenan, 1987). Sometimes the settlements calculated from oedometer test results are seven to ten times higher than the observed values (Landva, 1987).

In summary, the full displacement pressuremeter is very suitable for practical engineering applications. However, the effects of soil disturbance resulting from the installation need to be investigated. The applicability of current interpretation methods for the pressuremeter test needs to be examined in detail. For silty soils, the traditional oedometer test method could not produce a realistic estimation of compressibility because of sample disturbance and stress release. In-situ test methods, on the other hand, provide an opportunity to obtain more reliable measurements of compressibility for silty soils.

For pressuremeter tests in general, there are several needs for further research as stated by Mair and Wood (1987). It is not clear why ultimate strengths derived from pressuremeter tests is too high by comparison with other tests. Subsequently, it is unclear

whether the pressuremeter is giving a correct strength and other tests are giving incorrect strengths, or vice versa. There is also more need of comparisons of pressuremeter tests with other tests in different clays. Another problem is that there are very few case histories in which pressuremeters have been used to determine parameters which have been used in design and the performance subsequently compared with the calculations.

1.2 OBJECTIVES

The first objective of this study is to develop practical methods to determine the compressibility and undrained strength of silty soils from the full displacement pressuremeter test.

The second objective is to validate the proposed methods for full displacement pressuremeter test interpretations.

The third objective is to have a thorough understanding of the compressibility and strength characteristics of the silty soils being tested.

1.3 SCOPE OF WORK

In order to achieve the above objectives, this study consists of theoretical analysis, laboratory tests, in-situ tests and case studies.

The disturbance effect of the installation process for FDP test was analyzed by a numerical method. Laboratory tests included oedometer tests, undrained triaxial compression tests and simple shear tests. In-situ tests were the piezocone tests, the full displacement pressuremeter tests and the self-boring pressuremeter tests.

There are three sites where in-situ tests were performed: the test embankment site in Sackville, New Brunswick (NB); the Diplomatic Hotel site in Fredericton, NB; and

the National Research Council Campus, Montreal Road, Ottawa. The samples for conducting laboratory tests were obtained from the test embankment site.

1.4 OUTLINE OF THE THESIS

The background information is reviewed in the following chapters. Several cases of settlement calculations for silty soils are presented. These cases involve a multi-storey building, a satellite tower and an embankment problem. The settlement calculated from traditional oedometer tests is compared to the observed values. The development of the in-situ tests is briefly reviewed. Several in-situ test methods are described in detail: the dilatometer test, the piezocone test and pressuremeter test. Methods of estimating settlement with traditional consolidation tests and with the pressuremeter tests are also described.

Chapter three deals with the interpretation of the full displacement pressuremeter test. The disturbance effect of the probe installation is discussed and a detailed analysis was carried out to study the disturbance behaviour. A series of simple shear tests were performed to determine the parameters needed for the analysis. Based on the numerical analysis of the disturbance effect, a method of calculating the shear modulus was proposed. The FDP tests were performed at three silty clay sites and the repeatability of these results were examined.

Methods for evaluating undrained shear strength are examined in chapter four. Undrained strengths were obtained from the full displacement pressuremeter tests by available interpretation methods. Values from these method were compared in terms of magnitude and repeatability. The disturbance effect on the interpretation of undrained strength was also studied.

In order to validate the proposed methods for estimating the shear modulus and

undrained strength, results from the full displacement pressuremeter tests are compared to the laboratory tests in chapter five. The laboratory tests, includes sampling and testing process, are described. Results of shear modulus and undrained strength from the full displacement pressuremeter tests are compared with those from different laboratory tests.

Chapter six presents comparisons of the full displacement pressuremeter tests with other in-situ tests. The soil stratigraphy uniformity of the test site is evaluated by the results of piezocone tests. The repeatability of shear modulus results from the full displacement tests is evaluated. The results of compressibility from the full displacement pressuremeter tests are compared with those from the self-boring pressuremeter tests. The repeatability of undrained strength from the full displacement pressuremeter tests is studied. Results of shear strength from piezocone tests, vane tests and self-boring pressuremeter tests are presented and compared with the results from the full displacement pressuremeter tests.

In chapter seven, two case studies of estimating settlement are presented. One involves a multistorey building while the other deals with an embankment. The result of full displacement pressuremeter tests interpreted by the proposed method is applied in these two cases. A comparison is made of the results calculated from the full displacement test data and from traditional consolidation tests. The measured settlements are also presented and compared to the calculated values.

A summary is given in chapter eight, together with the conclusions.

In the appendix, the detail of the FDP testing is examined. The preparation procedure is illustrated, which includes saturation, volume and pressure calibration. The test procedure described including installation of the probe and process of loading and unloading. The data acquisition is also examined.

Chapter 2

LITERATURE REVIEW

2.1 DISTRIBUTION OF SILTY SOILS AND TYPES OF SOIL STRUCTURES

Silty soils (clayey silt or silty clay) are distributed all over the world. They are usually deposited in coastal areas. In Canada they are found in New Brunswick (Landva et al., 1987; Keenan et al., 1986), Vancouver (Konrad et al., 1985), Beaufort Sea (Kack et al., 1986; Konrad et al., 1985), Quebec (Lefebvre, et. al., 1991), and many other areas (Konrad et al., 1985, Landva et al., 1987, Law, 1991). They are also widely distributed in the United States (Bakhtar et. al., 1991), Norway (Senneset et al., 1988), Sweden (Borgesson, 1985; Bengtsson et. al., 1991, Bergenstahl and Lovisa, 1991), United Kingdom (Hude, 1985), Finland (Tammirinne and Leinonen, 1982), Italy (D'Elia and Grisolia, 1974), Iran (Behpoor and Ghahramani, 1989), Australia (Adikari et al., 1985), Spain (Cuellar et. al., 1991), Japan (Yoshida et. al., 1991), and large areas in China (Cheng-hou et. al., 1990).

The need for offshore geotechnical investigations is for the building of safe, economical oil and gas exploration and production structures. For onshore construction, the type of foundations or structures includes embankment, raft foundations, satellite foundations, tunnels, bridges, roads, etc.

One case for offshore structure is in the Beaufort Sea near Alaska and the Northwest Territories on the Arctic Ocean (Kack et al., 1986). The design and construction of artificial islands for hydrocarbon exploration required a precise definition of the behaviour of the island foundation. In 1985 a sandbag-retained island called ADGO G-24

needed to be built in the shallow waters of the Beaufort Sea by Esso Resources Canada Ltd. The previous geotechnical investigation for another island ADGO H-29 revealed that there was a layer of weak clayey silt layer about 7 m below the mudline. Hence further investigation was required prior to construction to determine whether stabilization was required.

At the mouth of the Fraser River in Vancouver there are similar silty soil deposits as in the Beaufort Sea. A series of in-situ tests, including piezocone tests, flat plate dilatometer tests, field vane tests and the Cambridge self-boring pressuremeter tests were conducted in the clayey silt deposits underlying 15 m of fine sand (Konrad et al., 1985). The purpose of this investigation is to understand the engineering properties of the foundation material for safer, more economical oil exploration structures for developing Canada's offshore energy resources.

2.2 CHARACTERISTICS OF SILTY SOILS

Silty soils usually have low plasticity. They are sensitive to sampling disturbance. The settlements estimated from traditional consolidation tests are usually much higher than actual values.

In the investigations of silty soils at Beaufort Sea, conventional soil sampling and laboratory testing do not provide sufficient information. From the laboratory testing program it was concluded that due to the low plasticity of the soils, it was impractical to prepare samples of the clayey silt for conventional laboratory testing (Konrad et al., 1985). Unconsolidated undrained triaxial tests on marine samples were shown to provide a very unrealistic estimate of material strength due to the generation of unrepresentative pore pressures in the sample. Hence geotechnical investigations have relied heavily on in-situ soil testing to provide the data necessary to define the foundation behaviour.

In the investigations of silty soils at the mouth of Fraser River in Vancouver, soil samples were obtained from 17.3 m to 24 m. It was concluded that samples of the clayey silt formation obtained with thin walled stainless steel Shelby tubes were considerably disturbed (Konrad et al., 1985).

Senneset et al. (1988) conducted piezocone tests and laboratory tests in silty soils. In this study, two offshore sites and one onshore site were included. The shear strength parameters determined from laboratory triaxial tests were used as the basis of comparison for the piezocone interpretations. It was pointed out that there are limitations for this kind of comparison since sample disturbance influenced the triaxial test results.

The disturbance of sampling for silty soils has a direct effect on the measurement of compressibility. For clay soils, Burland et al. (1977) illustrated in the state of the art review of foundation behaviour that simple traditional settlement calculations are usually adequate for practical purpose provided the appropriate in-situ soil properties have been obtained. For silty soils however, cases have shown that the traditional one dimensional method based on laboratory tests usually leads to overestimation of settlement. (Keenan et al., 1986; D'Elia and Grisolia, 1974; Landva et al., 1987)

In downtown Fredericton, New Brunswick, a multistorey building was built on a raft foundation for the use of the Bank of Montreal (Landva et al., 1987). This building rises nine stories above one level of underground parking. The size of the raft is about 40 m x 50 m. The raft is 1.06 m thick and has two-way reinforcement both at the top and at the bottom. The building is a reinforced concrete structure with precast panel cladding.

The soil stratigraphy at the site consists of a random fill varying between 1.8 m and 3.9 m in thickness, underlain by a coarse brown sand layer 2.4 - 3.6 m thick. The sand is underlain by 15 m of clayey silt and 15 m of silty clay, followed by layers of silt, sand, and clay extending to a depth of at least 43 m. The structure and the soil stratigraphy are shown in Figure 2.1.

Figure 2.1 Bank of Montreal building (Alkins, 1983)

During the construction of the bank building, four block samples were obtained from the upper portion of the clayey silt layer. The samples were taken by pushing open-ended 250 mm square steel containers into the clay exposed in the elevator foundation area.

Before the construction, several types of instrumentation were installed. This includes earth pressure cells, piezometers and various settlement points. The monitored performance data of the building including the net pressure increase, the settlement and the excess pore pressure are shown in Figure 2.2.

The settlement calculation based on the consolidation test results yielded a value of about 190 mm. It was noticed that the specimen of remoulded soil gave roughly the same results as those of the "undisturbed" soil samples. The measured settlement at the time of the end of construction was about 16 mm. The final settlement four years later was about 40 mm. (Landva et al., 1987).

Figure 2.2 Behaviour of foundation (Landva, 1988)

Therefore the settlement calculated from laboratory consolidation results is about 4 to 5 times of the actual settlement.

At Fucino Plains, which is about 90 km from Rome, Italy, there is a satellite communication station (D'Elia and Grisolia, 1974). One of the antennae is a paraboloid-shaped structure 29.56 m in diameter placed on a reinforced concrete cellular structure (Figure 2.3). The subsoil consists of grey slightly organic silty clays. The water table is 1.3 m below the ground. Undisturbed samples 87 mm in diameter were taken using a

Figure 2.3 Geometry of antenna tower and its foundation (D'Elia & Grisolia, 1974)

Denison-type sampler. The properties of the subsoil are shown in Figure 2.4. The oedometric compressibility parameters in Figure 2.4 refer to the virgin curve obtained using the Schmertmann construction. The total net pressure applied on the soil is given as 10 kN/m^2 . The radius of the structure is 14.51 m .

The settlement was calculated using the equation:

$$\Delta \rho = \frac{\Delta q I'}{k} \quad (2.1)$$

where $\Delta \rho$ is the settlement corresponding to a load increment Δq , I' is a nondimensional influence factor depending upon the ratio E_v/kD and Poisson's ratio, and D is the diameter, k is a constant reflecting the increase of modulus with depth.

Figure 2.4 Properties of the subsoil (D'Elia & Grisolia, 1974)

Figure 2.5 (a) average settlement; (b) total load and construction sequence (D'Elia & Grisolia, 1974)

The Young's modulus is determined by:

$$E = E_0 + kz \quad (2.2)$$

Where the modulus E_0 and k are determined from the ratio of Young's modulus to the undrained strength value. By comparing with the initial tangent modulus based on

undrained compression tests, the ratio was taken as 100. The final computed settlement which is the arithmetic mean between the settlement computed at the centre and that computed at the edge of the loaded area is found to be equal to 250 mm. Five settlement points were placed on the foundation upper floor and the values of settlement were monitored for four years. The final settlement measured was about 56 mm. The monitored settlement curve with loading versus time are shown in Figure 2.5. Therefore the calculated settlement is four times the actual settlement.

Table 2.1 Calculation of settlement from laboratory tests

depth From Base from Grd.		m_v (m^2/MN)	M (MN/m^2)	$\Delta\sigma'$ (kPa)	Settlement (mm)
3.75	7.50	2.30	0.43	9.30	133.7
8.75	12.50	3.00	0.33	8.50	159.4
13.75	17.50	1.80	0.56	7.80	87.8
18.75	22.50	1.00	1.00	5.00	31.3
23.75	27.50	0.80	1.25	3.70	18.5
28.75	32.50	0.70	1.43	2.90	12.7
33.75	37.50	0.50	2.00	2.30	7.2
38.75	42.50	0.50	2.00	1.80	5.6
43.75	47.50	0.50	2.00	1.40	4.4
$\Delta P=10$ kPa $R=14.51$ m				Total =	395 mm

With the values of the size of the structure, net pressure and the compressibility parameters given by the original authors, one dimensional settlement estimation was carried out. The detailed calculation is shown in Table 2.1. The final settlement calculated is 395 mm, which is about seven times of the actual settlement.

The third case was a full scale field test program (Keenan et al., 1986). In 1974, the New Brunswick department of Transportation approved the construction of a new bridge approximately 800 m long over the Saint John River in Fredericton, New Brunswick. At the location of the south abutment the underlying soils were found to be several layer of soft silty clays or clayey silt. Great difficulty was experienced in attempting to obtain "undisturbed" samples of the clayey silt using standard procedures.

Using preliminary laboratory results of 1974 and 1975, it was estimated that the compressible clayey silty subsoils would settle approximately 380 mm under the load of the high approach embankment at the south end of the bridge, which was not acceptable for the design. Hence, in 1976 the New Brunswick Department of Transportation and the Division of Building Research, National Research Council of Canada agreed to cooperate in a full scale field test program.

From the settlement results obtained from the full scale testing program, it was concluded that "standard sampling techniques could not be utilized to obtain undisturbed samples of the silty subsoil encountered in this area. It follows that consolidation tests conducted on these samples did not yield true data for calculating settlement".

2.3 GENERAL REVIEW OF IN-SITU TESTS

In recent years in-situ tests have been widely used in geotechnical investigations. The main reason for the increasing need of in-situ measurements of soil properties is that as our knowledge of the behaviour of real soils increases, so does our appreciation of the inadequacy of conventional laboratory testing (Wroth, 1984). The marked consequences of the inevitable disturbance that is caused in any soil specimen, however carefully it has been sampled, transported and reconsolidated in the laboratory, are all too evident. For the case of silty soils this is even more clear. As mentioned earlier, for some silty soils great difficulty was experienced in obtaining soil samples (Keenan et al., 1986). In other cases, where soil samples were obtained, the values of calculated settlement based on laboratory tests on these samples are four to seven times the actual measured settlement values (Landva et al., 1987; D'Elia and Grisolia, 1974).

Table 2.2 Applicability of in-situ test methods (Campanella and Robertson, 1983)

	Soil Type Profile	Density (D _r)	Angle of Friction (ϕ)	Undrained Shear Strength (s _u)	Pore Pressure (u)	Stress Histroy (OCR)	Modulus, Young's and Shear (E&G)	Compressibility (m _v & C _c)	Consolidation (c _b & 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
Elec. Piezo/Friction	A	A	A	B	B	A	A	B	B	A	B	B
Acoustic probe	C	B	B	C	C	-	C	C	-	-	-	C
Dilatometer	B	A	B	C	B	-	B	B	B	-	-	C
Vane Shear	B	C	-	-	A	-	B	-	-	-	-	-
Standard Penetration Test	B	B	B	C	C	-	-	-	C	-	-	A
Seismic CPT downhole	C	C	C	-	-	-	-	A	-	-	-	B
V _s Blade	-	-	-	-	-	-	B	-	-	-	-	-
Resistivity Probe	B	B	A	B	C	-	C	C	C	-	-	C
Borehole Permeability	C	-	-	-	-	A	-	-	-	E	A	-
Hudraulic fracture	-	-	-	-	-	B	B	-	-	C	C	-
Screw Plate	C	C	B	C	B	-	B	A	B	C	C	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
Self-boring Pressuremeter	B	B	A	A	A	A	A	A	A	A	B	A
Self-boring devices:												
K ₀ 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

A -High applicability

B -Moderate applicability

C -Limited applicability

There are many kinds of in-situ tests for different applications. A summary of in-situ tests and their perceived applicability is shown in Table 2.2 (Campanella and Robertson, 1983). Jamiolkowski et. al. (1985) classified three areas of soil behaviour which can be obtained through in-situ tests:

1. Assessment of initial state variables, which includes:
 - (1) soil profiling and identification,
 - (2) measurement of existing in-situ horizontal stress and
 - (3) assessment of the stress history of the soil deposit.
2. Deformation parameters
3. Flow and consolidation characteristics.

In respect of soil identification, the following in-situ tests appear to be particularly useful: 1) Electrical standard friction cone test (De Ruiter 1982); 2) Piezocone (Robertson and Campanella, 1982; Campanella and Robertson, 1988; Robertson et al., 1988); 3) Marchetti Flat Dilatometer (Marchetti 1980, 1982; Campanella et. al. 1985). These techniques make it possible to obtain detailed information on the soil profile, including the identification of different layers and the location of drainage boundaries.

The measurement of initial in-situ horizontal stress, σ'_{ho} or σ_{ho} , is still a most challenging task in geotechnical engineering. Despite much effort during the last two decades, the problem still seems far from being satisfactorily solved. The main difficulties are due to the fact that none of the existing in-situ devices can be inserted into the soil without disturbance. Thus the values of σ_{ho} measured by the different instruments are compared to each other or to σ_{ho} from laboratory tests, without really knowing the correct result (Jamiolkowski et al., 1985). One possible exception with soil disturbance while inserting to the ground is the self-boring pressuremeter.

At present there are no in-situ techniques which allow the direct determination of the maximum past pressure. Efforts have been made to determine the maximum past pressure through the piezocone tests (Konrad and Law, 1987; Robertson et al., 1988, Sully et al., 1988).

Because of the inevitable sampling disturbances, the most evident limitation of laboratory tests is in the measurement of deformation parameters, particularly when dealing with silty soils as described before. Hence the determination of deformation parameters by in-situ tests is considered to be of great practical interest. The self-boring pressuremeter test (Wroth, 1984; Mair and Wood, 1987; Baguelin et. al., 1978) is the most recommended in terms of test quality for measuring deformation parameters. Other test methods such as the cone penetration test (Schmertmann, 1972, 1978; Schmertmann et al., 1978; Campanella and Robertson, 1983; Robertson and Campanella, 1984), the dilatometer (Robertson and Campanella, 1990; Campanella and Robertson, 1990) and the plate loading tests (Kay and Parry, 1982; Marsland, 1971) can also be used for obtaining deformation parameters.

Several types of instrument can be used for studying the characteristics of flow and consolidation (Jamiolkowski et al., 1985). The holding test using the self-boring pressuremeter (Clarke et al., 1979) and the piezocone dissipation test (Baligh and Levadoux, 1980; Torstensson, 1975) are two examples among these tests.

Another area of soil behaviour which can be obtained by in-situ tests is the undrained shear strength of clayey soils. The self-boring pressuremeter test, field vane test (Wroth, 1984), piezocone test (Robertson and Campanella, 1982; Campanella and Robertson, 1988) and the flat dilatometer test (Lutenegger, 1988) can all be used for obtaining values of the undrained shear strength.

Among all the in-situ tests mentioned above, the two in-situ tests that hold out the most promise are the self-boring pressuremeter and the piezocone tests (Wroth, 1984).

The self-boring technique of insertion causes minimal disturbance and allows the possibility of "near perfect" testing of undisturbed soil. It has also the great advantage of leading to direct evaluation (without the need for empirical relationships) of the following important soil conditions and properties: in-situ lateral total stress σ_{ho} , shear modulus G , undrained shear strength S_u , and the horizontal coefficient of consolidation of clays c_h .

The piezocone is a tool of great promise for rapidly and reliably obtaining a soil profile. The measurement of pore pressures during the penetration of a cone has enhanced its importance considerably. In order to gain the maximum advantage from this test, the design of piezocones needs to be standardized

The vast increase in offshore search for natural resources has lead to special needs for offshore geotechnical investigations. It has been shown (Konrad et al., 1985, Kack et al., 1986) that the cone penetration test is particularly suitable in such investigations. Also the cone pressuremeter device which is a combination of the pressuremeter and the piezocone has great potential in offshore investigations.

The Department of Civil and Environmental Engineering at Carleton University has been very active in the area of interpretation and development of in-situ tests (Eden and Law, 1980; Law and Eden, 1980; Law, 1984, Law, 1985; Konrad and Law, 1987; Law, 1987; Bauer et al., 1988; Shields and Bauer, 1975; Bauer et al., 1973, Selvadurai, 1984).

Due to the limit of space, only the dilatometer, the cone penetrometer and the pressuremeter tests will be described here.

2.4. DILATOMETER TEST

Table 2.3 DMT tested materials (Lutenegger, 1988)

Material	Reference
Sensitive marine clay	Lacasse and Lunne (1983) Fabius (1985) Bechai et al. (1986) Hayes (1986) Lutenegger and Timian (1986) Lutenegger (1987)
Soft non-sensitive clays	Minkov et al. (1984) Ming-Fang (1986) Saye and Lutenegger (1986 a)
Lacustrine clay	Chan and Morgenstern (1986)
Glacial tills and/or very stiff over-consolidated clays	Davidson and Boghrat (1983) Schmertmann and Crapps (1983) Powell and Uglow (1986)
sand	Schmertmann (1982) baldi et al. (1986) Clough and Geoke (1986) Lacasse and Lunne (1986) Schmertmann et al. (1986)
Deltaic silt	Campanella and Robertson (1983) Konard et al. (1985)
Loess	Lutenegger and Donchev (1983) Hammandshiv and Lutenegger (1985) Lutenegger (1985,1986)
Peat	Hayes (1983) Kaderabek et al. (1986)
Compacted fill	Borden et al. (1985)
Industrial slime	GPE (1984)
Soft/medium rock	Sonnenfeld et al. (1985)

The Flat Dilatometer Test (DMT) was introduced by Marchetti (Marchetti, 1980) as a new in-situ penetration test for soils. The equipment and test procedure are simple

and the test provides repeatable nearly continuous data. These data have been empirically correlated to soil type, undrained shear strength, coefficient of earth pressure at rest, overconsolidation ratio and constrained modulus. Since the introduction of DMT, it has been widely used for a broad range of soils. Table 2.3 list the soil types that have been tested by DMT.

Figure 2.6 Marchetti Dilatometer (Marchetti, 1980)

The DMT consists of a flat-plate penetrometer which is instrumented with a flexible, circular diaphragm mounted on one face of the blade and a console to provide operational control. The dimensions and geometry of the blade are shown in Figure 2.6.

The testing procedure is summarized below (for more detailed information of the testing procedure see Schmertmann, 1986):

1. Push the blade into the ground to a desired test depth;
2. Expand the flexible diaphragm using compressed gas.

3. As the pressure is slowly increased and membrane moves outward against the soil, record the pressure when an electric signal identifies the pressure required for diaphragm to lift off the plane of the blade.
4. As the diaphragm expansion continues, record the pressure when a second electric signal denotes the central diaphragm expansion has reached 1.1 mm.

The two pressures are denoted as the A and B readings respectively. These two pressures are then corrected for diaphragm inertial resistance via a simple calibration procedure such that:

$$p_o = A + A_{\text{correction}} \quad (2.3)$$

$$p_1 = B - B_{\text{correction}} \quad (2.4)$$

The pressures p_o and p_1 are used along with an estimate of the vertical effective stress, σ'_{vo} and in-situ pore pressure, u_o , at the elevation of the test, to provide three indices denoted as:

$$\text{Material Index: } I_D = \frac{p_1 - p_o}{p_o - u_o} \quad (2.5)$$

$$\text{Horizontal Stress Index: } K_D = \frac{p_o - u_o}{\sigma'_{vo}} \quad (2.6)$$

$$\text{Dilatometer Modulus: } E_D = 38.2(p_1 - p_o) \quad (2.7)$$

The use of these indices for predicting a variety of soil parameters was proposed by Marchetti (Marchetti, 1980) who presented a series of empirical correlations based on numerous conventional laboratory and field test data.

Figure 2.7 Correlation between K_o and K_D (Marchetti, 1980)

Figure 2.8 Correlation between OCR and K_D (Marchetti, 1980)

The relationship of grain size distribution versus I_p is shown in Table 2.4

The relationship of coefficient of earth pressure at rest, K_o , for uncemented clays versus the Horizontal Stress Index, K_D , is shown in Figure 2.7. The relationship of overconsolidation ratio, OCR, for uncemented clays versus the Horizontal Stress Index, K_D is shown in Figure 2.8. The ratio of c_u/σ'_v versus the Horizontal Stress Index, K_D , is shown in Figure 2.9. With this relationship the undrained strength can be obtained.

Figure 2.9 Correlation between c_u/σ_v and K_D (Marchetti, 1980)

Table 2.4 Proposed Soil Classification Based on I_D Value (Marchetti, 1980)

Peat or sensitive clays (1)	Clay		Silt			Sand	
	Clay (2)	Silty clay (3)	Clayey silt (4)	Silt (5)	Sandy silt (6)	Silty sand (7)	Sand (8)
I_D Values	0.10	0.35	0.6	0.9	1.2	1.8	3.3

Since the initial introduction of DMT, relationships between soil parameters and the three basic indices have been proposed. They are shown in Table 2.5.

A third pressure reading designated as C may also be obtained by controlled gas deflation after obtaining the B-Reading and denotes the pressure at which the diaphragm recontracts the plane of the blade. The pressure p_2 is obtained from the C-Reading as:

$$p_2 = C - A_{\text{correction}} \quad (2.8)$$

Based on p_2 , Lutenege and Kabir (1988) proposed an additional DMT index U_D :

Table 2.5 Interrelationships between soil parameters and DMT Indices

Soil Parameter	DMT Index	Reference
s_u (clays)	I_D, K_D	Marchetti (1980)
ϕ' (sands)	I_D, K_D , thrust or adjacent q_c	Schmertmann (1982) Marchetti (1985)
K_o (clays)	I_D, K_D	Marchetti (1980, 1986)
K_o (sands)	K_D , thrust	Schmertmann (1982)
OCR (clays)	I_D, K_D	Marchetti, (1980)
OCR (sands)	K_D , thrust	GPE (1983)
M	I_D, E_D, K_D	Marchetti (1980)
E_i	I_D, E_D	Robertson et al. (1988)
E_{25}	I_D, E_D	Campanella & Robertson (1983) Baldi et al. (1986)
Cyclic stress ratio to cause liquefaction	E_D	Robertson and Campanella (1986)
k_s (subgrade reaction modulus)	p_o, K_D	Schmertmann and Vrapps (1983) Robertson et al. (1988)
CBR	E_D	Borden et al. (1985)

$$U_D = \frac{p_2 - u_o}{p_o - u_o} \quad (2.9)$$

This parameter approximates the total pore pressure around the DMT blade. It appears to have significant merit in determining site stratigraphy and stress history (Lutenegger, 1988).

2.5 CONE PENETRATION TEST

The development of the cone penetration test (CPT) comes from the initial purpose of probing with rods through weak soils to locate a firmer stratum. In the first stage only the load on the cone or the tip resistance was measured. Then a friction sleeve to measure local skin friction over a short length above the cone was introduced. The load was first measured by mechanical device. The new method use electric transducers mounted immediately above the cone or the friction sleeve to measure the load. For detailed review one can refer to the book "Cone Penetration Testing" (Meigh, 1987). Several types of cone penetrometer tips are shown in Figure 2.10 (Meigh, 1987).

The CPT has three main applications:

1. to determine the soil profile and identify the soils present
2. to interpolate ground conditions between control boreholes
3. to evaluate the engineering parameters of the soils

The latest development of the cone penetration test includes the measurement of pore pressure during the penetration (Baligh et al., 1981; Campanella and Robertson, 1981), which is called piezocone test or CPTU. Figure 2.11 presents examples of previous and current piezocone designs. The position of the pore pressure element was suggested to be located immediately behind the tip for the following reasons (Campanella and Robertson, 1988):

- 1 porous element is much less subject to damage and abrasion
- 2 measurements are less influenced by element compressibility
- 3 position is appropriate for correction due to unequal end areas
- 4 good stratigraphic detail is still possible

This new feature enhanced the functions of traditional cone penetration test greatly. The main advantages of the CPTU over the conventional CPT are (Campanella and

Figure 2.10 Cone penetrometer tips (Meigh, 1987)

Figure 2.11 Piezocones with various pore pressure element locations (Campanella & Robertson, 1988)

Robertson, 1988):

- * ability to correct measured cone data to account for unbalanced water forces due to unequal end areas in cone design,
- * ability to evaluate flow and consolidation characteristics,
- * improved soil profiling and identification,
- * improved evaluation of geotechnical parameters.

Due to these capabilities the CPTU has a wide range of applications in geotechnical engineering. The following review will be concentrated on the interpretation of CPTU results.

The basic values directly measured from CPTU are 1. tip resistance, q_c , 2. side friction, f_s , and 3. pore pressure, u . For soft clays, it is important to correct the pore

Figure 2.12 Unequal end area correction (Campanella & Robertson, 1988)

pressure effect for the tip resistance q_t . As the water pressure can act on the exposed surfaces behind the cone tip. (see Figure 2.12) These water forces result in measured tip resistance (q_t) do not represent true total stress resistance of the soil. The tip resistance q_t is corrected by the following equation (Baligh et al., 1981; Campanella et al., 1982):

$$q_T = q_c + u(1 - a) \quad (2.10)$$

Where:

q_T = corrected total tip resistance

u = pore pressure generated immediately behind the cone tip

a = net area ratio, see Figure 2.12.

Soil Profile

The major application of CPTU data has been for soil profiling. Traditionally soil classification from CPT data has been related to tip resistance q_c , and friction ratio, $FR = (f_s/q_c)100\%$. With CPTU one important parameter for soil classification is the pore pressure parameter ratio, B_q , defined as:

$$B_q = \frac{\Delta u}{q_T - \sigma_{vo}} \quad (2.11)$$

Where:

Δu = excess pore pressure ($u - u_o$)

q_T = cone resistance corrected for pore pressure effects

σ_{vo} = total overburden stress.

Several classification charts have been proposed based on q_T , or q_c , u or B_q , and friction ratio FR . The chart proposed by Senneset and Hanbu (1984) and modified to use q_T (Campanella and Robertson, 1988) is shown in Figure 2.13. The chart is based on pore pressure measured immediately behind the cone tip. Jones and Rust (1982) proposed a chart using Δu and q_c , which can consider the case of negative pore pressure. The chart is shown in Figure 2.14. Robertson et al. (1986) proposed a chart which can use all three pieces of data (q_T , u , f_s) in the form of q_T , B_q and FR as shown in Figure 2.15. These

Figure 2.13 Soil classification chart from data proposed by Senne set and Janbu (1984)

three charts are global in nature and should be used as a guide to define soil behaviour type based on CPTU data. However, it is strongly recommended that local correlations or regionally adjusted classification charts be developed (Campanella and Robertson, 1988). Factors such as changes in stress history, sensitivity, fissuring and cementation will influence the classification using either the FR or the B_q chart.

Undrained Shear Strength

Interpretation of soil strength from CPTU is dependent on drainage conditions. Generally undrained shear strength (s_u) is determined from undrained penetration. The undrained shear strength is not a unique parameter and depends on type of test, stress

Figure 2.14 Soil classification chart from CPTU data proposed by Jones and Rust (1982)

path followed, rate of strain and orientation of the failure planes (Wroth, 1984). Therefore CPTU data is generally interpreted based on empirical or approximate theoretical solutions. Different references have been used for comparison of s_u with many empirical methods. It was recommended (Campanella and Robertson, 1988) that the reference s_u should be taken as the field vane value and should be clearly stated.

The undrained shear strength s_u is usually estimated from the measured cone tip resistance, q_c , using the following bearing capacity equation:

$$s_u = \frac{q_c - \sigma_o}{N_k} \quad (2.12)$$

where N_k is an empirical or semi-empirical cone factor and σ_o is generally taken to be the total overburden pressure (σ_{vo}). With the corrected cone resistance q_T , the equation

Figure 2.15 Classification system from CPTU data (Robertson et al., 1986)

becomes:

$$s_u = \frac{q_T - \sigma_{v0}}{N_K} \quad (2.13)$$

Konard and Law (1987) classified the interpretation of cone penetration tests into five classes:

1. The classical plasticity approach to bearing capacity of deep foundations;
2. Cavity expansion theories combined with classical plasticity;
3. The principle of conservation of energy with cavity expansion;

4. Analytical and numerical approaches using linear and nonlinear stress-strain relationships;

5. Empirical studies attempting to relate cone resistance to undrained shear strength derived from different tests and in some cases, to additional soil index parameters.

Comprehensive reviews of s_u evaluation from CPT and CPTU data have been presented by Robertson et al. (1986), Campanella and Robertson (1988), Konrad and Law (1987). The method proposed by Konrad and Law (1987) will be presented here.

It is assumed that the unit shear stress at the cone-soil interface can be expressed as:

$$\tau = (\sigma_n - u_m) M \tan \phi' \quad (2.14)$$

where ϕ' is the effective friction angle of the normally consolidated soil and M is the friction factor for the soil acting on the cone surface.

The pore-water pressure is measured above the base of the cone. The pore pressure in the failure zone (u_m) is approximated by:

$$u_m = \alpha u \quad (2.15)$$

where α is a factor usually greater than 1. In soft soils α is in the range of 1 - 1.1. The assumptions of proposed stress field acting on cone and expansion of spherical cavity are shown in Figure 2.16. From Figure 2.16 with considerations of force equilibrium at failure give the following expression for a cone with apex angle θ :

$$q_T = \sigma_n + M \tan \phi' (\sigma_n - \alpha u) \cot \theta \quad (2.16)$$

With the introduction of $F = M \tan \phi' \cot \theta$ as an overall soil-cone parameter, the above equation can be written as:

Figure 2.16 (a) proposed stress field; (b) expansion of spherical cavity (Konard and Law, 1987.a)

$$\sigma_n = \frac{q_t + F\alpha u}{1 + F} \quad (2.17)$$

The pressure σ_n was assumed to be equal to the maximum internal pressure (P_{us}) required to expand a spherical cavity from zero radius to the cone radius R , which can be expressed as:

$$P_{us} = \frac{4}{3}(\ln I_R + 1)S_{uc} + \sigma_t \quad (2.18)$$

Where S_{uc} is the undrained shear strength mobilized by the cone, and I_R is the soil's rigidity index. Combining the two equations, one obtains the undrained shear strength mobilized during the penetration process as:

$$S_{uc} = \frac{q_T - (1+F)\sigma_i + F\alpha u}{\frac{4}{3}(1+F)(\ln I_R + 1)} \quad (2.19)$$

In the proposed method, q_T , u can be determined directly from CPTU tests. The other parameters σ_i , M , α , $\tan\phi'$ and I_R , need to be evaluated independently.

The initial total stress σ_i can be approximated by the total vertical stress if K_0 is not known. M can be taken between 0.5 and 0.75 for smooth steel surfaces. For usual roughness, M can be taken equal to unity. For general application, $\phi' = 35^\circ$ as an average value of effective friction angle for silty soils and $\phi' = 27^\circ$ for clays. The rigidity index is defined as G/s_u .

Deformation Characteristics

For many years engineers have tried to correlate the cone resistance with different soil deformation moduli in order to compute the settlements of foundations (De Beer and Martens, 1957; Trofimenkov, 1974; Schmertmann, 1972; Sanglerat, 1972; Dahlberg, 1975; Mitchell and Gardner, 1975; Meigh, 1985).

All these attempts may be grouped into two basic categories:

1. Methods correlating the observed settlements of existing structures with q_c measured before construction (Schmertmann, 1972; Schmertmann et al., 1978).
2. Correlations based on a comparison between field measured q_c and laboratory assessed deformation moduli.

The estimation of drained parameters such as the one dimensional compression index, C_c , or compressibility, m_v , from an undrained test should be treated with caution, especially when based on general empirical correlations. Mitchell and Gardner (1975)

made a comprehensive review of the numerous correlations between cone resistance and drained modulus. Most of these take the general form:

$$M = 1/m_v = \alpha q_c$$

Table 2.6 Estimation of Compression Index, C_c , from c_u/σ'_{vo} ratio (Schmertmann, 1978)

C_u/σ'_{vo}	approx. OCR	$C_c/(1+e_1)$
0 - 0.1	less than 1	greater than 0.4 (still consolidating)
0.1 - 0.25	1	0.4
0.26 - 0.50	1 to 1.5 (assume 1)	0.3
0.51 - 1.00	3	0.15
1 - 4	6	0.10
over 4	greater than 6	0.05

Sanglerat et al. (1972) developed a comprehensive array of α values for different cohesive soil types with different cone resistance values. Schmertmann developed a slightly more logical method that related to the c_u/σ'_{vo} ratio to the overconsolidation ratio (OCR) and then to the one dimensional compression index of the soil, C_c , as shown on Table 2.6.

For NC and lightly OC clays and silts up to firm in consistency (q_c less than about 1.2MN/m^2), a first approximation can be obtained by using the α_M values in Table 2.7. (Meigh, 1985).

These are applicable for a small stress increment (up to about 100kN/m^2) above in situ vertical effective stress.

The data from which the α_M values were obtained was presented by Sanglerat (1979). This data was based on cone values from a variety of penetrometers, but mainly from the Dutch mantle cone penetrometer. There is a good deal of uncertainty about the

Table 2.7 Coefficient of constrained modulus for normally-consolidated and lightly overconsolidated clays and silts (Sanglerat, 1979)

Soil	Classification	$\alpha_M (= M/q_c)$	
		Mantle cone (M)	Reference tip (R)
Highly plastic clays and silts	CH, MH	2 to 6	2.5 to 7.5
Clays of intermediate or low plasticity:	CI, CL		
$q_c < 0.7 \text{ MN/m}^2$		3 to 8	3.7 to 10
$q_c > 0.7 \text{ MN/m}^2$		2 to 5	2.5 to 6.3
Silts of intermediate or low plasticity	MI, ML	3 to 6	3.5 to 7.5
Organic silts	OL	2 to 8	2.5 to 10
Peat and organic clay:	Pt, OH		
$50 < w < 100\%$		1.5 to 4.0	1.9 to 5.0
$100 < w < 200\%$		1.0 to 1.5	1.25 to 1.9
$w > 200\%$		0.4 to 1.0	0.5 to 1.25

the ratio between q_c from the mantle cone and q_c from the reference test penetrometer tip (R). On a simple theoretical basis, considering the additional force on the mantle as the product of its area and the remoulded strength of the clay, the ratio can be expected to be between about 1.15 and 1.5 for clays of low or moderate sensitivity ($2 < S_r < 8$). Limited field evidence suggests a ratio of 1.25, and this has been adopted in arriving at the suggested α_M values for the reference tip in Table 2.7. These methods provide only a rough estimate of soil compressibility.

Stress History

Baligh et al. (1980) suggested that the pore pressure measured during undrained cone penetration may reflect the stress history of a deposit. Since then several methods have been suggested to correlate various pore pressure parameters to the overconsolidation ratio, OCR. A summary of the main pore pressure parameters were given by Campanella and Robertson (1988).

Methods have been suggested to relate CPTU data to preconsolidation stress (p'_c) (Konrad and Law, 1987). The proposed simplified stress condition is shown in Figure

Figure 2.17 Simplified stress conditions around a cone (Konard and Law, 1987.b)

2.17. The preconsolidation pressure is calculated by the following equation:

$$p'_c = q_n - u_m = \frac{q_T - \alpha u}{1 + M \tan \phi' \cot \theta} \quad (2.20)$$

Where α is a factor usually greater than 1. In soft soils α is in the range of 1 - 1.1. M can be taken between 0.5 and 0.75 for smooth steel surfaces. For usual roughness, M can be taken equal to unity. For general application, $\phi' = 35^\circ$ as an average value of effective friction angle for silty soils and $\phi' = 27^\circ$ for clays. Parameter θ is the cone apex angle.

A review of published correlations (Campanella and Robertson, 1988) shows that no unique relationship exists between the pore pressure ratios proposed above and OCR, because pore pressure measured at any one location are influenced by clay sensitivity, preconsolidation mechanism, soil type and local heterogeneity.

Wroth (1984) pointed out the basic soil characteristics for interpretation from pore pressure. Only the shear induced excess pore pressure reveals the nature of the soil behaviour and depends on stress history. With the complex nature of cone penetration it is not an easy task to isolate the shear induced pore pressures. It was suggested that the difference between pore pressures measured on the face and somewhere behind the tip may correlate better with OCR (Campanella et al., 1985; Sully et al., 1987, 1988).

2.6 PRESSUREMETER TEST

A pressuremeter test consists of placing a cylindrical probe in the ground and expanding the probe to pressurize the soil horizontally. An in-situ volume - radial displacement curve is obtained from the pressuremeter test. The pressuremeter test is repeated at various depths in order to get profiles of soil parameters.

Kogler, in 1933 in Germany, is credited with having developed the first preboring pressuremeter; however Kogler did not pursue his idea (Briaud, 1992). Serious development started only in 1955 in France. Menard developed a preboring pressuremeter to measure the in situ soil deformation properties. Since then a great deal of effort have been devoted in the development of pressuremeters and their interpretations. Progress has been made in the interpretation and use of pressuremeter data in France, England, Japan, Canada, Italy, Norway and the United States.

Pressuremeter testing is becoming more and more popular because of the fact that, in theory, the boundary conditions are controlled and well defined, as are the stress and strain conditions in the surrounding soil mass, and that its ability to measure in-situ both deformation and strength parameters at the same time. An empirical approach has been developed and practised in France over the past two decades with considerable

success. Results from the pressuremeter tests are empirically related to geotechnical parameters or, more directly, to performance of foundations. However, in recent years, the development of pressuremeter testing has pursued the general aim, followed in other forms of in-situ or laboratory test, of attempting to make direct measurements of the particular fundamental soil parameter required.

A major difference between categories of pressuremeter tests lies in the method of installation of the pressure device in the ground. There are four categories based on installation methods:

1. Menard pressuremeter (MPM)
2. Self-boring pressuremeter (SBP)
3. Push-in pressuremeter (PIP)
4. Full displacement pressure (FDP)

The Menard type pressuremeter is placed into a predrilled hole and inflated to provide a pressure-volume expansion curve. Figure 2.18 shows a Menard pressuremeter device made by Cambridge Insitu. Soil disturbance due to stress relief in the prebored hole is usually significant. Because of its sensitivity to disturbance, a standardized procedure is used to form the hole and to conduct the tests. The data from this type of probe is used mainly in an empirical manner.

In an effort to minimize soil disturbance, self-boring pressuremeters were developed in France and England (Bague et al., 1972; Wroth and Hughes 1974). Self-boring pressuremeters are all similar in principle in that they are, in essence, miniature tunnelling machines which are steadily jacked into the ground. The soil displaced by the instrument enters the cutting head where it is broken into small pieces by a rotating cutter, then flushed to the surface. Figure 2.19 shows the illustration of the Cambridge self-boring pressuremeter.

Figure 2.18 Menard pressuremeter device (Cambridge Insitu, Mair and Wood, 1985)

With the increasing interest in offshore investigation for natural resources and the consequent need to obtain good measurements of the in-situ properties of clays offshore, a pressuremeter was required which could be simply used with commercially available wire-line drilling equipment. This resulted in the development of the push-in pressuremeter test, PIP, (Henderson et al., 1979, Reid et al., 1979) which is illustrated in Figure 2.20. This is normally inserted by pushing either into an undersized pre-cored hole or into the bottom of a borehole without any pre-coring. The soil displaced by the probe enters the body of the instrument during insertion. Since there is a limiting depth

Figure 2.19 Details of a self-boring pressuremeter (Weltman and Head, 1983)

of penetration before soil fills the instrument, tests cannot be conducted on a continuous basis. Also the probe is made of a thick walled tube, disturbance will inevitably have occurred. The initial boundary conditions are far from clear.

The full displacement pressuremeter is an alternative approach to the push-in pressuremeter. The end of the probe has the same geometric shape as with a cone penetrometer. The probe is pushed into soft soil. In this case a significant but controlled amount of disturbance will occur. When combined with a cone penetrometer, it is called cone pressuremeter. Figure 2.21 shows an FDP probe used by Robertson (1982).

The parameters that can be measured from pressuremeter testing is as follows:

1. deformation modulus

Figure 2.20 Push-in pressuremeter device (Weltman & Head, 1983, Reid et al., 1982)

Figure 2.21 A full displacement pressuremeter probe in the inflated state (Robertson, 1982)

2. strength

- (a) undrained strength for clays or weak rocks
- (b) angle of shearing resistance for sands
- (c) angle of dilation for sands

3. in-situ total horizontal stress

From more specialized pressuremeter tests, coefficient of horizontal consolidation c_h , and angle of shearing resistance for clay, ϕ' , can also be estimated.

The shear modulus of G is a measure of its elastic deformation characteristics. Young's modulus may be estimated from G .

If the clay around the pressuremeter is behaving truly elastically, the shear modulus, G , can be calculated from the slope of the pressure-expansion curve. The relevant expressions are:

$$G = \frac{1}{2} \frac{dp}{d\epsilon_c} \quad (2.21)$$

or

$$G = V \frac{dp}{dV} \quad (2.22)$$

Where ϵ_c and V are, respectively, the cavity strain and current volume of the cavity.

Figure 2.22 Determination of shear moduli G_i and G_w

The initial slope of the $p : \epsilon_c$ or $p : V$ curves can thus be used to estimate the initial modulus, G_i (Figure 2.22). In soft clays, however, it is quite likely that plastic deformations begin at an early stage of the test. In order to ensure that the soil is all responding elastically, it is more satisfactory to determine the shear modulus of the clay from the slope of an unload-reload cycle performed after the expansion has clearly reached a plastic phase.

Figure 2.23 Range of shear stress for elastic response in elastic-perfectly plastic material (Wroth, 1982)

If an unload-reload cycle is to be treated as elastic, the range of unloading has to be not so large that plastic deformations occur (Wroth, 1984). The allowable size can be estimated for an ideal elastic-plastic material with shear stress - shear strain relation as shown in Figure 2.23 (Wroth, 1982). During the plastic phase of expansion in such a material, the shear stress in the wall of the cavity (and beyond) is equal to c_u , where an element of soil in the wall of the cavity has reached a point such as P on the stress-strain curve (Figure 2.23(a)). For an ideal plastic material, unloading from P is elastic until a shear stress of c_u in the opposite direction is reached at point X (i.e. a reduction of shear stress of $2 c_u$ can be applied). This corresponds to a reduction in radial stress (i.e. cavity pressure) by $2 c_u$, and it represents the change in stress required to take the soil at the wall of the cavity from a state of failure with the radial stress as the major principal stress to a state of failure with the radial stress as the minor principal stress. A curve can be constructed on the pressuremeter expansion diagram (Figure 2.23(b)), indicating the boundary of the region within which elastic unloading behaviour may be anticipated.

In practice, clays do not respond as ideal elastic-plastic materials, and changes of pressure considerably smaller than twice the estimated undrained strength should be used if significant hysteresis on the unload-reload cycles is to be avoided.

2.7 ANALYSIS OF FULL DISPLACEMENT PRESSUREMETER

An analysis of push-in pressuremeter test was presented by Houlsby and Withers (1988). In the analysis, the initial installation of the push-in pressuremeter was modelled theoretically as the expansion of a cylindrical cavity within the soil. The expansion phase of the pressuremeter test was modelled as a continued expansion of the same cylindrical cavity. The contraction phase was modelled as a cylindrical contraction. The stages of the pressuremeter test are shown in Figure 2.24, and the stress states are shown in Figure 2.25.

The following is a summary of the results of their analysis:

1. the expansion occurs at constant pressure (limit pressure)

$$P_L = \sigma_{ho} + s_u(1 + \ln I_r) \quad (2.23)$$

where:

P_L = the limit pressure;

σ_{ho} = initial horizontal stress;

s_u = the undrained shear strength;

I_r = the rigidity index;

2. initial unloading is at slope $2G$ from (P_L, ϵ_c) to $(P_L - s_u, \epsilon_c - 1/I_r)$;

where:

ϵ_c = pressuremeter strain at maximum expansion;

Figure 2.24 a. in-situ conditions; b. after installation; c. at maximum expansion; d. during contraction. (Houlsby & Withers, 1988)

3. plastic unloading occurs along the curve:

$$P = P_L - 2s_u \left\{ 1 + \ln(\sinh[e_s - e]) - \ln(\sinh \frac{1}{f_r}) \right\} \quad (2.24)$$

The unloading curve is illustrated in Figure 2.26. The shear modulus is obtained from the initial slope of the unloading curve. ($m = 1$ for cylindrical, $m = 2$ for spherical). The undrained strength is calculated from the intercept XY in the Figure. The intercept XY can be calculated as:

$$(e_s - e) \frac{dP}{de} = f \frac{2(2+m)}{3} s_u \quad (2.25)$$

Where f is a function purely of $(m+1)(e_s - e)/2$. The value of f is near unity for all realistic values.

Figure 2.25 stress state: (a) at maximum expansion; (b) during contraction (Houlsby & Withers, 1988)

Figure 2.26 constructions to determine G and s_u (Houlsby & Withers, 1988)

2.8 SETTLEMENT CALCULATION

2.8.1 Traditional Terzaghi's method

Terzaghi proposed a method to calculate settlement based on the oedometer test. It is assumed that compression of the subsoil takes place without transverse deformations and that consolidation is uniaxial. For clay soils this method is still the most used method for calculation of consolidation settlement.

Usually the data from a oedometer test is plotted as loading pressures (σ'_z) versus corresponding values of void ratio (e), where the pressure is in logarithmic scale and the void ratio is in normal scale. The compression index C_c can be obtained from the $e : \log(\sigma'_z)$ curve by the following equation:

$$C_c = - \frac{\Delta e}{\Delta \log(\sigma'_z)} \quad (2.26)$$

Another way to describe the stress-strain behaviour in confined compression is the coefficient of volume change m_v , which is the reciprocal of constrained modulus, M :

$$m_v = \frac{\Delta e_v}{\Delta \sigma_v} = \frac{1}{M} \quad (2.27)$$

For normally consolidated soil the settlement can be calculated from the compression index C_c :

$$s_{oed} = \sum_{i=1}^n \Delta h_i \frac{C_c}{1+e_o} \log\left(\frac{\sigma'_z + \Delta \sigma'_z}{\sigma'_z}\right) \quad (2.28)$$

With the coefficient of volume change m_v , the settlement can be calculated as:

$$s_{oed} = \sum_{i=1}^n \Delta \sigma'_z m_v \Delta h_i \quad (2.29)$$

Where:

s_{oed} = settlement from oedometric compression;

Δh_i = the thickness of the layers into which the subsoil is divided for the purpose of calculation;

e_o = the initial void ratio;

σ'_z = initial vertical effective stress;

$\Delta \sigma'_z$ = the increase of vertical stress because of net pressure increase.

With overconsolidated soil the effect of preconsolidation pressure should be take into account in the calculation. The vertical stress increase is usually calculated from theory of linear homogeneous isotropic halfspace.

2.8.2 Settlement From Pressuremeter Data

The pressuremeter settlement equation was proposed by Menard and Rousseau (1962). The basis for this approach was given by Baguelin et al. (1978). Total settlement was divided into two components: an undrained or no volume change settlement s_u which takes place rapidly and a consolidation settlement. For a rigid circular plate on an elastic half space the undrained settlement is calculated by:

(2.30)

The consolidation settlement s_c is calculated as:

$$s_c = \frac{\pi}{16} (1-2\nu') \frac{qB}{G} \quad (2.31)$$

Hence the total settlement is given as:

$$s_t = \frac{\pi}{16} \frac{qB}{G} + \frac{\pi}{16} (1-2\nu') \frac{qB}{G} \quad (2.32)$$

The above equation is the backbone of the pressuremeter equation for settlement (Menard and Rousseau, 1962):

$$s = \underbrace{\frac{2}{9E_d} q B_o \left(\lambda_d \frac{B}{B_o} \right)^{\alpha}}_{\text{deviatoric settlement}} + \underbrace{\frac{\alpha}{9E_c} q \lambda_c B}_{\text{spherical settlement}} \quad (2.33)$$

Where:

- s = final footing settlement;
- E_d = modulus within the zone of influence of deviatoric tensor;
- E_c = modulus within the zone of influence of spherical tensor;
- q = footing net bearing pressure;
- B_o = reference width of 60 cm;
- α = rheological factor, (Table 2.8);
- λ_d, λ_c = shape factor (Figure 2.27);

For the calculation of E_c and E_d , Figure 2.28 shows the decomposition of the soil for settlement analysis. E_c is taken as the modulus of the first layer under the footing:

$$E_c = E_1 \quad (2.34)$$

E_d is taken as an equivalent modulus within 16 layers of $B/2$ thick (Menard, 1975):
where $E_{p/q}$ is the harmonic mean of the moduli of layers p to q . For example:

Table 2.8 Menard's factors (Briaud, 1992)

Soil type	Peat		Clay		Silt		Sand		Sand and gravel	
	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α
Over-consolidated			> 16	1	> 14	2/3	> 12	1/2	> 10	1/3
Normally consolidated	For all values	1	9-16	2/3	8-14	1/2	7-12	1/3	6-10	1/4
Weathered and/or remoulded			7-9	1/2		1/2		1/3		1/4
Rock	Extremely fractured $\alpha = 1/3$		Other $\alpha = 1/2$				Slightly fractured or extremely weathered $\alpha = 2/3$

Figure 2.27 Menard's shape factors (Briaud, 1992)

$$\frac{1}{E_d} = \frac{1}{4} \left(\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3/4/5}} + \frac{1}{2.5E_{6/7/8}} + \frac{1}{2.5E_{9/16}} \right) \quad (2.35)$$

$$\frac{3}{E_{3/4/5}} = \frac{1}{E_3} + \frac{1}{E_4} + \frac{1}{E_5} \quad (2.36)$$

In the case of a raft or an embankment rests on a soft layer which is thinner than $B/2$, the settlement of the soft layer is calculated as (Baguelin et al., 1978):

Figure 2.28 Division of soil layers for settlement analysis (Briaud, 1992)

$$s = \sum_{i=1}^n \frac{\alpha_i \beta \Delta \sigma_i}{E_i} \Delta z_i \quad (2.37)$$

Where n is the number of layers constituting the soft layer and β is a function of the safety factor, F :

$$\beta = \frac{2}{3} \times \frac{F}{F-1} \quad (2.38)$$

A new pressuremeter approach was proposed by Briaud (1992) based on the elasticity formula (Janbu et al., 1956):

$$s = \mu_0 \mu_1 (1 - \nu^2) q \frac{B}{E} \quad (2.39)$$

Where:

μ_0, μ_1 = influence factors (Figure 2.29);

ν = Poisson's ratio;

In the new approach, the modulus E is replaced as E_{eff} , which can be obtained from a special pressuremeter test (Briaud et al., 1987).

Figure 2.29 Chart for elasticity influence factor (Janbu et al., 1956)

Chapter 3

INTERPRETATION OF SHEAR MODULUS FROM FDP TEST

3.1 INTRODUCTION

The use of in-situ tests to complement laboratory tests is becoming more and more important in geotechnical engineering (Jamiołkowski et al. 1985; Meigh et al. 1987). Since Menard developed the first pressuremeter, a lot of effort has been devoted to the improvement of the pressuremeter test. The main reason is that compared with other in-situ tests, the pressuremeter test has one special attraction. Theoretically, the boundary conditions are well defined (Figure 3.1) and controlled during the test (Baguelin et al. 1978; Mair and Wood 1987). However, the advanced pressuremeters, such as the self-boring pressuremeter, have become so complicated, that they are less practical from the engineer's point of view. For the self-boring pressuremeter device, despite the good quality of its test results in terms of reduced disturbance effect, and the ability in obtaining in-situ horizontal stress and modulus, its application is still limited to research work and a few important construction projects. Even when it is used in "important" projects, the reason to use it is often because of the interest of researchers.

The development of the full displacement pressuremeter could change the direction towards more practical applications. The principle and procedure of the full displacement pressuremeter test are described in Appendix A. Compared with other pressuremeter devices such as the Menard pressuremeter and the self-boring pressuremeter, the full displacement pressuremeter device is easy to operate, time efficient and cost effective. With a 20 m borehole, 20 to 25 tests can be done in one day's work (8 hours).

Figure 3.1 Boundary conditions for a pressuremeter test and installation of the FDP

The particular device used in this study requires no power supply, permitting tests to be conducted at remote places. This feature is especially helpful for preliminary geotechnical investigations, since often in the stage of preliminary investigation, it is difficult to have access to a power supply on site.

However, as the full displacement pressuremeter is pushed into the soil in a way different from the Menard or the self-boring pressuremeter, the method for interpreting test results used for the Menard or the self-boring pressuremeter cannot be directly applied to the full displacement pressuremeter test. A new method is needed, which should be able to take into account the effect of the installation disturbance.

In the following sections, the installation process of a full displacement pressuremeter test will be examined in detail. The disturbance effect during installation will be analyzed numerically by elastic theory. Shear modulus will be obtained based on the results of this disturbance study.

3.2 INSTALLATION OF FDP DEVICE

The installation process is illustrated in Figure 3.1. The soil around the tip of the FDP device is divided into three areas: A, B, and C. During the penetration of the tip, soil in areas C and B mainly experience lateral movement, which can be modelled by cylindrical expansion. This process is called "installation expansion" hereafter. Not all the soil is expanded laterally. Part of the soil in area A would be pushed downwards. The soil at the surface of the tip and the probe will experience severe shearing movement and remoulding. The neighbouring soil will also be affected by the shearing. Consequently, an annulus of disturbed soil is formed around the probe after the installation.

In order to help us understand the installation process better, an imaginary initial

equivalent diameter, d_{eq} , is introduced, as shown in Figure 3.1. The installation process can be described as if the soil (B and C in Figure 3.1) is expanded from the equivalent diameter cylindrically to the size of the probe diameter. Therefore the pressuremeter test (cylindrical expansion) can be considered to have started at the time of probe installation (insertion). As a consequence, the pressure at the time of the inflating of the probe would be higher than the initial horizontal pressure for the soil.

One known effect of installation expansion is that pressures when the probe starts

Figure 3.2 Illustration of two stages of the expansion

to expand are higher than the initial in-situ values. For the FDP test, the test data or the curve is obtained after the installation is complete. The cylindrical expansion process, as shown by the dotted line AB in Figure 3.2, actually starts before the recording of the test data. Hence, the test data, or the volume - pressure curve represents only the information of the second phase of the cylindrical expansion, as represented by the line BC in the

same Figure. This was found to be consistent with the observations from the FDP test results performed in this study.

3.3 ANALYSIS OF DISTURBANCE EFFECTS

Several people have studied the disturbance effects of the Menard pressuremeter tests (Baguelin et al., 1972, 1978; Ladanyi, 1974; Sayed, 1988). In their studies, one thin layer of disturbed soil was considered. For the FDP test, however, the installation method is very different from the Menard type or the self-boring type. The soil around the FDP probe is not disturbed by drilling, but by the mixed actions of shearing and expansion. One layer of disturbed soil does not adequately reflect the extent and degree of disturbance.

The installation of the FDP device is similar to the penetration of a cone

Figure 3.3 Model test results (Mlynarek, 1982)

penetrometer. Much work (Mlynarek et al., 1982; Borst et al., 1982; Koumoto et al.,

1982) has been devoted to the analysis of static cone penetration. As the insertion processes of the FDP test and the cone test are almost identical, valuable information can be obtained from their results of soil disturbance effects. Mlynarek et al. (1982) conducted a model test of static cone penetration in clay. Figure 3.3 (redrawn from the original figure) shows their result of the deformed shape of the clay around the probe. The model test result indicates that the disturbance is greatest at the surface of the probe and decreases with distance away from the probe. Borst et al. (1982) and Koumoto et al. (1982) used the finite element method (FEM) to analyze the plastic zone induced by cone penetration. They found that the radius of the plastic zone ranges from 2 to 3 times R_0 , where R_0 is the radius of the cone device.

The above consideration suggests that the characteristics of the disturbed annulus is

Figure 3.4 Assumptions for the disturbed annulus

not uniform. The annulus could be characterized by layers of material with different

moduli values. The reduction of the modulus depends on the degree of disturbance, which is related to the vertical movement of the soil (Figure 3.4). The variation of the modulus therefore can be assumed to follow the pattern of the vertical movement of the soil around the probe (Figure 3.4b). The distribution or change of modulus with r can be described as:

$$G = G_0 \left[1 - (1 - \alpha) \frac{R_0^2}{r^2} \right] \quad (3.1)$$

where r = radius, G_0 = the shear modulus of the undisturbed soil, $\alpha = G_1/G_0$ at $r = R_0$, where G_1 is the shear modulus of the soil at the soil/pressuremeter interface.

Figure 3.5 Change of shear modulus with shear strain

A qualitative approach can be given to explain the reasons for the above assumptions. These explanations are based on the following three aspects: 1) modulus-strain relationship can be obtained from simple shear test results; 2) modulus increases with the increase of confining pressure; 3) remoulding effect at the soil-probe interface greatly reduces the modulus.

Figure 3.5(1) shows a typical shear stress - strain relationship from a simple shear

Figure 3.6 Shear strain distribution around FDP probe.

Figure 3.7 Modulus distribution around the probe relating to shear strain

test. A tangent modulus (of given strain) - strain relationship can be obtained from the stress - strain relationship and is shown in Figure 3.5(2). For the soil around the probe, the relationship of vertical displacement with the radius is shown in Figure 3.6(1). This relationship illustrates the result of the model test as shown in Figure 3.3. The strain - radius relationship can be evaluated from the displacement - radius relationship and the

result is shown in Figure 3.6(2).

Figure 3.8 Change of modulus because of horizontal stress change

Figure 3.9 Modulus distribution after considering stress and strain effects

From the shear strain - radius relationship and the modulus - shear strain relationship, the modulus - radius relationship can be estimated. The graphical procedure is shown in Figure 3.7. The result of the modulus - radius relationship is shown in Figure 3.7(3). It is known that for any soil test, the modulus will increase with the increase of confining pressure. This phenomenon is shown in Figure 3.8(1). For soil around the probe, the horizontal stress induced by the probe installation is shown in Figure 3.8(2). Combining the horizontal pressure - radius relationship and the modulus - pressure increase relationship, one can estimate the increase of the modulus with radius, as shown in Figure 3.8.

The results shown in Figure 3.7(3) and Figure 3.8(3) were redrawn in Figure 3.9. In Figure 3.9(2), line ab represents the modification of modulus decrease because of the remoulding effect at the probe-soil interface. Then the overall result of modulus distribution is obtained by combining Figure 3.9(1) and Figure 3.9(2) and is shown in Figure 3.9(3), as assumed in Equation 3.1.

Figure 3.10 Stress path for a pressuremeter test

A test curve and the corresponding stress path of a FDP test is shown in Figure 3.10. The shear modulus is calculated from the unloading-reloading curve C-D-E. As

suggested by Wroth (1984), the path C-D-E will be in elastic range if the unloading magnitude does not exceed $2s_u$, where s_u is the undrained strength. Hence elastic behaviour can be assumed in the study of disturbance on the shear modulus which is obtained from the unloading-reloading curve.

The generalized Lamé's solution is used for solving the problem of n layers of cylindrical annulus (Sayed et al., 1987). The matrix form of the equation is:

$$[D]\{x\}=\{b\} \quad (3.2)$$

where $[D]$ = stiffness matrix, $\{b\}$ = applied pressure vector, and:

$$\{x\}^T=\{A_1 \ C_1 \ A_2 \ C_2 \ \dots \ A_n \ C_n\} \quad (3.3)$$

where A_i (with the unit of a force), C_i (with the unit of a pressure) are constants to be determined.

$$\{b\}=\{-P \ 0 \ 0 \ \dots \ 0\} \quad (3.4)$$

where P is the pressure applied on the inner surface of the cylinder.

For $i = 2, 4, \dots, 2n-2$, the i^{th} row of the matrix $[D]$ is:

$$\{d_i\}=\{0 \ 0 \ \dots \ \frac{j}{R_i^2} \ 2 \ -\frac{j+3}{R_i^2} \ -2 \ \dots \ 0 \ 0\} \quad (3.5)$$

where $j = i-1, l=i/2$.

For $i = 3, 5, \dots, 2n-1$, the i^{th} row of matrix $[D]$ is:

$$\{d_i\}=\{0 \ 0 \ \dots \ -\frac{j}{2G_l R_l^2} \ \frac{1}{\bar{K}_l} \ \frac{1}{2G_{l+1} R_{l+1}^2} \ \frac{1}{\bar{K}_{l+1}} \ \dots \ 0 \ 0\} \quad (3.6)$$

where $j = i-2, l = (i-1)/2$.

The first row and the last row are expressed as:

$$\{d_1\} = \left\{ \frac{1}{R_0^2} \ 2 \ 0 \ \dots \ 0 \ 0 \right\} \quad (3.7)$$

$$\{d_{2n}\} = \left\{ 0 \ 0 \ \dots \ \frac{1}{R_n^2} \ 2 \right\} \quad (3.8)$$

The circumferential strain for the i^{th} layer is computed from the equation:

$$\epsilon_{\theta,i} = -\frac{A_i}{2G_i r^2} + \frac{C_i}{\bar{K}_i} \quad (3.9)$$

where:

$$\bar{K}_i = \frac{E_i}{2(1-2\nu_i)(1+\nu_i)} = \frac{G_i}{(1-2\nu_i)} \quad (3.10)$$

and G_i , ν_i = shear modulus and Poisson's ratio of the i^{th} layer, respectively. Under different pressures of P1 and P2, the corresponding strain $\epsilon_{\theta,i}(1)$ and $\epsilon_{\theta,i}(2)$ can be calculated as:

$$\epsilon_{\theta,i}(2) - \epsilon_{\theta,i}(1) = \frac{1}{G_i} \{ [-A_i(2) + A_i(1)] / (2R_0^2) + [C_i(2) - C_i(1)] * (1 - 2\nu_i) \} \quad (3.11)$$

where subscript 1 refers to parameters associated with the first layer, (1) and (2) corresponding to pressure, P1 and P2, respectively. Let

$$\{ [-A_i(2) + A_i(1)] / (2R_0^2) + [C_i(2) - C_i(1)] * (1 - 2\nu_i) \} = N \quad (3.12)$$

Then the apparent shear modulus G_{app} can be obtained by:

$$G_{\text{app}} = \frac{P2 - P1}{2(\epsilon_{\theta,i}(2) - \epsilon_{\theta,i}(1))} = \frac{G_i(P2 - P1)}{2N} = \frac{\alpha G_i(P2 - P1)}{2N} \quad (3.13)$$

where $\alpha = G_1/G_0$, G_1 is the modulus at soil-probe interface. G_0 is the modulus for intact soil.

The disturbance index I_p is defined as:

$$G_{ap} = G_0(1 - I_p) \quad (3.14)$$

then

$$I_p = 1 - \frac{G_{ap}}{G_0} = 1 - \frac{\alpha(P2 - P1)}{2N} \quad (3.15)$$

Determination of α

The ratio of the modulus at the soil-probe interface to the modulus of intact soil is defined as $\alpha = G_1/G_0$. In a simple shear test, the initial modulus calculated from the linear range of strain - stress curve can be used to represent the modulus for intact soil. The modulus calculated at high strain or near failure can be used to represent the modulus at the probe-soil interface right after the full displacement pressuremeter device is installed.

The stress state of the soil during the installation or the shearing process is affected by several factors. As a consequence of the installation of the probe, there is an increase in the horizontal stress beyond the initial horizontal stress. The vertical shear stress also increase and varies with the radius. Before the installation, the principal stresses are in the horizontal and the vertical directions. During the installation, which can be described as a shearing process, the directions of the principal stresses will change or rotate as a result of the changing horizontal stress and vertical shearing force. This process cannot be fully modelled by the traditional tests, where the principal stress directions remain horizontal and vertical. The desired laboratory test should be able to consider the effect

of principal stress rotation.

The laboratory tests were conducted at the soil laboratory in the former Geotechnical Section, National Research Council of Canada. The shear apparatus used is the "Simple Shear Tester" made by Seiken Inc., Tokyo, Japan. As principal stress rotation needs to be taken into account in the test, a new control system was developed to replace the old control system. In the old control system, the horizontal loading signal was generated by a function generator. The new control system consists of an IBM PC computer, a transmission board which can convert electrical signals to digital signals or digital to electrical, and several electric-to-pressure transducers. Software was developed for the controlling system. A scientific analysis language called ASYST was used in the software development.

In the testing process, the angle of principal stress rotation is entered first. Calculation is carried out and the resulting pressure values for the horizontal and the vertical loading are sent to the E/P transducer from the computer through the D/A board. The pressure values are read from the pressure transducer and the data sent to the computer. Comparisons are carried out with the prescribed values and adjustments made through proper output signals from the computer. The flow chart of the control system is shown in Figure 3.11.

The soil specimens used are from the 203 mm diameter samples obtained from the embankment site in Sackville, New Brunswick in August 1989. In-situ tests were also conducted at the same site. The samples were covered and sealed with wax on the site. They were transported with care to the National Research Council soil laboratory in Ottawa and stored in a humid storage room. The depth from which the samples were obtained are 6.3 m and 6.6 m. The soil is silty clay with slight grey colour. There are organic materials contained in the soil.

A series of simple shear tests was performed in the summer of 1991, using the

Figure 3.11 Flow chart of the new control system for simple shear test

Table 3.1 Information for simple shear tests

sample#	Depth(m)	Trimming angle	Cell Pressure (kPa)	Back Pressure (kPa)
F1-T12-E2-#7	6.3	90	300	200
F1-T12-E2-#3	6.3	90	245	200
F1-T12-E2-#2	6.3	90	245	200
F1-T12-E2-#10	6.3	0	300	200
F1-T13-F1-#19	6.6	0	247.5	200
F1-T13-F1-#17	6.6	0	247.5	200
F1-T13-F1-#15	6.6	0	247.5	200
F1-T13-F1-#14	6.6	0	247.5	200

newly developed computer aided control system. Table 3.1 shows information on the specimen number, consolidation pressure, etc. The samples were trimmed in two directions. One was the same as the in-situ (vertical) direction. The other way was 90 degrees (horizontal) from the original direction. For the specimen trimmed in the original direction, different rotations of principal stresses were used and were maintained during the testing process.

Figure 3.12 to Figure 3.14 show the results of the controlled principal stress rotation. It can be seen from these results that the control system was able to alter the pressure to achieve the desired principal stress rotation angles. Figure 3.15 to Figure 3.20 show the results of shear stress - shear strain relationship from these simple shear tests.

Figure 3.12 Angle of controlled principal stress rotation, #19 (unit: degree)

Figure 3.13 Angle of controlled principal stress rotation, #14 (unit: degree)

Figure 3.14 Angle of controlled principal stress rotation, #2 (unit: degree)

Figure 3.15 Result of controlled principal stress rotation test, #19

Figure 3.16 Result of controlled principal stress rotation test, #14

Figure 3.17 Result of controlled principal stress rotation test, #2

Figure 3.18 Result of controlled principal stress rotation test, #3

Figure 3.19 Result of controlled principal stress rotation test, #7

Figure 3.20 Result of controlled principal stress rotation test, #10

The initial shear modulus and the shear modulus at high shear strains were calculated from these graphs. The results of initial shear modulus and the tangent shear modulus at high strain are shown in Table 3.2. From the table we can see that the ratio of modulus ranges from 0.009 to 0.04. The average value of the modulus ratio is 0.024.

A computer program was compiled for analyzing the effect of the disturbed annulus and calculating the disturbance index, I_p . The program was written in QuickBasic for IBM PC or compatible. Values of Poisson's ratio and modulus ratio, α , need to be entered first. The modulus is calculated by Equation 3.1. The items for matrix [D] and {b} are computed with values of the radius and the corresponding modulus. Two values of pressure are assigned and Equation 3.2 is solved which gives the result of vector {x} in Equation 3.3 under the loading of these pressures. Finally, the disturbance index I_p is

Table 3.2 Results of modulus ratio from simple shear tests

sample#	G _{initial} (MPa)	G at large strain (MPa)	ratio α	Shear strain (%)
F1-T12-E2-#7	2.6	0.073	0.028	12
F1-T12-E2-#3	2.19	0.064	0.029	12
F1-T12-E2-#2	1.73	0.070	0.040	10
F1-T12-E2-#10	3.4	0.032	0.009	10
F1-T13-F1-#19	1.76	0.042	0.024	10
F1-T13-F1-#17	2.57	0.034	0.013	16
F1-T13-F1-#15	1.90	0.050	0.026	10
F1-T13-F1-#14	2.53	0.053	0.021	10

calculated from the results of constants in vector $\{x\}$ following Equation 3.12 to Equation 3.15.

Numerical Analysis

For the numerical solution, the annulus is divided into 16 layers, from $R = R_0$ (the soil-probe interface) to $R = 30 R_0$, where R represents the radius and R_0 represents the radius of the probe. The thickness of layers is smaller near the probe and become larger with the increase of radius. The exact position is determined by the following equation:

Figure 3.21 Illustration of annulus division for numerical analysis

$$R(i) = \frac{R_0}{N+1} (N+(i+1)^{2.5}) \quad (3.16)$$

where $i = 0$ to 15, when $i=16$, $R = 30 R_0$ and $N = 100$.

Figure 3.21 shows the division of the annulus which was used for the numerical analysis. The modulus is then calculated by combining Equation 3.1 with Equation 3.17. Figure 3.22 shows the modulus distribution calculated by Equation 3.1 and the approximation by dividing the annulus into 16 layers. Figure 3.23 shows the zoomed in version with radius ranging from R_0 to $6R_0$.

From the results of simple shear tests described before, the modulus ratio α ranges from 0.009 to 0.04. Actual analyses were carried with the values of α from 0.01 to 0.1. The value of Poisson's ratio used in the analysis is 0.499 instead of 0.5 to prevent numerical problems. The results are shown in Table 3.3. The disturbance index

Figure 3.22 Modulus distribution for $\alpha = 0.1$ and 0.01

Figure 3.23 Detailed illustration of Modulus distribution when $\alpha = 0.1$ and 0.01

calculated ranged from 0.48 to 0.54. In order to determine the average disturbance index I_p , the modulus distributions with α equal to 0.01 and 0.1 are shown in Figure 3.24 and Figure 3.25. It can be seen that the differences of modulus distribution are small. Based on this, the final value of I_p is determined by taking the average of all values from Table 3.3. The final result is 0.51.

Figure 3.24 Distribution of modulus in the annulus

Figure 3.25 Distribution of modulus for the disturbed annulus in zoomed in version

Table 3.3 Analysis results for disturbed annulus

α	I_p
0.01	0.546
0.02	0.538
0.03	0.530
0.04	0.523
0.05	0.516
0.06	0.508
0.07	0.502
0.08	0.495
0.09	0.488
0.10	0.482
average $I_p =$	0.512

3.4 CALCULATION OF SHEAR MODULUS FROM FDP TESTS

When the value of disturbance index is known, the calculation of the modulus is straight forward. With the definition of disturbance index:

$$G_{ap} = G (1 - I_p) \quad (3.17)$$

Therefore:

$$G = \frac{G_{ap}}{1 - I_p} \quad (3.18)$$

Another way to calculate the modulus is to modify the values of strain measurement to account for the disturbance effect. From Equation 3.13 and the definition of

disturbance index:

$$G_{ap} = G(1 - I_p) = \frac{P2 - P1}{2(\epsilon_{\theta,1}(2) - \epsilon_{\theta,1}(1))} \quad (3.19)$$

Dividing the equation by $(1 - I_p)$ leads to:

$$G = \frac{P2 - P1}{2(\epsilon_{\theta,1}(2)(1 - I_p) - \epsilon_{\theta,1}(1)(1 - I_p))} \quad (3.20)$$

Defining

$$\epsilon' = \epsilon_{\theta,1}(1 - I_p), \quad (3.21)$$

one obtains:

$$G = \frac{P2 - P1}{2(\epsilon'(2) - \epsilon'(1))} \quad (3.22)$$

The second method helps us to understand that the disturbance of the soil affects the shear modulus through the increase of strain.

3.5 FDP TEST RESULTS AND DISCUSSIONS

An extensive laboratory and in-situ testing program was carried out. In the program, five series of FDP tests were performed at three sites. These FDP tests are represented by pen1 and pen2 for the Sackville site, pen3 for the Diplomatic Hotel site (Fredericton), and nrc1, nrc2 for the National Research Council (NRC) site (Ottawa). Detailed

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information of the soil profiles is presented in chapter 5 and chapter 6.

Three types of shear moduli were obtained from the FDP tests. The modulus of the first type, represented by G_{new} , was calculated from the unload-reload slope, with modification of the disturbance index I_p . The modulus of the second type, represented by G_{end} , was calculated by the method proposed by Houlsby and Withers (1988). The third type of modulus, represented by G_0 , was calculated from the initial slope of the FDP test curve. (Note: In the graphs presented, G_{new} , G_{end} and G_0 may be shown as G_{new} , G_{end} and G_0 .) Results of the values of the three types of shear moduli are shown in Figure 3.26 to Figure 3.30.

The magnitudes of the three types of shear moduli were different. Values of the initial shear modulus were much smaller than those of the other two types of moduli. As the modulus was calculated from the initial loading slope, the low magnitude the initial shear modulus indicated that the soil was seriously disturbed before the inflation of the probe. As well, it was indicated that the surrounding soil reached to a plastic stage very in the loading process. For an ideal pressuremeter test, where the soil is not disturbed, the initial loading slope would be similar, or equal to the unloading-reloading slope. Hence, the unload-reload slope was used for estimation of the shear modulus, preventing the effects of the plastic behaviour in the initial loading.

Comparison of values of G_{new} and G_{end} varied from site to site and test to test. For the test pen1 of the Sackville site, values of G_{new} were higher than those of G_{end} . For test pen2 of the same site, values of G_{new} were lower than those of G_{end} . For test pen3 of the Diplomatic Hotel site, values of G_{new} were about twice the values of G_{end} . For tests at National Research Council, both tests of nrc1 and nrc2 showed good agreements between values of G_{new} and G_{end} .

Due to the variation of the magnitudes of the obtained shear moduli, the consistency and repeatability of these interpretation methods need to be examined.

Figure 3.26 Values of shear moduli measured from test #pen1

Figure 3.27 Values of shear moduli measured from test #pen2

Figure 3.28 Values of shear modulus measured from test #pen3

Figure 3.29 Values of shear modulus measured from test #nrc1

Figure 3.30 Values of shear modulus measured from test #nrc2

Repeatability of each method

For any in-situ tests to be useful, it is essential that these tests yield repeatable results. By "repeatable results", it means that any two tests performed in soils under the same conditions should yield the same results. Hence, on a site with relatively uniform soil stratigraphy, close values should be expected from two series of parallel tests. Only if repeatability is ensured, then the values from these tests are meaningful and can be compared or correlated to other in-situ test results. For FDP tests, the assurance of repeatability is even more important, because of the inevitable soil disturbance associated with the method of installation, as described in section 3.2.

The piezocone test is the best method for identification of soil stratigraphy (Meigh, 1978). It is one of the major applications of the piezocone test. Values of the pore

pressure, the tip resistance and the side friction measured from the piezocone test can be used to accurately identify tested soils.

Piezocone tests were conducted to examine the uniformity of soil stratigraphy and identify the soil layers. Results of three piezocone tests are shown in Figure 6.5 to 6.7 for the Sackville site. Low values of the pore water pressure and high values of the tip resistance indicated the existence of sand layers for soils under about 12 m deep. These sand layers were not uniformly distributed along the horizontal direction, as indicated by the tip resistance values. Soils of the top 12 m seemed to be relatively uniform with two exceptions. One is that test TM2 indicated existence of locally distributed thin sandy soil layers from 10 m to 12 m. The other one is that the overall results of the tip resistance showed slight variation from test TM2 to test TM5. The tip resistance from test TM5 were a little higher than that from test TM2.

Results of two piezocone tests conducted at the NRC site are shown in Figure 6.8 and Figure 6.9. These data indicated good agreement between the two tests, with the exception for the top crust layer ranging from top to about 5 m. Detailed data of soil profile information for all the three sites are presented in chapter 6.

Parallel tests were carried out at the sites of Sackville and the National Research Council. Results of the shear moduli were calculated by the proposed method and are shown in Figure 3.31 and Figure 3.32. For the Sackville site, from depth of 3 m to 13 m, results of shear moduli from the tests pen1 and pen2 agreed with each other very closely. For soils above 3 m and under 13 m, the values of shear modulus did not agree with each other, as indicated of the non uniformity by the results of the piezocone tests. For the National Research Council site, excellent agreement was also found between the results of the parallel tests nrc1 and nrc2.

Results of shear modulus G_{end} were calculated by the Houlsby and Withers' method and are shown in Figure 3.33 and Figure 3.34. Values of G_{end} from the tests pen1 and

pen2 at Sackville site did not give repeatable results (Figure 3.33). For tests at the National Research Council site, however, good agreement was found between the values of G_{end} from tests nrc1 and nrc2. As well, for the tests nrc1 and nrc2, the values of shear modulus G_{end} , which were calculated by the Houlsby and Withers' method, agreed with those of G_{new} , which were calculated by the proposed method. These comparisons are presented in Figure 3.29 and Figure 3.30.

These comparisons demonstrated that the proposed method for calculating the shear modulus G_{new} can yield repeatable results. The Houlsby and Withers' method, however, does not always give repeatable results of shear modulus. These comparisons were based on more than forty FDP test points on the Sackville site and about twenty FDP test points on the NRC site.

It was illustrated as well, that when the Houlsby and withers' method does give repeatable results, as for the tests of nrc1 and nrc2, comparable values of G_{end} and G_{new} can be obtained, in other words, these two methods can produce very close results. Hence, the Houlsby and Withers' method should be used as a complementary means to check the repeatability of a FDP test.

Once the consistency and the repeatability for the FDP test are achieved, the next question is whether this testing method can yield reliable values of shear modulus. In chapter 5 and chapter 6, comparisons are made between the FDP results and those of laboratory tests and other in-situ tests.

Results of G_0 provided another method for checking the repeatability of the disturbance or the repeatability of the parallel tests. Values of G_0 for the Sackville and National Research Council sites are shown in Figure 3.35 and Figure 3.36. Good agreements were found in the values of initial shear moduli for both sites.

In conclusion, repeatable results of shear modulus can be obtained from the FDP tests.

Figure 3.31 Results of shear modulus of test #pen1 & #pen2 from new method

Figure 3.32 Results of shear modulus of test #nrc1 & #nrc2 from new method

Figure 3.33 Results of modulus for #pen1 & #pen2 by Houlsby and Withers' method

Figure 3.34 Results of modulus for #nrc1 & #nrc2 by Houlsby and Withers' method

Figure 3.35 FDP test results of the initial moduli for tests pen1 & pen2

Figure 3.36 FDP test results of the initial moduli for tests nrc1 and nrc2

Importance of performing proper volume-pressure correction

It is discussed in Appendix A that proper volume and pressure correction is needed in order to obtain reliable results from FDP test data. It is also pointed out that the standard procedure proposed in the test manual for volume correction can lead to a negative unload-reload slope, which in turn leads to negative values of shear modulus. The results of FDP test data were analyzed using both the standard procedure and the modified procedure for volume correction. The results are shown in Figure 3.37 to 3.41. In the graphs, G_{ur} represents the shear modulus from unload-reload curve. G_{ur_old} represents the shear modulus value resulting from the old correction method while G_{ur_new} stands for shear modulus resulting from the new correction method.

By examining these results, it can be seen that for high values of shear modulus, the standard procedure leads to negative values of shear modulus. For shear modulus of low magnitude, the results are less affected by the volume correction method. This is the case for all the three sites where the tests were conducted. Especially for the results of test pen3, which was performed at the site of the Diplomatic Hotel, the shear moduli values obtained from depths between 5 m to 8 m fall in the range of negative 50 MPa. For the site of NRC where tests nrcl and nrc2 were performed, negative shear modulus were also obtained using the standard volume correction method for the top hard layer soil. Results for tests pen1 and pen2 were least affected by the volume correction method as the magnitudes of shear moduli were small compared with results from the other two sites.

The method proposed by Houlsby & Withers (1988) for calculating shear modulus from the end unloading slope was also used. Comparison results of the effects of the standard procedure and the modified procedure for volume correction are shown in Figure 3.42 to Figure 3.44. It can be seen that the standard method procedure leads to

either much higher values of shear modulus or negative shear modulus. The results of test pen3 and results of tests nrc1 and nrc2 are significantly affected by the volume correction method.

Figure 3.37 FDP test result of #pen1 at Sackville site

Figure 3.38 FDP test result of #pen2 at Sackville site

Figure 3.39 FDP test result of #pen3 at the Diplomatic Hotel site

Figure 3.40 FDP test result of #nrc1 at NRC site

Figure 3.41 FDP test result of #nrc2 at NRC site

Figure 3.42 FDP test result for correction methods comparison, #pen3

Figure 3.43 FDP test result for correction methods comparison, #nrc1

Figure 3.44 FDP test result for correction methods comparison, #nrc2

Chapter 4

INTERPRETATION OF UNDRAINED STRENGTH FROM FDP TEST

4.1 INTRODUCTION

Undrained shear strength has been determined primarily from undrained triaxial tests. With the development of in-situ testing methods, it is measured, as well by in-situ tests such as the field vane test, the piezocone test and the pressuremeter test. To illustrate the basic definition of undrained shear strength, the results of typical undrained triaxial compression tests on undisturbed and disturbed clay samples are shown in Figure 4.1. The results are shown as a plot of deviator stress versus axial strain.

Figure 4.1 Typical triaxial test result

For curve B of the undisturbed specimen, the deviator stress rises to a maximum,

then falls off as the strain is increased beyond the peak. This peak point corresponds to the so called "peak strength". Beyond this peak point, the deviator stress gradually reaches to a relatively steady value which can be referred to as the ultimate strength. For curve A of the disturbed specimen, the deviator stress rises to the ultimate strength point without displaying a peak point. Another practice in determining undrained strength from the triaxial test is by a criterion of strain magnitude, instead of peak value or the ultimate strength. The undrained strength obtained is determined by the choice of the failure criterion. Values of undrained strength from the pressuremeter tests are usually the ultimate strength.

In theoretical analysis, curves A and B (Figure 4.1) are usually represented by an idealized curve of perfect plastic behaviour, as shown in curve C.

It is shown in the literature (Mair and Wood 1987) that values of undrained strength from the self-boring pressuremeter are often much higher than those from undrained triaxial tests. One of the reasons for explaining this phenomena is that soil is disturbed by sampling. Another factor that should be considered is the disturbance effect from the pressuremeter. It has been shown that the soil disturbance of pressuremeter test leads to higher value of undrained strength (Baquelin et al., 1978; Law and Eden, 1980) instead of reducing the strength as in the case of laboratory tests.

For the FDP test, the disturbance is much more severe than the self-boring pressuremeter or the Menard pressuremeter test, as the FDP probe is forced into the soil without pre-drilling. Several methods were investigated here and compared with one another with the aim to find a method least affected by the disturbance so that reliable results can be obtained by this method.

In the following sections, available methods for estimating undrained strength from pressuremeter tests are briefly described first. The computer program used for the calculation with these methods is described and the results of undrained strength from

these methods for all FDP tests are presented. Magnitude and consistency of the resulting undrained strength from these methods are discussed. Closely examined are the effects of the installation method on the shape of the volume - pressure or strain - pressure relationship and hence the results of undrained strength. The best method for evaluating the undrained strength is recommended, based on these studies.

4.2 METHODS FOR CALCULATING UNDRAINED STRENGTH

4.2.1 Estimation from the slope of $P - \log_e \Delta V/V$ curve

Figure 4.2 Idealized curve of perfect plastic behaviour

From the analysis of the undrained expansion of a cylindrical cavity (Palmer, 1972; Ladanyi, 1972 and Baguelin et al., 1972), the shear stress - shear strain curve can be deduced from the measured pressure - cavity strain relationship. The exact expression for the shear stress, τ , at a cavity strain, ϵ_c is:

$$\tau = \frac{1}{2} \epsilon_c (1 + \epsilon_c) (2 + \epsilon_c) \frac{dp}{d\epsilon_c} \quad (4.1)$$

For small values of strain:

$$\tau = \epsilon_c \frac{dp}{d\epsilon_c} \quad (4.2)$$

Another form using change of volume instead of strain can be written as:

$$\tau = \frac{dp}{d[\log_e(\Delta V/V)]} \quad (4.3)$$

A typical result of FDP test data plotted as $p - \log_e(\Delta V/V)$ is shown in Figure 4.2. At large deformations, the curve tends to become linear, which can be interpreted as an indication that the surrounding soil is approaching a condition of perfect plasticity, with the shear stress remaining constant with continuing strain as shown in Figure 4.2.

With this method, the calculation is simple and straight forward. Only the curve of $p - \log_e(\Delta V/V)$ needs to be graphically drawn first, and the slope of the linear portion at large deformation will give the result of undrained shear strength.

4.2.2 Estimation from limit pressure

When a cylindrical cavity is expanded in elastic-perfectly plastic soil, the limit pressure is defined as the pressure when the pressuremeter cavity expands indefinitely. The limit pressure can be calculated by (Gibson and Anderson, 1961):

$$p_L = \sigma_{ho} + S_u [1 + \log_e(G/S_u)] \quad (4.4)$$

Given a value of p_L , the undrained strength can be estimated:

$$S_u = \frac{(p_L - \sigma_{ho})}{1 + \log_e(G/S_u)} \quad (4.5)$$

Let:

$$N_p = 1 + \log_e(G/S_u) \quad (4.6)$$

Then the undrained strength is calculated as:

$$S_u = (p_L - \sigma_{ho})/N_p \quad (4.7)$$

Figure 4.3 Relationship of pressuremeter constant and G/S_u

where the parameter N_p is called the "pressuremeter constant" (Marsland and Randolph, 1977). The variation of N_p with $\log(G/S_u)$ is shown in Figure 4.3, from which one can see that N_p is not very sensitive with the change of G/S_u . If one assumes an average value of $G/S_u = 90$, N_p is equal to 5.5. Variation of 10% of N_p or the strength S_u means the G/S_u varies from 60 to 156, a reasonable range for soft soils. More accurate value of N_p can be obtained by iterative calculations, while the shear modulus G is estimated using the method proposed in the last chapter.

The limit pressure, p_L , for which $\Delta V/V = 1$, cannot be attained in a real

Figure 4.4 Estimation of limit pressure

pressuremeter test. Estimation of p_L therefore requires extrapolation from data plotted as $p - \log_e(\Delta V/V)$. As mentioned previously, the curve of $p - \log_e(\Delta V/V)$ becomes linear at large deformations. The extrapolation of that linear portion of $p - \log_e(\Delta V/V)$ to the point where $\Delta V/V = 1$ gives the value of p_L . Figure 4.4 shows how the limit pressure can be estimated from the curve of $p - \log_e(\Delta V/V)$.

In Equation 4.7, the in-situ total horizontal stress σ_{ho} is used instead of the reference pressure p_o at the start of the pressuremeter test. The best available estimation of σ_{ho} should be used in the calculation. Especially in the FDP test, values of the measured reference pressure at the start of test are usually higher than the total in-situ horizontal stress.

The limit pressure can also be estimated by the volume - pressuremeter relationship proposed by Lemee (1973):

$$p = \frac{bV^2 + ac}{V^2 + a} \quad (4.8)$$

This equation could be used to represent the pressuremeter curve. It gives an

asymptote $p = b$ when the volume V goes to infinity. Three factors a , b , c need to be determined. With the original method proposed by Lemee, the factors a , b , c are obtained first by transforming the equation into a straight line. Then, a linear regression analysis is performed to find the parameters for the straight line. Factors a , b , c are obtained from the parameters of this straight line. In this study, the curve fitting method using the simplex algorithm introduced in Appendix A, was used to obtain the factors a , b , c . With this method, no transformation of the factors is necessary. Once the limit pressure is obtained with this method, the same procedure as above can be followed to calculate the undrained strength.

4.2.3 Method proposed by Houlsby and Withers

Figure 4.5 Method proposed by Houlsby and Withers, (1988)

Houlsby and Withers (1988) proposed a method for estimating undrained strength from the unloading curve as shown in Figure 4.5. The data are plotted as pressure p versus strain ϵ . The slope at point S on the unloading curve is used to calculate undrained

strength S_u . Segment XY can be expressed as:

$$XY = 2S_u \left(\frac{2+m}{3} \right) f = (\epsilon_e - \epsilon_s) \frac{dP}{d\epsilon} \quad (4.9)$$

Table 4.1 Values of the parameter f .

$(\epsilon_e - \epsilon_s)$	f
0.0	1.000
0.1	1.003
0.2	1.013
0.3	1.030
0.4	1.052
0.5	1.082

In the case of cylindrical cavity, $m=1$, therefore:

$$2S_u f = (\epsilon_e - \epsilon_s) \frac{dP}{d\epsilon} \quad (4.10)$$

where f is a function of $(m+1)(\epsilon_e - \epsilon_s)/2$ or $(\epsilon_e - \epsilon_s)$ as $m=1$. Its values are shown in Table 4.1. For all realistic values of $(\epsilon_e - \epsilon_s)$, f can be assumed to be 1. Let the slope at point S be $K_s = dP/d\epsilon$, then the undrained strength can be calculated as:

$$S_u = \frac{(\epsilon_e - \epsilon_s)}{2} K_s \quad (4.11)$$

4.2.4 Estimation from curve fitting method

An empirical method for evaluating soil parameters from pressuremeter test data is proposed by Arnold (1981). The pressure - strain relationship from a pressuremeter test is assumed to take the following form:

$$\sigma = Q + \frac{\epsilon}{a + b\epsilon} \quad (4.12)$$

Where a , b , and Q are constants (a , b are different with those from Equ. 4.8), which can be determined by a curve fitting method as described in Appendix A. The initial shear modulus and the undrained strength can be obtained through the constants a and b in the equation.

From the above equation:

$$\frac{d\sigma}{d\epsilon} = \frac{a}{(a+b\epsilon)^2} \quad (4.13)$$

The shear stress - strain relationship can be obtained as:

$$\tau = \epsilon \frac{d\sigma}{d\epsilon} = \frac{a\epsilon}{(a+b\epsilon)^2} \quad (4.14)$$

Then:

$$\frac{d\tau}{d\epsilon} = \frac{a(a-b\epsilon)}{(a+b\epsilon)^2} \quad (4.15)$$

For $\tau = \tau_{\max}$, $d\tau/d\epsilon = 0$. Since a is not zero, then $(a - b\epsilon) = 0$, from which $\epsilon = a/b$. Substituting into the expression for shear stress τ , we have:

$$S_u = \tau_{\max} = \frac{1}{4b} \quad (4.16)$$

The above equation is used to obtain the undrained shear strength.

4.3 COMPUTATION OF UNDRAINED STRENGTH

Traditionally the result of a pressuremeter test is drawn on paper and analyzed by a graphical method. This method is tedious and relatively time consuming, especially for processing a large amount of test results. A computer program was developed for processing the test data and calculating the undrained strength with the methods described

above. As sometimes the computer cannot correctly identify and process the test data, it is necessary to have graphical output on the screen for checking the reasonableness of the data. Also, good quality curve fitting is necessary for the data to be manipulated and analyzed by computer. As described in chapter 3, the simplex algorithm method was used for the curve fitting. A standard procedure or program function was built for this purpose.

The test data were divided into three parts. The first part was the loading curve, which starts from the beginning to the maximum loading pressure and did not include the unload-reload curve. The second part was the unload-reload data. The third part was the final unloading curve in the end of the test. The final unloading curve was used for the calculation of S_u by the method of Houlsby and Withers. The loading curve was used for the calculation of S_u by the rest of methods described before.

In the plotting of the loading curve as p vs $\text{Log}(\Delta V/V)$, the linear range needs to be identified in order to obtain the limit pressure and the slope of this linear part of the curve. It was found through the analysis that if the whole loading curve was divided evenly into three parts, the last part would always be in the linear range. Therefore, the data of this part were used for the calculation of the slope.

Figure 4.6 to Figure 4.13 show the graphical outputs during the calculation for soil at a depth of 5.5 m. Figure 4.6 shows the complete recorded data of the test. Then, the data set was divided into three parts as loading data, unload-reload data and final unloading data for analysis to be conducted later. Figure 4.7 shows the unload-reload data and the final unload data for examination of the correct division. The loading curve of the recorded data was smoothed by curve fitting and the outcome is shown in Figure 4.8. After pressure and volume corrections, the results are shown in Figure 4.9. The final unloading curve in the graph was used for the calculation of undrained strength with the method proposed by Houlsby and Withers.

Figure 4.6 Screen output of uncorrected FDP data

Figure 4.7 Screen output of unload-reload and the final unloading data

Figure 4.8 Recorded data after curve fitting

Figure 4.9 Screen output of FDP data after pressure & volume corrections

Figure 4.10 FDP test result at Sackville site (depth=5.5 m)

Figure 4.11 Curve fitting result from Lemee's method

Figure 4.12 Result of pressure - $\text{Log}(\Delta V/V)$ curve

Figure 4.13 Selection of data for extrapolation

Arnold's empirical method was applied to the data of the loading curve. The predicted curve as expressed by Equation 4.12 and the recorded curve are shown together in Figure 4.10.

Figure 4.11 shows the results of the analysis with the equation proposed by Lemee (1972). Figure 4.12 and Figure 4.13 shows the results of data plotted as P vs $\text{Log}_e(\Delta V/V)$. The linear range of this curve is shown in Figure 4.13. The undrained strength was calculated from the slope of this linear range. The limit P_L was also obtained by extrapolation of this straight line to $\Delta V/V = 1$.

4.4 RESULTS OF UNDRAINED STRENGTH AND DISCUSSIONS

As mentioned in chapter 3, five series of FDP tests were performed at three sites. They are represented by pen1 and pen2 at the Sackville site, pen3 at the Diplomatic Hotel site in Fredericton, and nrc1, nrc2 at the NRC site. Strength estimated by the method of Houlsby and Withers is represented as Su_{hw} . Su_{Lemee} represents the strength calculated from the equation proposed by Lemee. Su_{Pl} represents the strength estimated from the limit pressure obtained from plot of $P - \text{Log}_e(\Delta V/V)$. Su_{Ln_vp} represents strength estimated from the slope of linear portion of $P - \text{Log}_e(\Delta V/V)$ curve, and Su_B stands for the strength estimated by the curve fitting method. (In the graphs, these values may be represented as Su_{hw} , Su_{Lemee} , Su_{Pl} , Su_{Ln_vp} and Su_B .)

4.4.1 Magnitude Comparison

All the methods described were used to estimate the undrained strength. The results are shown in Figure 4.14 to Figure 4.18.

Figure 4.14 Results of FDP test pen1

Figure 4.15 Results of FDP test pen2

Undrained Strength from the Diplomatic Hotel Site (kPa)

Figure 4.16 Results of FDP test pen3

Figure 4.17 Results of FDP test nrc1

Figure 4.18 Results of FDP test nrc2

As shown in the results, these five methods did not give the same magnitude of undrained strength for each test. Su_B yielded the highest results among all five methods. Especially for results of pen2, values of Su_B were so high that they could not compare with the rest methods at all. Su_{Ln_vp} yielded the second largest values of strength. Its values were more close to the results of Su_B than to the other three methods, as shown in the test of pen1, nrc1, nrc2 (Figure 4.14, Figure 4.17, Figure 4.18). In comparison, the values of the undrained strength from Su_{Pt} and Su_{Lemcc} were much smaller than those of Su_{Ln_vp} . It can be seen from all the test results that values of Su_{Ln_vp} were almost twice as those of Su_{Pt} and Su_{Lemcc} . For tests pen1, pen2 and pen3, Houlsby and Withers' method gives very close results as those from Su_{Pt} and Su_{Lemcc} . But the results of Su_{hw} were scattered for test nrc1 and nrc2.

In summary, the curve fitting method may lead to unreasonable results of Su_B . Out of these tests, the Houlsby and Withers' method yielded scattered results for two tests nrc1 and nrc2. The limit pressure method and the slope of $P - \log_e(\Delta V/V)$ method led to stable results for all five tests. Very close values of undrained strength were obtained from the limit pressure method and Houlsby & Withers' method. The curve fitting method gave values close to those from the slope of the $P - \log_e(\Delta V/V)$ method. The values of Su_{Ln_vp} and Su_B were twice as those obtained from the results of Su_{Pt} , Su_{Lemcc} and Su_{hw} .

4.4.2 Repeatability Comparison

As discussed in chapter 3, the ability of yielding repeatable results for a uniform site is essential for any method of in-situ tests. Among the five FDP tests, test pen1 and test pen2 were conducted on the same site of Sackville. Tests nrc1 and nrc2 were conducted

on the same site at the National Research Council. Results of the corrected bearing resistance and the pore pressure from the piezocone tests are shown in Figure 4.19. The location of piezocone test TM2 was close to FDP test pen1 and the location of piezocone test TM5 was close to FDP test pen2. They were about 30 m apart. These piezocone results indicated that the subsoil for the site was not perfectly uniform. The bearing resistances of TM5 were slightly higher than those from TM2. For the NRC site, as the two FDP tests were conducted within two meters range, the variation was expected to be small.

For each interpretation method, values of undrained strength from parallel tests at both sites are illustrated together in one figure as shown in Figure 4.20 to Figure 4.24.

The results interpreted from the limit pressure method are shown in Figure 4.20. Close results were obtained from test pen1 and pen2. There was a small difference between these two tests, which was in agreement with the trend as indicated by the results of piezocone tests TM2 and TM5. For the NRC site, undrained strength from nrc1 and nrc2 were in good agreement. Figure 4.21 shows the results of undrained strength calculated from Lemee's method. The pattern was very similar to those obtained from the limit pressure method as shown in Figure 4.20.

Figure 4.22 shows the results of undrained strength interpreted from Houlsby and Withers' method. For the Sackville site, close values were obtained from the two parallel tests pen1 and pen2. However, for depth below 15 m, the piezocone data indicated a hard layer of sandy soil, which was not reflected from the resulted strength profile obtained from Houlsby and Withers' method. For the NRC site, data of undrained strength from Houlsby and Withers' method did not give to stable and repeatable results at all.

Results of undrained strength calculated from the slope of $P - \log_e(\Delta V/V)$ curve are shown in Figure 4.23. For the Sackville site, the difference of strength values between

tests pen1 and pen2 seemed to be too large compared to the results of piezocone tests. Values of strength from pen2 were about twice as those from test pen1. Excellent repeatability was found in the results of nrc1 and nrc2 from NRC site.

For the curve fitting method, test results from Sackville site did not show repeatability at all. For NRC site very close results were obtained from the two parallel tests, as shown in Figure 4.24.

The above comparisons demonstrated that the "limit pressure method" and the Lemee's method can lead to consistent and repeatable results of undrained strength. The rest of the methods can sometimes give comparable results, but are not stable or consistent for different sites. The "limit pressure method" and Lemee's method give very close values of undrained strength. However, the amount of calculations needed for the "limit pressure method" is less than that for Lemee's method.

4.4.3 Discussions

The above test results indicated that the limit pressure method was superior to the other methods. Consistent and repeatable results of undrained strength were obtained for all the tests at the three sites. Possible reasons are presented in the following discussions.

The values of the undrained strength from the limit pressure method were on the lower bound of the results from the rest methods, which signified less disturbance effects. Studies showed that for pressuremeter tests, the soil disturbance led to higher values of undrained strength (Baquelin et al., 1978). As mentioned earlier, the values of Su_{La_vp} and Su_B were about twice those of Su_{PI} , Su_{Lemee} and Su_{hw} . Values of Su_{PI} were on the lower bound, which indicated being less effected by the soil disturbance.

As well, it was shown by Baquelin et al. (1978) that the limit pressure P_L

Figure 4.19 Results of piezocone tests at Sackville site

Figure 4.20 Undrained strength obtained from limit pressure method

Figure 4.21 Undrained strength obtained from Lemee's method

Figure 4.22 Undrained strength obtained from Houlsby and Withers' method

Undrained Strength Obtained from Slope of $p - \log (\Delta V/V)$ Curve (kPa)

Figure 4.23 Undrained strength obtained from slope of $p - \log_e(\Delta V/V)$ curve

Figure 4.24 Undrained strength obtained from curve fitting method

was not sensitive to soil disturbance. Therefore, more reliable values of undrained strength should be obtained from the limit pressure method. On the other hand, the values of $Su_{L_n_{vp}}$ and Su_B depended on the overall loading curve, which were more sensitive to the soil disturbance.

Another evidence supporting the limit pressure method can be obtained from the comparison of results from the limit pressure method to the results from the Houlsby and Withers' method. Different parts of the test curve were used from these two methods. Values of Su_{hw} were obtained from the slope of the final unloading curves, while values of Su_{pl} were calculated from the slope of $P - \log_e(\Delta V/V)$ curve in the final loading stage. Yet, when repeatable results of undrained strength were obtained from the method of Houlsby and Withers, as shown in Figure 4.22 for test pen1 and pen2, good agreement was found as well, in the values of Su_{hw} and Su_{pl} , as shown in Figure 4.14 and Figure 4.15.

Based on the test results and above discussions, it may be concluded that the limit pressure method can lead to repeatable results and is better than the other methods. The Houlsby and Withers' method may be used as a complement for the limit pressure method, providing a double checking for the results.

4.5 EFFECTS OF THE INSTALLATION ON THE UNDRAINED STRENGTH

As described in Chapter 3, because of the method of the probe installation, the recorded test data represents only the second phase of the pressuremeter test as shown in Figure 3.1. Therefore, the exact initial state, or reference point for the idealized pressuremeter test is not known.

The effects of the unclear initial state on the values of undrained strength are examined here.

Figure 4.25 Assumed pressure - strain relationships for FDP test

From the examination of FDP installation in chapter 4, the testing was considered to start from an equivalent diameter. Also, for an ideal test, the measured initial horizontal pressure should be equal to the in-situ horizontal total stress. Based on these two aspects, the possible strain - pressure relationships can be assumed as shown in Figure 4.25. The differences of these possible curves come from different assumptions of initial slopes. The first assumption is that the initial slope equals the unload-reload slope, which is shown as line A. The assumption of the lowest initial slope is that the initial slope equals the initial slope of measured data, which is shown as line D. Between line A and line D, two other assumptions can be made as shown by line B and C. Line B represents the extrapolation of the recorded data. Line C assumes that the initial slope is 0.55 of the unload-reload slope. This assumption comes from the results of SBP tests. The initial slopes of self-boring piezometer test results were calculated by curve fitting and the average ratio of initial slope to the unload-reload slope was found to be 0.55.

The detailed results from self-boring pressuremeter tests are shown in Table 4.2, where G_{ur} stands for the shear modulus from unload-reload curve. These assumptions will be referred as assumption "A", assumption "B", assumption "C" and assumption "D".

Table 4.2 Results of moduli from SBP tests

Test#	Depth	G_0	G_{ur}	G_0/G_{ur}
	(m)	(MPa)	(MPa)	
s89t6	2.4	1.4	1.57	0.89
s89t2	3.9	1.2	2.04	0.59
s89t7	3.9	1.05	2.04	0.52
s89t3	5.4	1.26	2.81	0.45
s89t8	5.4	1.73	2.81	0.61
s89t4	6.9	1.4	3.87	0.36
s89t5	12.1	4.53	9.86	0.46
			Average ratio=	0.55

If a reasonable K_0 value can be assumed, the total horizontal pressure can be calculated. Together with the assumptions of initial slope, the best curve can be obtained with a numerical curve fitting method. The equation used for describing the test curve is assumed the same as Equation 4.12, which is given again here:

$$\sigma = Q + \frac{e}{a + be} \quad (4.17)$$

The ratio of probe radius R_0 and the equivalent radius R_e is defined by K_R , where

$$K_R = \frac{R_0}{R_e} \quad (4.18)$$

Here we have unknowns a , b , and K_R . As the value of " a " reflects the initial slope whose value can be assumed in terms of the unload-reload slope or the initial slope of

Figure 4.26 FDP test result of pen1 based on assumption C

Figure 4.27 FDP test result of pen1 based on assumption D

Figure 4.28 FDP test result of pen1 based on assumption A

Figure 4.29 FDP test result of pen1 based on assumption B

Figure 4.30 FDP test result of pen3 based on assumption C

Figure 4.31 FDP test result of pen3 based on assumption D

Figure 4.32 FDP test result of pen3 based on assumption A

Figure 4.33 FDP test result of pen3 based on assumption B

Figure 4.34 FDP test result of nrc1 based on assumption C

Figure 4.35 FDP test result of nrc1 based on assumption D

Figure 4.36 FDP test result of nrc1 based on assumption A

Figure 4.37 FDP test result of nrc1 based on assumption B

the recorded data, there are only two independent parameters left: "b" and "K_R". Again the simplex algorithm method was used for the numerical curve fittings. The data of all the tests were analyzed by the curve fitting method with assumptions mentioned above. For the three test sites, the results of one test for each site are presented here and for each test, four graphs are given, each corresponding to a different assumption. For the Sackville site, the results of curve fitting for tests at 6 m are shown from Figure 4.26 to Figure 4.29. For the Diplomatic Hotel site, the results for the test at 10 m are shown from Figure 4.30 to Figure 4.33. Figure 4.34 to Figure 4.37 show the results the of test at 7 m at the site of National Research Council. In all the graphs the transition point of the installation phase and the testing phase is divided by a dotted vertical line.

With the test data without modification, a total of five volume - pressure curves can be obtained from one FDP test. From the results presented in section 4.4, it was shown that consistent and repeatable results of undrained strength can be obtained from Lemee's method and the limit pressure method. The method proposed by Houlsby and Withers also yielded comparable values of undrained strength with those from the limit pressure method for Sackville site. Therefore, the three methods corresponding to Su_{Lemee} , Su_{Pl} and Su_{hw} were used for estimating undrained strength.

Results of undrained strength calculated with the method proposed by Houlsby and Withers showed that the four assumptions do not affect the results of Su_{hw} . The reason is that Su_{hw} was calculated from the unloading curve, instead of the loading curve, which is not influenced by the variation of the initial volume resulting from the four assumptions.

Results of Su_{Lemee} and Su_{Pl} calculated from five volume - pressure curves as mentioned before, are presented in Figure 4.38 to Figure 4.47. These results included tests of all the five boreholes conducted at three sites. It was clearly shown that the assumptions made regarding the initial slopes affect the results of Su_{Lemee} . Different

Figure 4.38 FDP test result based on different assumptions (pen1)

Figure 4.39 FDP test result based on different assumptions (pen1)

Figure 4.40 FDP test result based on different assumptions (pen2)

Figure 4.41 FDP test result based on different assumptions (pen2)

Figure 4.42 FDP test result based on different assumptions (pen3)

Figure 4.43 FDP test result based on different assumptions (pen3)

Figure 4.44 FDP test result based on different assumptions (nrc1)

Figure 4.45 FDP test result based on different assumptions (nrc1)

Figure 4.46 FDP test result based on different assumptions (nrc2)

Figure 4.47 FDP test result based on different assumptions (nrc2)

assumptions led to different values of $Su_{L_{mcc}}$. This was especially evident in the case of test pen3 as shown in Figure 4.42.

Results of Su_{pl} , however, indicates that the method of limit pressure from the plot of pressure - $\log_e(\Delta V/V)$ led to very close values of undrained strength with all the possible volume - pressure relationships. Values of Su_{pl} from the unmodified volume pressure curve represent the lower bound of strength obtained with this method, which falls on the safe side.

Figure 4.48 Example for estimating undrained strength by limit pressure method

The results of Su_{pl} from all the FDP tests showed that within all the possible ranges of curves representing the installation phase, close values of limit pressure could be obtained. This phenomena can be illustrated more clearly if all the five sets of data are plotted as pressure - $\log_e(\Delta V/V)$ in one graph. One example is given in Figure 4.48 for the test at 10 meter of pen1. The extrapolations of the linear segments from all the curves led to the same point for $\log_e(\Delta V/V) = 0$ or $(\Delta V/V) = 1$, which is the definition of limit

pressure.

The above results were obtained with the value of K_0 assumed as 0.6. As the results of self-boring pressuremeter tests indicate, the values of K_0 ranged from 0.6 to 0.8. The results of the FDP tests were analyzed again with the values of K_0 assumed as 0.7 and 0.8. The results show that the variation of K_0 has very little effect on the undrained strength obtained through the limit pressure method. Only part of the results are presented here. Figure 4.49 to Figure 4.52 show the resulted undrained strength from curves obtained from assumption "C" for pen1, pen2, nrc1 and nrc2. Figure 4.53 shows the results from curves obtained from assumption "D" for pen3.

In conclusion, when the limit pressure method was used, the effects of the unclear initial state on the undrained strength were insignificant. Results of undrained strength from all the other methods were affected.

Figure 4.49 FDP test result when $K_0 = 0.6, 0.7, 0.8$ for test pen1

Figure 4.50 FDP test result when K₀ = 0.6, 0.7, 0.8 for test pen2

Figure 4.51 FDP test result when K₀ = 0.6, 0.7, 0.8 for test nrc1

Figure 4.52 FDP test result when $K_0 = 0.6, 0.7, 0.8$ for test nrc2

Figure 4.53 FDP test result when $K_0 = 0.6, 0.7, 0.8$ for test pen3

Chapter 5

COMPARISON OF FDP TESTS WITH LABORATORY TESTS

5.1 INTRODUCTION

Laboratory tests have been widely used in studies of soil behaviour since the beginning of soil mechanics. The reason is that laboratory tests have the following advantages:

1. Well defined boundary conditions
2. Controlled drainage conditions
3. Preselected and well defined stress paths
4. (For small strain levels) uniform strain fields
5. Identification of the soil nature and physical features

The one dimensional consolidation test or oedometer test is one of the earliest soil tests and the most often used laboratory test for clay soils if the compressibility needs to be evaluated. It simulates the situation of soil under one dimensional loading and the resulting consolidation. Parameters obtained from oedometer tests are used for the one dimensional settlement calculation and in some cases used as parameters for the finite element analysis. The one dimensional method is still considered a reliable method for clays which are not sensitive to sampling disturbances (Burland, 1971). For some soils such as silty clays, sampling will cause severe disturbance. When the oedometer tests are performed on this kind of soil, the deduced compressibility will be too high and settlement will be overestimated.

While the oedometer test is used for one dimensional soil testing, the triaxial test

is used for testing under three dimensional stress conditions. Though strictly speaking, triaxial tests provide radically symmetric stress and strain conditions, these are considered to be satisfactory for many actual problems. With undrained triaxial tests on clays, undrained shear strength as well as undrained Young's modulus can be obtained.

The simple shear test device has been used as well, for measuring shear strength and shear modulus. The test is carried out by increasing the horizontal shear stress while varying the vertical stress to maintain constant volume.

A series of laboratory tests including oedometer tests, triaxial tests and modified simple shear tests was performed to evaluate the compressibility and shear strength behaviour. These results were used to compare with the compressibility and strength parameters obtained from the full displacement pressuremeter tests. Comparisons of FDP tests to results of other in-situ tests are presented in the next chapter.

5.2 SOIL PROFILE

Soil samples were obtained from a test embankment site. It is situated between Sackville and Aulac in the Province of New Brunswick, Canada. The location is illustrated in the map as shown in Figure 5.1 (Rowe et al., 1991). The soil stratigraphy (Figure 5.2) at the site consists of a random fill top soil and followed by a clay crust reaching to about 1.2 m deep. This is underlain by a silty clay down to 20 m. From the top to the bottom of the silty clay, the colour of the soil varies from brown, brown to black, reddish, and light grey. Depths between 1.2 m and 3.2 m and between 4.3 m and 10 m, the soil contains organic materials and straws. There are sand lenses in the soil from 3.2m to 4.2m. Below 10 m, the soil becomes a stiff silty clay. The data of natural water content, plastic limit, liquid limit and organic content is shown in Figure 5.3. (Rowe et al., 1991).

Figure 5.1 Location of the testing embankment (Rowe et al., 1991)

Figure 5.2 Soil profile

5.3 SPECIMEN PREPARATION AND TESTING METHODS

The soil investigation was conducted in August, 1989. Both tube samples and large diameter samples were obtained from the site in Sackville. The diameter for the large sample was 20 cm. For tube samples the diameter was about 7.5 cm. All the samples were covered and sealed with wax immediately after they were taken. The samples were transported by car from the site to the Geotechnical Laboratory of National Research

Figure 5.3 Soil properties of Sackville site

Council in Ottawa. Thick foam was placed under the samples to reduce possible disturbance during transportation. The samples were stored in the humid storage room at the National Research Council laboratory. The disturbance caused by sampling could be substantially reduced for specimens cut from the large diameter samples.

Figure 5.4 Consolidation specimen container

5.3.1 Oedometer test

The standard apparatus was used for the consolidation tests. Loads were applied with a mechanical lever system by dead weights. The consolidation specimen container was the floating-ring style, which is shown in Figure 5.4. The area of the test ring was 20.08 cm^2 with a height of 2.01 cm. A trimmer, or cylindrical cutter was used for trimming the sample down to the inside diameter of the consolidometer ring. Except for a few loads in the beginning, all the loads were applied for 24 hours or longer. Also, the time - displacement data was recorded to ensure 100% consolidation for every load. One example of time - displacement data is shown in Figure 5.5. A total of six tests were performed. Detailed information is shown in Table 5.1.

Figure 5.5 Time - displacement data for consolidation test

Figure 5.6 Triaxial test device

Table 5.1 Oedometer specimen number and depth

Sample Number	Depth (m)
249-1-2	0.6
249-3-3	2.0
249-6-1	4.0
249-9-1	6.3
248-12-1	11.5
248-14-2	16.0

5.3.2 Triaxial Test

The apparatus used for triaxial test meets the standard of ASTM. The schematic diagram is shown in Figure 5.6. The average height of the specimens was about 14.5 cm and the average diameter was about 7.25 cm. The soil specimen was K_0 consolidated under the in-situ initial stress states for 24 hours before it was tested. The back pressure used was 200 kPa. The values of pore pressure parameter B at the end of consolidations were all above 97%. Loading was strain rate controlled and the strain rate used was 2% per hour. All data were read electronically. A computer data acquisition system was used to obtain the data and the data was converted to IBM DOS data files afterwards. Six tests were performed and information of the specimens is shown in Table 5.2.

5.3.3 Simple Shear Test

For the simple shear tests, the automatic control system described was used. This system made it possible to apply pure shear to a soil specimen.

Table 5.2 Triaxial spencimen number and depth

Sample Number	Depth (m)
249-1-1	0.6
249-3-4	2.0
249-6-2	4.0
249-9-2	6.3
248-12-2	11.5
248-14-1	16.0

The shape of the specimen used was similiar to one for a consolidation test. The diameter was about 7 cm and the height was about 2 cm. The specimen was trimmed in a way as would for a oedometer test specimen. The specimens were all consolidated under isotropy pressure for 24 hours before being tested. Very fast loading rate was used for the testing process. Normally, the whole loading process lasts about one hour before the specimen is sheared to failure.

5.4 LABORATORY TEST RESULTS

5.4.1 Oedometer test results

Six oedometer tests were conducted. The results were plotted as Log(pressure) versus Void Ratio and are shown from Figure 5.7 to Figure 5.12. For the first four tests unloading processes were performed. For the last two tests for soil at depths of 11.5 m and 16 m no unloading was performed. The curves were analyzed using the method of Schmertmann (1955). The initial in-situ stress σ_o was calculated first. The Maximum preconsolidation pressure σ_p was determined by Casagrande's (1936) method. Then the in-situ initial compression curve can be obtained. An example for soil at 4 m is shown in Figure 5.13. With the in-situ initial stress σ_o and the maximum precompression pressure σ_p , the over consolidation ratio OCR can be calculated as:

Figure 5.7 Consolidation test result for #249-1-2

Figure 5.8 Consolidation test result for #249-3-3

Figure 5.9 Consolidation test result for #249-6-1

Figure 5.10 Consolidation test result for #249-9-1

Figure 5.11 Consolidation test result for #248-12-1

Figure 5.12 Consolidation test result for #248-14-2

Consolidation Test

#249-6-1 D=4 m

Figure 5.13 An example of oedometer test results

$$OCR = \frac{\sigma_p}{\sigma_o} \quad (5.1)$$

The compression index C_c can be obtained as:

$$C_c = - \frac{de}{d(\log_{10} p)} \quad (5.2)$$

With the compression index and initial void ratio, the constrained modulus M is calculated from:

$$M = \frac{(1+e_o)\sigma_{va}}{0.435C_c} \quad (5.3)$$

in which σ_{va} is the average pressure at which M is calculated. The results of the oedometer tests were summarized in Table 5.3. The results of compression index and constrained modulus are also shown in Figure 5.14 and Figure 5.15.

Table 5.3 Summary of oedometer tests

Sample#	Depth (m)	e_o	σ_o (kPa)	σ_p (kPa)	OCR	C_c	M (MPa)
249-1-2	0.6	0.94	5.1	254.0	54.0	0.30	4.14
249-3-3	2.0	1.23	17.0	60.3	3.6	0.43	1.03
249-6-1	4.0	1.67	34.0	45.7	1.3	0.67	0.78
249-9-1	6.3	1.15	54.0	54.0	1.0	0.39	1.00
248-12-1	11.5	0.89	98.0	97.8	1.0	0.21	2.54
248-14-2	16.0	1.04	133.0	136.0	1.0	0.37	2.05

Figure 5.14 Results of compression index from oedometer tests

Figure 5.15 Results of constrained modulus from oedometer tests

5.4.2 Triaxial test results

Six triaxial tests were performed with specimens from tube samples. The results of axial strain versus shear stress are shown from Figure 5.16 to Figure 5.21. For test number 249-3-4 and 249-6-2, the data show that in the initial stage of loading, about 1% of axial strain occurred under very small load. This happened most likely as a result of loose contact between the specimen cap and the steel rod which transfers load from the load cell. The loose contact could be formed during time of consolidation. After this the shear stress - strain relationship developed normally.

Shear strengths were obtained from these shear stress - strain curves. The maximum axial strain of 15% was used as failure criterion for evaluating undrained strength. In the calculation, the effect of rubber membrane was considered, as well. The following equation was recommended in the ASTM standard for calculating the effect of rubber membrane:

$$\Delta \frac{(\sigma_1 - \sigma_3)}{2} = \frac{2E_m t_m \epsilon_1}{D_c} \quad (5.4)$$

where:

$\Delta(\sigma_1 - \sigma_3)/2$ = the correction of shear stress (kPa),

D_c : = diameter of specimen after consolidation (mm),

E_m : = Young's modulus for the membrane material (kPa),

t_m : = thickness of the membrane (mm), and

ϵ_1 : = axial strain.

For the average values of $D_c = 35.4$ mm, $E_m = 1400$ kPa, $t_m = 0.5$ mm and $\epsilon_1 = .15$, the value for correction was found to be 6 kPa. This value was used only as a reference for determining the shear strength. With closer examination of the shear stress - strain curves as shown in Figure 5.16 to Figure 5.21, it can be seen that after reaching the point

Figure 5.16 Triaxial test result for #249-1-1

Figure 5.17 Triaxial test result for #249-3-4

Figure 5.18 Triaxial test result for #249-6-2

Figure 5.19 Triaxial test result for #249-9-2

Figure 5.20 Triaxial test result for #248-12-2

Figure 5.21 Triaxial test result for #248-14-1

which seemed to be a peak value, the shear stress went up again and reaches the highest value. This pattern of shear stress increase was considered an indication of the rubber membrane deformation, instead of the stress hardening. The values of the axial strains for the initial peaks were high, which were around 6% to 8%. As well, the differences between the final maximum values and the first peak values ranged from 5 to 10 kPa, which compared well with the calculated reference correction value for the membrane. Hence, the value of the first peak point was taken as the undrained shear strength. The results are presented in Table 5.4.

Table 5.4 Results of undrained shear strength from triaxial tests

Sample Number	Depth (m)	Strength (kPa)
249-1-1	0.6	27.0
249-3-4	2.0	19.0
249-6-2	4.0	20.0
249-9-2	6.3	22.0
248-12-2	11.5	35.0
248-14-1	16.0	56.0

Hook's Law is used to calculate Young's modulus:

$$\Delta e_x = \frac{1}{E} [\Delta \sigma_x - \mu (\Delta \sigma_y + \Delta \sigma_z)] \quad (5.5)$$

$$\Delta e_y = \frac{1}{E} [\Delta \sigma_y - \mu (\Delta \sigma_x + \Delta \sigma_z)] \quad (5.6)$$

$$\Delta e_z = \frac{1}{E} [\Delta \sigma_z - \mu (\Delta \sigma_x + \Delta \sigma_y)] \quad (5.7)$$

For an undrained triaxial test, the increases of total stress in horizontal directions are zero. Then, from Equation 5.7, the undrained Young's modulus is calculated as:

$$E_u = \frac{\Delta \sigma_z}{\Delta \epsilon_z} \quad (5.8)$$

The slopes for modulus calculation are shown in Figure 5.16 to Figure 5.21. The slope is constructed as from the origin to a point where its value of shear stress is about half of the magnitude of shear strength. For specimen 249-3-4 and 249-6-2, the starting point for strain used is 1%.

Young's modulus for drained condition can be estimated from the measured values of pore water pressure during the test. The pore pressure parameter A for undrained triaxial test can be written as:

$$A = \frac{\Delta u}{\Delta \sigma_z} \quad (5.9)$$

In terms of effective stresses, the axial strain can be expressed as:

$$\epsilon_z = \frac{1}{E'} [\sigma'_z - 2\mu' \sigma'_x] \quad (5.10)$$

where E' is the Young's modulus for drained condition, σ'_z is the effective stress in vertical direction and σ'_x is the effective stress in the horizontal direction, and μ' is the effective Poisson's ratio. The effective stress σ'_z and σ'_x can be expressed in terms of A as:

$$\Delta \sigma'_z = (1-A) \Delta \sigma_z \quad (5.11)$$

$$\Delta \sigma'_x = -A \Delta \sigma_z \quad (5.12)$$

Substituting Equations 5.11 and 5.12 into Equation 5.10, we have:

$$\Delta \epsilon_z = \frac{1}{E'}(1-A+2A\mu')\Delta \sigma_z \quad (5.13)$$

Therefore:

$$E = \frac{\Delta \sigma_z}{\Delta \epsilon_z} = \frac{E'}{(1-A+2A\mu')} \quad (5.14)$$

Hence:

$$E' = E(1-A+2A\mu') \quad (5.15)$$

The Young's modulus for drained condition were calculated with measured A values and also, with a value assumed as 1/3. The Poisson's ratio for the drained condition was assumed to be 0.3. Results of drained Young's modulus, together with the undrained Young's modulus, are presented in Table 5.5.

Table 5.5 Results of undrained and drained Young's modulus of triaxial tests

Test#	Depth	ϵ_u	$E'(A=1/3)$	$E'(\text{measured } A)$
	(m)	(MPa)	(MPa)	(MPa)
249-1-1	0.6	0.82	0.73	0.70
249-3-4	2.0	1.82	1.55	1.58
249-6-2	4.0	1.75	1.43	1.52
249-9-2	6.3	4.18	3.29	3.64
248-12-2	11.5	3.07	2.18	2.67
248-14-1	16.	6.85	5.56	5.96

The results of undrained strength, undrained Young's modulus and drained Young's modulus are shown in Figure 5.22 to Figure 5.24, as well.

Figure 5.22 Triaxial test results of undrained strength

Figure 5.23 Triaxial test results of undrained Young's modulus

Figure 5.24 Triaxial test results of drained Young's modulus

Figure 5.25 Comparison of stress paths for triaxial tests

A consolidated drained triaxial test was performed on soil from the same borehole as the undrained test specimens. The depth of the specimen is 6.6 m. The effective stress path of this drained test is different from that of the undrained triaxial test. A comparison between the two effective stress paths for comparable depth is shown in Figure 5.25. For a drained test, pore pressure in the specimen should be small enough to be neglected. Therefore, the stress path should followed a straight line.

Drained Young's modulus and Poisson's ratio can be obtained from a drained triaxial test. For a drained triaxial test, Equation 5.5 to 5.7 can be written as:

$$E'\Delta e_x = \Delta \sigma'_x - \mu'(\Delta \sigma'_x + \Delta \sigma'_z) \quad (5.16)$$

$$E'\Delta e_z = \Delta \sigma'_z - 2\mu'\Delta \sigma'_x \quad (5.17)$$

Solving these two equations one obtains:

$$E' = \frac{\Delta \sigma'_x \Delta \sigma'_z + (\Delta \sigma'_z)^2 - 2(\Delta \sigma'_x)^2}{(\Delta \sigma'_x + \Delta \sigma'_z)\Delta e_z - 2\Delta \sigma'_x \Delta e_x} \quad (5.18)$$

$$\mu' = \frac{\Delta \sigma'_x \Delta e_z - \Delta \sigma'_z \Delta e_x}{\Delta \sigma'_x (\Delta e_z - 2\Delta e_x) + \Delta \sigma'_z \Delta e_z} \quad (5.19)$$

For the drained test (Depth = 6.6 m) performed, the calculated drained Young's modulus was 3.59 MPa and the Poisson's ratio was 0.264. When the Poisson's ratio was assumed to be 0.3, the drained Young's modulus from the undrained test (Depth = 6.3 m) was 3.29 MPa (with measured A) and 3.64 MPa (with A=1/3). This indicates that the drained Young's modulus from the undrained test agrees with that from the drained test. As well, the assumption of Poisson's ratio for the drained condition was reasonable.

In order to compare the compressibility results from triaxial tests with that of oedometer tests, the constrained modulus was evaluated from the drained Young's modulus. For a consolidation test, the horizontal strain $\Delta\epsilon_x = \Delta\epsilon_y = 0$. From Equation 5.5 and Equation 5.6, we have:

$$\Delta\sigma'_x = \Delta\sigma'_y = \frac{\mu'}{1-\mu'} \Delta\sigma'_z \quad (5.20)$$

Substitute them back to Equation 5.7 one obtains:

$$\epsilon_z = \frac{1}{E'} [\Delta\sigma'_z - 2\mu' \frac{\mu}{1-\mu'} \Delta\sigma'_z] \quad (5.21)$$

Therefore, the constrained modulus can be calculated as:

$$M = \frac{\Delta\sigma'_z}{\Delta\epsilon_z} = \frac{E'(1-\mu')}{(1+\mu')(1-2\mu')} \quad (5.22)$$

With the value of Poisson's ratio assumed to be 0.3:

$$M = 1.35E' \quad (5.23)$$

The shear modulus was obtained from the undrained tests by converting the values undrained Young's modulus. They were used for comparison to the results from the full pressuremeter tests. The following equation is used for obtaining G:

$$E_u = 2(1+\mu)G \quad \text{or} \quad G = \frac{E_u}{2(1+\mu)} \quad (5.24)$$

The Poisson's ratio for the undrained condition is 0.5.

The results of constrained modulus and shear modulus converted from undrained moduli are shown in Table 5.6 and as well, in Figure 5.26 and Figure 5.27. The values of these shear moduli were used for comparison with those from the FDP tests.

Figure 5.26 Constrained modulus converted from triaxial results

Figure 5.27 Shear modulus from triaxial test result

Table 5.6 Constrained and shear moduli converted from undrained Young's modulus

Test#	Depth	M	G
	(m)	(MPa)	(MPa)
249-1-1	0.6	0.95	0.27
249-3-4	2.0	2.13	0.61
249-6-2	4.0	2.04	0.58
249-9-2	6.3	4.90	1.39
248-12-2	11.5	3.60	1.02
248-14-1	16.	8.02	2.28

5.4.3 Simple shear test results

As described in 5.3.3, the so-called simple shear test is actually a shear test with controlled principal stress directions. Pure shear test results are presented here. By pure shear it is meant that the loading is controlled in a way that only increases of shear stress applied on the specimen without any normal stress change. Some test results have already been shown in Chapter 3. One more example is given in Figure 5.28. Undrained shear strength and shear modulus were obtained from the curves of shear stress - strain stress, as shown in Figure 5.28.

In order to compare the results of shear tests with the results of triaxial tests and oedometer tests, the shear modulus was obtained from the undrained Young's modulus E_u and constrained modulus M . From Equations 5.22 and 5.24 we have:

$$M = \frac{2G(1-\mu')}{(1-2\mu')} \quad (5.25)$$

The Young's modulus and constrained modulus were calculated by Equation 5.24 and 5.25 and the results are shown in Table 5.7. The results of shear modulus, undrained strength and the undrained Young's modulus are also shown in Figure 5.29 to Figure 5.31.

Table 5.7 Summary of simple shear test results

Specimen	Depth	G	Su	Eu	M
	(m)	(MPa)	(kPa)	(kPa)	(MPa)
#2	6.3	1.73	24.0	5.19	6.05
#3	6.3	2.18	23.0	6.55	7.65
#9	6.3	2.0	20.0	6.0	6.99
#17	6.6	2.57	26.0	7.71	8.99

Figure 5.28 Simple shear test result for #9

Figure 5.29 Shear modulus from the simple shear tests

Figure 5.30 Shear strength from the simple shear tests

Figure 5.31 Undrained Young's moduli from the simple shear tests

5.5 COMPARISON OF COMPRESSIBILITY AND UNDRAINED STRENGTH

5.5.1 Comparison of compressibility

The compressibility parameter, measured by the FDP test is shear modulus. For the oedometer test, the direct measurement is compression index C_c , which was converted to constrained modulus as given in Table 5.3. The shear modulus can be estimated by the following equation if the drained Poisson's ratio is known:

$$G = \frac{M(1-2\mu')}{2(1-\mu')} \quad (5.26)$$

As mentioned in section 5.4.2, comparison of a drained test to the undrained test indicates that the Poisson's ratio for the drained condition may be assumed as 0.3. The resulting values of shear modulus converted from constrained modulus are shown in Figure 5.32. The results of shear modulus from FDP tests pen1 and pen2 are presented for comparison in this graph, as well. Clearly, the values of shear modulus estimated from oedometer tests are much smaller than those from the FDP tests. Although the stress paths of oedometer test and FDP test are totally different, which means the comparison of their results is not theoretically sound, the comparison results, shown in Figure 5.32, do give a clear indication that settlement estimated from the results of oedometer tests would be much higher than those from results of the FDP tests.

In a similar way, the shear modulus was converted from the undrained Young's modulus from the triaxial tests. The Poisson's ratio for the undrained condition is 0.5. Hence, the shear modulus can be obtained from dividing the undrained Young's modulus by a factor of 3. The results of shear modulus from triaxial tests together with the results

from FDP tests are shown in Figure 5.33. Although the values of shear modulus from triaxial tests are not as low as those from oedometer tests, they are still much lower than the shear modulus obtained from the FDP tests.

Figure 5.34 shows the results of shear modulus from simple shear tests together with the results from FDP tests. The simple shear tests were performed for soil at depths of 6.3 m and 6.6 m. The two FDP tests were all performed at a depth of 6.5 m. The values of shear modulus from simple shear tests compared much better with those from FDP tests than the results from oedometer and triaxial tests. The average value of shear modulus from simple shear tests was about 2.1 MPa and the average from FDP test was about 2.9 MPa.

In conclusion, between the laboratory tests, the oedometer tests yielded the lowest modulus values. Values of moduli from the triaxial tests are much higher than those from the oedometer tests. Simple shear tests gave the highest values among all these three types of laboratory tests.

The overall comparison indicates that the compressibility measured from the laboratory tests were much higher than that from the FDP tests.

5.5.2 Comparison of Undrained Strength

The results of undrained shear strength from triaxial tests are shown in Figure 5.35, together with the results from the FDP tests. The strengths from the FDP tests were obtained with the method of limit pressure. The profile of undrained shear strength from the triaxial tests agreed very well with the resulting undrained strength from the FDP test pen1. Values of strength from FDP test pen2 were higher than those from triaxial tests. The average values of undrained strength from the FDP tests were calculated and are presented in Figure 5.36 together with the triaxial test results. The values of triaxial tests

represented a lower bound of undrained strength measured from both types of testing. At depth around 16 m, the undrained strength of the triaxial test did not seem to agree with the average value from the FDP tests. Figure 5.35 shows that the values of the undrained strength fluctuate with change of depth for soil below 15 m. The values of the triaxial undrained strength did agree with the undrained strength values from test pen1.

The undrained strengths estimated from the simple shear tests are shown in Figure 5.37 together with the results from the FDP tests. The depths of soils tested ranged from 6.3 to 6.6 m. The results from the simple shear tests varied but fell into a close range. From the graph, it can be seen that the average values of the simple shear tests agreed with the average values of the FDP tests.

Hence, the values of undrained strength from the laboratory tests agreed with those from the FDP tests.

5.6 DISCUSSIONS

In the comparisons of compressibility measurements, the Poisson's ratio for drained condition was assumed as 0.3. The effects of this assumption need to be examined, as values of the Poisson's ratio varies for different soft silty clays. They could range from 0.15 to 0.35.

In obtaining the drained Young's modulus, E' , from the undrained Young's modulus, E_u , for the triaxial tests, the equation 5.15 was used. With the values of the Poisson's ratio assumed as 0.15 to 0.35, the ratios between E' and E_u were calculated and the results are shown in Table 5.8, where the values of A was taken as $1/3$. These data clearly indicated that the differences of resulting drained Young's moduli caused by the assumptions of drained Poisson's ratio were small. The biggest difference was only 11% for the μ' assumed to be 0.3 and 0.15.

3

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Table 5.8 Effects of the Poisson's Ratio on Converting E_u to E'

Assumed μ'	Resulting Ratios	Comparison with $\mu' = 0.3$
$\mu' = 0.15$	$E' = 0.77 E_u$	11%
$\mu' = 0.20$	$E' = 0.80 E_u$	8%
$\mu' = 0.30$	$E' = 0.87 E_u$	
$\mu' = 0.35$	$E' = 0.90 E_u$	7%

In comparing the triaxial test and the oedometer test, the constrained modulus M was converted to a drained modulus, E' , by following the equation 5.22. Since it was the undrained modulus, E_u , being directly measured, M should be related to E_u by combining equation 5.15 with 5.22. Thus, we have:

$$M = \frac{E_u(1 - A + 2A\mu')(1 - \mu')}{(1 + \mu')(1 - 2\mu')} \quad (5.27)$$

With the Poisson's ratio assumed as 0.3 and 0.15, the resulting difference was about 30%. This was quite significant for the comparison between the oedometer test and the triaxial test. The measured values of constrained modulus from the oedometer tests and the converted values of constrained modulus from the triaxial test results are presented in Table 5.9. The values from the triaxial tests were approximately three times of those from the oedometer tests when the Poisson's ratio was assumed as 0.3. With the Poisson's ratio reduced to 0.15, the resulting constrained moduli became 30% smaller, but in general, they were still higher than those from the oedometer tests.

In the comparisons of laboratory tests with the FDP tests, the effects of the Poisson's ratio variation were very small and insignificant. For the triaxial tests, the 11% variation of the moduli did not affect the comparison with the FDP results at all. The comparison was shown in Figure 5.33. For the oedometer tests, the moduli were many

Table 5.9 Comparison of Constrained Modulus from Oedometer and Triaxial Tests

		$\mu' = 0.3$	$\mu' = 0.15$
Depth (m)	M (Oedometer) (MPa)	M (Triaxial) (MPa)	M (Triaxial) (MPa)
0.6	4.1	0.95	0.66
2.0	1.0	2.11	1.47
4.0	0.78	2.03	1.41
6.3	1.0	4.85	3.39
11.5	2.54	3.56	2.49
16.0	2.05	7.95	5.55

times smaller than those of the FDP tests, as shown in Figure 5.32. Hence, the 30% variation of the constrained moduli from the oedometer tests did not affect the comparisons to the FDP results.

For the simple shear tests, the shear moduli were directly measured from the tests. Hence, the Poisson's ratio does not affect the comparisons with the FDP tests.

Among the laboratory tests, the oedometer test yielded the lowest values of compressibility. The values from the triaxial test were higher than those of the oedometer test. The simple shear test yielded the highest values. The values from these three tests were all smaller than those from the FDP test. The reasons can be attributed to soil disturbance, size of the specimen used, and confining stress release.

The soil samples were under certain stress confinement when in in-situ state. When they were taken out from the in-situ position to the ground, the confining stresses were released. Thus, some of the bindings between the soil particles were weakened or destroyed. Stress release was experienced by all the samples.

The specimen size of the oedometer tests is much smaller in comparison with that used for the triaxial tests. The height was only about 2 cm. During the trimming process,

the specimens were inevitably disturbed. The specimen size used for the triaxial test was much larger, with the height of 14.5 cm and the diameter of 7.25 cm. As well, much less trimming was applied on the triaxial test specimen. Only the two ends were cut and trimmed, in order to obtain the desired specimen height.

For the simple shear test, the much higher loading rate used seemed to be the reason for the high shear moduli obtained. The specimens were failed typically within two hours, while for the triaxial tests, it took about 24 hours. More studies are needed to understand the effects of the loading rate on the shear modulus.

For the FDP test, the soil tested was under the in-situ state. No stress release was experienced by the soil. Although the soil was disturbed during the probe installation, the disturbance occurred in a confined state, which prevented severe effects on the soil structure or the inner bindings between the soil particles. Another advantage is that the soil disturbance follows a certain pattern and the effects can be studied.

The good agreements of the undrained strengths between the laboratory tests and the FDP tests indicated that the disturbance effects on the undrained strength were very small. The reason could be the fact that the measured undrained strength was the ultimate strength. When the loading on the soil reaches the ultimate strength, the deformation or the strain are usually high. At this stage the bindings inside the soil has already been broken, hence, the factors that affect the modulus as discussed earlier do not have the same effects on the undrained strength.

Figure 5.32 Comparison of the oedometer and the FDP test results

Figure 5.33 The triaxial and the FDP test results

Figure 5.34 Shear moduli from the simple shear and the FDP tests

Figure 5.35 Undrained Strengths from the Triaxial and the FDP tests

Figure 5.36 Undrained Strengths from the Triaxial and the FDP tests

Figure 5.37 Undrained Strengths from the simple shear and the FDP tests

Chapter 6

COMPARISON OF FDP TESTS WITH OTHER IN-SITU TESTS

6.1 INTRODUCTION

In situ tests have become an important means of investigating behaviour of soils as foundation materials. Compared to laboratory tests, in situ tests have the following advantages (Jamiolkowski et al., 1985):

1. The volume of soil tested is larger than those in most laboratory tests; hence in situ tests should reflect more realistically the influence of the macrofabric on the measured soil characteristics.
2. Many devices produce a continuous record of the soil profile which allows the soil macrofabric and layer boundaries to be determined.
3. In situ tests can be carried out in soil deposits in which undisturbed sampling is extremely difficult or unreliable.
4. The soils are tested in their natural environment which may not be preserved in laboratory tests.
5. In general terms, soil exploration by means of in situ techniques is more economical and less time consuming than laboratory tests.

For soils which are sensitive to sampling disturbance, such as the silty clays described in Chapter 2, the development of a reliable and practical in-situ testing method is especially important, because it is no longer a complementary method but an independent testing method. From the results of laboratory tests presented in the last chapter, it was clearly shown for this specific soil, that traditional laboratory tests yielded

much smaller values of modulus than in-situ tests.

In the development of in-situ testing methods, there is a great need for comparison of results from different test methods. Especially for silty soils, very few comparison data, in terms of compressibility and strength, are available.

In-situ tests conducted in this study consist of the full displacement pressuremeter test, piezocone test and the self-boring pressuremeter test. Three sites were tested. Results of these tests are presented and compared with each other.

The piezocone has been recognized as a very efficient tool for soil classification. It can produce a continuous record of the soil profile. With the measurement of pore pressure development during the penetration, a piezocone test can generate even more accurate classification than the old cone penetration tests. It has been used also, to estimate the undrained shear strength and the preconsolidation pressure of the soil tested. In this study, the piezocone test is used to examine the uniformity of the testing site. Undrained strength was estimated from the piezocone test results and compared with the results from the FDP tests.

The self-boring pressuremeter test was developed to minimize the disturbance during the installation of the pressuremeter probe. With care of operation, disturbance of the soil during the insertion of the probe can be greatly reduced. However, there are apparent drawbacks associated with the self-boring pressuremeter test. Its operation is relatively complicated and time consuming. The cost of running self-boring pressuremeter tests is relatively high for it to be used in daily engineering practice. In this study, shear modulus and undrained shear strength were obtained from the self-boring pressuremeter tests. These values served as references for comparison with FDP test results and other test results.

6.2 PIEZOCONES AND SELF-BORING PRESSUREMETER TESTS

6.2.1 Piezocone Penetrometer Test

An electrical GMF 50-KN cone penetrometer fitted with a 60° tapered, 10 cm² area cone was used to measure tip resistance (q_c) and sleeve friction (f_c). The pore water pressures were measured through a 4 mm thick cylindrical porous stone located immediately above the conical tip. The test was monitored by two systems. The first was a Watanabe 4-pen recorder for tracing signals corresponding to q_c , f_c and u . The chart speed was driven by a rotary depth encoder so that movement of the chart paper was proportional to the actual depth of penetration. This type of recorder enables continuous monitoring, but the data are not amenable to digital analysis without tedious mechanical digitizing. A computer system was used, therefore, in addition to the chart recorder. The rate of penetration for all tests of this study was approximately 1 cm/s.

Before each test, a slightly oversized hole, about 300 mm deep, was made by pushing a rod into the ground. The piezocone was lowered into the hole to a depth just above the bottom. The output signals were monitored continuously for about 10 min. When the readings stabilized a set of zero reference readings was taken. After the test, the zero readings were taken again to compare with the original readings. The difference served to reveal the quality of the tests. For all the tests reported herein, no significant zero shift was observed.

6.2.2 Self-boring Pressuremeter Test

The self-boring pressuremeter used in this study is similar in principal to that described by Wroth and Hughes (1973). The probe was 80 mm in diameter and 0.9 m

long, with a cutting shoe and inside rotating cutter at the bottom. Water was pumped down the shaft of the cutter to flush to the surface the soil that enters the bottom of the probe. Above the shoe was an adiprene membrane section about 0.61 m long, protected by an outer flexible stainless steel sheath. Located near the centre of the membrane were sensors for measuring the total radial pressure applied to the soil, pore pressure, and the radial displacement of the membrane. Dry nitrogen gas was used to expand the membrane at a strain rate of 0.35%/min. To achieve a constant strain rate of testing, a servo-mechanism, a computer and a data acquisition system developed by K.T.Law (1984) were used. Data were stored on a cassette tape for on-site and future analysis. Details of this system has been described elsewhere (Law, 1984).

The test procedures (Law and Eden, 1980) generally consist of five steps: (1) measurement of membrane resistance; (2) self-boring to depth; (3) waiting for the pore pressure to stabilize; (4) loading the soil by inflating the membrane and (5) complete unloading.

Before inserting the probe into the ground, the resistance of the membrane was measured by slowly inflating the membrane in air and recording the pressure at different strains. For the first test, the hole was augured down to about 1.5 m above the intended depth. The probe was then lowered into the hole and allowed to self-bore at a rate dependent on the nature of the soil. The guiding factor was to obtain a homogeneous slurry without clay lumps in the cuttings washed out with water. A typical rate of 0.1 m/min was used. When the pressuremeter had bored to the desired depth, the pore-water pressure was measured by the effective pressure sensor. The probe was then inflated incrementally at a rate of 9.8 kPa/min. When the strain reached around 10%, unloading began at about 50 kPa/min. After the gas pressure was removed, and if the strain sensors indicated a complete return of the membrane to the pretest position, the probe was advanced to the next depth for another test.

6.2.3 Interpretation of Piezocone and Self-boring Pressuremeter Test

Tip resistance correction for piezocone

During the advancing of the cone, excess water pressures are generated and together with the initial pore water pressure that act on the exposed surface behind the cone tip and on the end of the friction sleeve. These water forces result in the measured tip resistance (q_c) does not represent the true total stress resistance of the soil. This error in the measurement can be overcome by correcting the measured q_c for unequal pore pressure effects using the following relationship (Baligh et al., 1981; Campanella et al., 1982):

$$q_T = q_c + u(1 - \alpha) \quad (6.1)$$

where:

q_T = corrected total tip resistance;

u = pore pressure behind the cone tip;

α = net area ratio, the ratio of $(1 - \alpha)$ used here is 0.34 (Konrad and Law, 1987).

Undrained shear strength from piezocone tests

The interpretation of undrained shear strength from the piezocone tests followed the method proposed by Konrad and Law (1987). Values of undrained strength from this method were found in agreement with those from field vane test and those from Vesic's model at two soft clay sites (Konrad and Law, 1987). The following relationship was proposed in their method:

$$S_u = \frac{q_T - (1+F)\sigma_i + F\alpha u}{\frac{4}{3}(1+F)(\ln I_R + 1)} \quad (6.2)$$

where:

S_u = undrained shear strength;

q_T = corrected tip resistance;

u = pore pressure;

σ_i = initial total stress;

I_R = rigidity index;

F = a parameter relating to effective friction angle;

α = a parameter of pore pressure at failure zone.

For the above parameters only two of them, namely q_T and u , can be determined from piezocone tests. The remainders have to be evaluated independently (Konrad and Law 1987).

The initial stress σ_i was originally assumed to be spherical and should be approximated by the in situ total octahedral stress. However, the determination of octahedral stress needs information of the coefficient of earth at rest, K_0 , which may not be readily available. Hence, the vertical total stress σ_{vo} was used for approximation of σ_i . The error of the resulting strength from this approximation was shown to be small and on the safe side (Konrad and Law 1987).

The factor F was defined as:

$$F = M \tan \phi' \cot \theta \quad (6.3)$$

Where M is the friction factor for the soil acting on the cone surface. For a smooth steel surface, M can be taken as 0.5 to 0.75 and for usual roughness, M can be taken equal to unity. The effective friction angle ϕ' was suggested as 35° for silty soils and 27°

for clays. The range of factor α was shown to be 1 - 1.1 for soft soils.

Shear modulus from self-boring pressuremeter tests

The shear modulus is calculated from the slope of the unload-reload curve of the self-boring pressuremeter tests by the following equation:

$$G = \frac{1}{2} \frac{dp}{d\epsilon_c} \quad (6.4)$$

where p is the pressure and ϵ_c is the cavity strain.

The methods for estimating undrained shear strength from pressuremeter tests were described in Chapter 5. The method of limit pressure was used for the undrained strength interpretation of the SBP tests.

6.3 SOIL PROFILES AND TESTS LAYOUT

In situ tests were carried out at three sites. These sites were the Sackville site in New Brunswick, the Diplomatic Hotel site in Fredericton, New Brunswick, and the test site at National Research Council campus on Montreal Road, Ottawa.

Information for the soil profile at the Sackville site was given in Chapter 6. The soils consist of mainly silty clays. There is a clay crust of about 1 m on the top.

The site of the Diplomatic Hotel is located besides the St. John River in the city of Fredericton, the capital of the province of New Brunswick. The soil profile consists of three major layers of soil. The top layer is uniform brown sand, and underlying the sand are layers of clayey silt and silty clay. The third layer of the soil consists of thin layers of stiff clay and dense sand.

The National Research Council site is in an open field within the Montreal Road

campus. The subsoil profile consists of Champlain Sea clay underlying a fissured crust 7 m thick. The natural water content is about 80% at a depth of 7 m and 60% for 17 m below ground surface. It is overconsolidated, with OCR values ranging between 1.7 and 3.7. For the upper 10 m, the sensitivity is relatively low but is extremely high (greater than or equal to 200) below a depth of 12 m (Konrad and Law, 1987).

Typical values of the properties of the soil for the National Research Council site are shown in the Table 1 (after Konrad and Law, 1987).

Table 6.1 Typical values of soil properties at NRC

Water content: %	80
Plastic limit: %	25
Liquid limit: %	60
Plasticity index: %	35
Clay content: %	70
Sensitivity	10-500
OCR	1.7-3.7
Ground elevation:m	61

The major application of the static cone penetration test has been for soil profiling. Soil classification from the static cone test data has been related to cone tip resistance q_c , and friction ratio, $FR (=f_s/q_c)$. Generally, sandy soils have high cone tip resistance and low friction ratios, and clayey soils have high frictions but low tip resistance. With the piezocone test, one extra parameter, the pore pressure, is measured. This feature greatly enhances the ability of soil profiling and identification. As mentioned before, the unbalanced water forces due to unequal end areas in cone design can be corrected. Thinner layers of soil changes can be identified. As suggested by Robertson et al. (1986), three parameters, namely q_T , B_q and FR should be used together. The charts used

for soil profiling were presented in Chapter 2.

The results of B_q , u , q_T and friction ratio FR from a piezocone test (tm3) at the Sackville site are shown in Figure 6.1. The soil classifications obtained from the results of piezocone are shown in the same figure, as well. The top 15 m contained a lot of organic materials as indicated by the friction ratio and the tip resistance q_T . From 15 m to about 17 m there were layers of sand lenses as indicated by the pore pressure parameter ratio B_q and the tip resistance q_T . Most of the soils were silty clay and soft silty clays. Combined with the results of test tm1 we know that there was a hard layer of sandy silt to sand starting at the depth of 22 m.

Results of B_q , u , q_T and friction ratio FR from test f89t1 at the Diplomatic Hotel site are shown in Figure 6.2, together with the soil profile obtained by using the piezocone data. Three layers of soil can be clearly identified from the piezocone data. B_q and q_T or FR and q_T all indicated the top 7.5 m sand layer. Then B_q and FR increase dramatically and the tip resistance drops, which show that this was a layer of clay silt to silty clay. Underlying the silty clay was the layer of sandy silt to sand.

Figure 6.3 shows the results of B_q , u , q_T and friction ratio FR from piezocone test nrc1. Generally three layers can be identified from the piezocone data. At the top was a layer of clay crust. If only FR data and Q_T data were available, the soil might have been identified as a silty sand. The pore pressure or the B_q indicates that the pore pressure increased steadily. The shape of q_T also reveals crust type behaviour. Below the clay crust were silt clay and soft clay.

The in-situ test program at the Sackville site was part of a test embankment project. The test program consisted of seven self-boring pressuremeter tests, two series of full displacement pressuremeter tests, and five piezocone tests. The layout of these tests are shown in Figure 6.4, which included the locations of sampling for laboratory tests and the locations of vane tests conducted by a Laval University team (Rowe et. al. 1991).

Figure 6.1 Piezocone results from test "tm3"

Figure 6.2 Piezocone test results for "f89t1"

Figure 6.3 Piezocone test results for "nrcl"

Figure 6.4 Locations of in-situ tests

At the sites of the Diplomatic Hotel and National Research Council, the tests were performed much more closely with one another than at the Sackville site.

6.4 COMPARISONS OF SHEAR MODULI

6.4.1 Introduction

Results of FDP tests from all three sites will be used in the comparison of measured shear modulus. The uniformity of soil stratigraphy at each site was investigated by piezocone tests and the outcome of variation of soil stratigraphy will be presented first. The results of shear modulus from FDP tests will be compared to the results of shear modulus measured from self-boring pressuremeter tests.

6.4.2 Soil Stratigraphy

Results of piezocone tests at the Sackville site are shown in Figure 6.5 to Figure 6.7. The values of pore pressure parameter ratio are shown in Figure 6.5. Figure 6.6 shows the pore pressure and the corrected tip resistance. Figure 6.7 shows the values of the side friction ratio. Generally, these results reflect similar soil stratigraphy at different locations. There are some variations at certain depths. The different results of tip resistance indicate that the top 1 to 2 m was not uniform. From 2 m to about 12 m, the soil stratigraphy was uniform as indicated by comparable values of pore pressure and tip resistance. The results of pore pressure parameter ratio however, indicate that there were thin layers of sand lenses in the silt clay. From about 12 m down the trend of tip resistance and pore pressure became quite different. Pore pressure dropped dramatically

Figure 6.5 Piezocone test results of B_q

Figure 6.6 Piezocone test results

Figure 6.7 Piezocone test results

and the tip resistance increased, which suggests that the content of sand layer increases from TM2 towards TM5.

Piezoecone tests were performed at two locations at the site of National Research Council of Canada. Results of pore pressure, tip resistance and the pore pressure parameter ratio from the piezocone tests are shown in Figure 6.8. Values of pore pressure and the pore pressure parameter ratio all indicate that the uniformity of soil stratigraphy was very good. The only variation comes from the tip resistance values for the top layer of about four meters thick. This was the crust layer composed of stiff clay.

For the Diplomatic Hotel site in Fredericton, the results of two piezocone tests are shown in Figure 6.9. The soil stratigraphy was mainly composed of three layers: the sand layer of the top 8 m; the silt clay; and the sandy silt layer. The two tests exhibit very close values of pore pressure parameter ratio and tip resistance.

In order to compare in-situ tests results at the Diplomatic Hotel with data available at the Bank of Montreal Building located in downtown Fredericton, the soil profile at the Bank of Montreal site was compared with the one at the hotel. The soil stratigraphy at the Diplomatic Hotel site was in the same geological formation along the St. John river as downtown Fredericton. Soil profile in downtown Fredericton is shown in Figure 6.10. The profile of piezocone data from both sites are shown in Figure 6.11. Both sites are composed of sand and silty clays. Based on the piezocone data, the soil of grey silty clay from 20 m to 30 m at the site of Bank of Montreal corresponded to the silty soil from 8 m to 16 m at the site of the Diplomatic Hotel.

6.4.3 Comparison with self-boring pressuremeter test

In terms of the ability to reduce soil disturbance, self-boring pressuremeter is still the best among all pressuremeter tests. Hence, the values of shear modulus measured

Figure 6.8 Piezocone test results

Figure 6.9 Piezocone test results

Figure 6.10 Soil profile at Fredericton (Alkins, 1983)

Figure 6.11 Comparison of piezocone data (Diplomatic Hotel & Downtown Fredericton)

from the self-boring pressuremeter tests were used for comparison with the shear modulus from the full displacement pressuremeter tests. Self-boring pressuremeter tests were performed at both the Sackville site and the National Research Council site. Data of self-boring pressuremeter tests from the Bank of Montreal site at downtown Fredericton was used for comparison with those from the Diplomatic Hotel site.

Figure 6.12 shows the results of shear modulus from self-boring pressuremeter tests and FDP tests. Self-boring pressuremeter tests were performed within the range of 2 m and 12 m depth. From 4 m down to 12 m, the agreement between the results of self-boring pressuremeter tests and the FDP tests were very good. At depth of about 2.5 m, modulus from self-boring pressuremeter test was smaller than that from FDP test. The difference was not significant and was most likely because of the soil variation.

The results of shear modulus from the self-boring pressuremeter and FDP tests are shown in Figure 6.13. The dots represent the self-boring pressuremeter test results, while circles represent the FDP test results. Although at 7 m depth the modulus from the self-boring pressuremeter test was smaller than that from the FDP test, the overall agreement between the two test results were still very good.

Figure 6.14 shows the comparison of measured shear modulus from self-boring pressuremeter tests and FDP tests for the Diplomatic Hotel site. The self-boring pressuremeter tests were conducted at the location of Bank of Montreal at Fredericton by J. Hughes (reported by Landva et al., 1987). As described before, the silty clay at the site of Diplomatic Hotel can be considered the same soil formation as the layer of grey silty clay at the site of Bank of Montreal. The resulting shear modulus from the FDP tests were plotted in the same graph as the self-boring pressuremeter results by adjusting the depth of FDP tests. Again the agreement between the values of shear modulus from the FDP and the self-boring pressuremeter tests were very good.

Figure 6.12 Shear modulus from FDP tests

Figure 6.13 Shear modulus from FDP tests

Figure 6.14 Shear modulus from FDP tests

6.5 COMPARISONS OF UNDRAINED STRENGTHS

6.5.1 Introduction

Undrained shear strengths were obtained from the FDP tests at all three sites. The strengths were calculated by the method of limit pressure. Comparisons were made to the results of self-boring pressuremeter tests, piezocone tests and field vane tests. As for the comparison of shear modulus, the repeatability of the measure undrained strength were examined. For self-boring pressuremeter test, the method of calculating undrained strength are discussed and the results compared.

6.5.2 Comparison to self-boring pressuremeter test

As discussed in Chapter 5, there are several methods for calculating undrained strength from pressuremeter tests. The two most commonly used are the limit pressure method and the slope of pressure - $\log_e(\Delta V/V)$ relationship method, as recommended by Mair and Wood (1987).

At the site of Sackville, seven self-boring pressuremeter tests were conducted at different depths. The undrained strengths were calculated using both the limit pressure method and pressure - $\log_e(\Delta V/V)$ slope method. The results are shown in Figure 6.15, together with undrained strength from the FDP tests. For the self-boring pressuremeter tests, values from the pressure - $\log_e(\Delta V/V)$ slope method were much higher than those calculated from the limit pressure method. The average ratio of the undrained strengths from these two methods was 1.85, as shown in Table 2.

Table 6.2 Results of undrained strength obtained with different methods

Depth (m)	Su_{Limit} (Limit pressure method)	Su_{Ln_slope} ($p:\log_e(\Delta V/V)$ slope)	$Su_{Limit}/$ Su_{Ln_slope}
2.4	21.04	32.45	1.54
3.9	20.51	36.34	1.77
3.9	21.94	37.69	1.72
5.4	15.20	32.89	2.16
5.4	26.94	49.41	1.83
6.9	17.76	37.95	2.13
12.1	40.48	73.93	1.82
		Average ratio =	1.85

Figure 6.15 shows that the results of undrained strength from self-boring pressuremeter test calculated by the limit pressure method agreed very well with the results from the FDP tests.

For the NRC site, results of the undrained strength from the self-boring pressuremeter tests and from FDP tests are shown in Figure 6.16. The values of undrained strength for the self-boring pressuremeter tests were calculated by pressure - $\log_e(\Delta V/V)$ slope method. The average ratio of values from pressure - $\log_e(\Delta V/V)$ slope method to limit pressure method, obtained from the Sackville site, was used for modifying the values (NRC site) from pressure - $\log_e(\Delta V/V)$ slope method. The results of modified undrained strength values agreed with the results of FDP tests very well.

For the data from downtown Fredericton, the values of undrained strength obtained

Figure 6.15 Undrained strength from FDP tests

Figure 6.16 Undrained strength from FDP tests

Figure 6.17 Undrained strength from FDP tests

by J. Hughes are represented by black dots in Figure 6.17. In the same figure, values of undrained strength from the FDP tests were represented by circles. It was clear that values of undrained strength from J. Hughes were much higher than those from the FDP tests. The method for obtaining the undrained strength was not specified (Landva et al. 1987). If we assume that they were obtained by pressure - $\log_e(\Delta V/V)$ slope method, then the data can be modified by the ratio of 1.85, obtained from data at the Sackville site (Table 6.2). The modified data are represented by black squares in Figure 6.17. They agreed closely with the values of the undrained strength from the FDP tests.

6.5.3 Comparison with piezocone test

Many methods have been proposed for calculation of undrained strength from piezocone test results. Konrad and Law's method (1987) is used here. This method was applied at three soft clay sites and was compared with other methods, and values of undrained strength from this method were found in agreement with those from field vane tests and those from Vesic's method (Konrad and Law, 1987). The relationship was given as Equation 6.2. In this method, one needs to know the corrected tip resistance q_t , pore pressure u , initial total stress σ_i , rigidity index I_R , effective friction angle between soil and probe surface and the parameter of pore pressure at failure zone α . The corrected tip resistance and the pore pressure can be obtained directly from the measurement. Initial total stress was assumed to be equal as the vertical total stress. The friction angle was assumed as 30° . And the parameter of pore pressure at the failure zone was assumed to be 1. The rigidity index was obtained from the results of undrained strength values and the shear modulus values from FDP tests. Figure 6.18 to Figure 6.20 show the values of rigidity index calculated from values of the FDP results for all three sites. For the Sackville site, the values of rigidity index ranged from 100 to 200.

Figure 6.18 Rigidity index from FDP tests

Figure 6.19 Rigidity index from FDP tests

Figure 6.20 Rigidity index from FDP tests

Figure 6.21 Undrained strength from piezocone and FDP tests

Figure 6.22 Comparison for undrained strength

Figure 6.23 Comparison for undrained strength

For the National Research Council site, the rigidity index values were all around 200, while data for the site of the Diplomatic Hotel indicated a range of 200 to 300.

Undrained strengths were calculated from piezocone test results and the parameters assumed above. Values of rigidity index shown in Figure 6.18 to Figure 6.20 were used. The resulted undrained strength values together with those from FDP tests are shown in Figure 6.21 to Figure 6.23. Excellent agreement was found between the FDP results and the piezocone results for all three sites.

6.5.4 Comparison with field vane test

Field vane tests were performed at all three sites by different people. For the Sackville site, field vane tests were performed in September, 1989 under the direction of Dr. Rowe (1991). Tests results at location V2 is used here for comparison. Vane Test data at National Research Council site was from Eden and Law (1980). Results of field vane tests performed by SUB SURFACE SURVEYS, Fredericton (Landva et al., 1987) at Bank of Montreal site were used for comparison with FDP results at the site of Diplomatic Hotel. Results of undrained strength values from vane tests together with the results from FDP tests are shown in Figure 6.24 to Figure 6.26. The overall agreements were good for all three sites.

6.6 DISCUSSIONS

The shear modulus measured by the pressuremeter test was in the horizontal direction. In the applications, however, the shear modulus is usually used for problems in the vertical directions. Hence, an assumption is actually made here for such applications, that is, the soil behaves isotropic.

Figure 6.24 Comparison of undrained strengths for the Sackville site

Figure 6.25 Comparison of undrained strengths for the NRC site

Figure 6.26 Comparison of undrained strengths for the Diplomatic Hotel site

The silty clays tested here were believed behaving isotropic, based on two reasons. The first reason is the characteristics of the original environment where the soil deposited. The second reason comes from laboratory test results.

The soils were originally deposited in sea water. In this kind of environment, the soil takes an open card house structure, with the particles randomly oriented. Hence at a macroscopic scale, the soil can be considered isotropic. This point was supported by the triaxial tests performed by Gnanendran (1993). No significant difference was found in the results of Young's modulus and undrained strength for specimens that were trimmed both vertically and horizontally.

As for the undrained strength, it was pointed out by Wroth (1984) that many factors affect its values, such as stress path, rate of strain, and orientation of the failure plane. These conditions vary with different types of testing methods. Hence, for any specific soil, a unique value of undrained strength s_u cannot be expected from different types of

tests.

On the other hand, in general practice, undrained strength is used as a basic parameter regardless of the testing method from which these values are obtained. Often the choice of the testing method is based on the convenience and availability.

There are two aspects need to be stressed in response to this conflict of understanding and application of the undrained strength. One aspect is that these factors that affect the undrained strength of each test method need to be examined for better understanding. The second approach is to improve the correlations between different test methods.

Among the commonly used in-situ tests, pressuremeter is the most promising one. The boundary conditions are well defined, as shown in Figure 3.1. A clear defined stress path is followed and the failure mode is well recognized as a pure shear process. The testing can be modelled as a cylindrical cavity expansion process and the interpretation theory is well established. It is based on these advantages that the effects of soil disturbance can be better appreciated.

However, caution should be taken in the interpretation of undrained strength from the pressuremeter test. Proper method needs to be used, as some interpretation method, such as the pressure - $\log_e(\Delta V/V)$ slope method, seems to be extremely sensitive to even slight soil disturbance. Even for the self-boring pressuremeter tests, which have the ability to reduce the soil disturbance substantially, the test results showed that the undrained strengths from the pressure - $\log_e(\Delta V/V)$ slope method were much higher than those from the limit pressure method (Figure 6.15 to 6.17).

Other studies also indicated that the pressure - $\log_e(\Delta V/V)$ slope method leads to too high values of undrained strength. It was shown by Fahey and Carter (1986) that values of undrained strength from the pressure - $\log_e(\Delta V/V)$ slope method are about twice of those from the field vane tests. Their study also suggests that the difference cannot be

explained solely on the basis of rate of testing and partial drainage during the test. In another study, FDP tests were performed in a soft silty clay site in the Lower Mainland area of British Columbia (Campanella et al., 1990). Values of undrained strength from the pressure - $\log_e(\Delta V/V)$ slope method are more than twice of those from the field vane tests.

Results of both the FDP tests and the self-boring pressuremeter tests indicated that the limit pressure method should be used for the undrained strength interpretation. In another study (Powell, 1990), the limit pressure method was also found to be able to yield consistent undrained strength values in a stiff heavily overconsolidated clay. Four types of pressuremeters were used. They are the self-boring pressuremeter, the Menard pressuremeter, push-in pressuremeter and the FDP. The limit pressure method of analysis worked for all devices. A comparison study of the full displacement pressuremeter tests and the dilatometer tests in clay was carried out by Lutenecker (1990). Results of undrained strength from FDP tests interpreted by limit pressure method were in good agreement with those from field vane tests.

The reason that causes too high values of undrained strength for the pressure - $\log_e(\Delta V/V)$ slope method is mostly likely due to soil disturbance, although for self-boring pressuremeter the disturbance is usually considered very "small". In a study of penetration effects on pressuremeter tests in clay, Huang and Haefele (1990) pointed out that for self-boring pressuremeter test, the position of the cutter during probe insertion can have significant effects on the test results. The "small" displacements around the probe are not necessarily "insignificant". When installing the self-boring pressuremeter in soft clays, the cutter is usually positioned at 15 to 29 mm above the leading edge of the self-boring pressuremeter tube. The leading edge has to penetrate into the soil for that distance before the soil is cut up and removed, which can cause enough disturbance.

For the piezocone performed, the results of the undrained strength agreed well with

those from the FDP test. However, the interpretation is essentially a semi-empirical method. Simplified assumptions were made in the method proposed by Konrad and Law (1987). Other methods proposed for the piezocone test all suffer the similar limitation (Robertson and Campanella, 1988).

The piezocone could be much more effective when combined with the FDP together. When combined together, the two tests can be performed together with much less time needed. In Konrad and Law's method, the rigidity index cannot be obtained directly from the piezocone test, which can be obtained from the FDP tests. There are other advantages for this combination. The piezocone has been used very successfully in identifying soil stratigraphy. When combined with a FDP, which has been demonstrated to be able to yield reliable shear modulus and undrained strength values, more comprehensive information can be obtained with least efforts.

The field vane test was recommended by Campanella and Robertson (1988) to serve as the reference in undrained strength comparisons. Although the actual stress distribution along the vane edges is not yet fully comprehended, it has been used over many years and much experience has been accumulated. The testing process is better understood than that of the piezocone test.

The correlations of undrained strengths between the laboratory and the in-situ test methods showed very good agreements. For more refined comparison, the results from the triaxial tests were on the lower bound, as shown in Figure 5.36. Values from the field vane tests, however, were on the higher bound, as shown in Figure 6.24 to 6.26. The largest difference between the vane tests and the FDP tests were about 30%. These differences should be bear in mind when using these values in designs.

Chapter 7

SETTLEMENT CALCULATION AND CASE STUDIES

7.1 SETTLEMENT CALCULATION

The development of the pressuremeter testing in the United Kingdom and elsewhere has pursued the general aim of making direct measurements of fundamental soil parameters. As it is described in Chapter 3, the unload-reload curve is used to estimate the shear modulus G_{ur} , which essentially is an elastic modulus. This can be further illustrated by the stress - strain relationship and the corresponding stress path as shown in Figure 7.1. As shown in Figure 7.1(b), the stress space is divided into two regions by the yield surface. Inside this surface only elastic strains occur, whereas outside both

Figure 7.1 Yield surface and stress strain relationship (Wroth & Houlsby, 1985)

elastic and plastic strains are possible. The unload-reload process in the pressuremeter test can be represented by line CA' in the stress space. For a pressuremeter test, the initial elastic loading as illustrated by AB in Figure 7.1(a) is usually not well defined, and is affected by soil disturbance. That is the reason why the unload-reload curve is used for estimating the elastic modulus.

For a soil element under the centreline of a foundation, the loading process can be best modelled by line the AD in Figure 7.1(a), which is a combination of elastic and plastic process. Hence the deformation under loading cannot be modelled by an elastic modulus, but should be estimated by a modulus from the slope AD. If the unload-reload modulus is directly used for estimating soil deformation or settlement, the result would be on the unsafe side (under-estimate).

In the French method proposed by Menard et al. (Baguelin et al., 1978), the modulus is obtained from the "approximately straight line" of the pressuremeter curve, which is illustrated in Figure 7.2 as line v_0 to v_f . The curve is a typical result from a Menard pressuremeter test. The value v_0 is the point when the probe inflates into contact with the walls of the borehole. The value v_f corresponding to the point at the end of the "straight line" portion.

Although the line from v_0 to v_f can be approximated by a straight line, it does not really represent elastic behaviour. Line v_0 to v_f is essentially equivalent to line AD, as illustrated in Figure 7.1(a). This statement is supported by the fact that typical self-boring pressuremeter test curves do not show "straight lines" in the beginning, as those from the Menard pressuremeter tests. And the results of the self-boring pressuremeter tests yield more reliable soil parameters than those from the Menard pressuremeter tests. The soil is less disturbed during a self-boring pressuremeter test than in a Menard pressuremeter test.

For more than 30 years the Menard pressuremeter has been used empirically

Figure 7.2 Illustration of French method for obtaining modulus G_M

throughout France for building and bridge foundations. Their method is well established, and comparisons of predictions with actual performance have shown that measured, long-term settlements, are often within 30% of the predicted values (Baguelin et. al., 1978).

A similar procedure to the French method can be followed for estimating settlement from the self-boring pressuremeter tests. The principle behind the procedure would be the same as illustrated in Figure 7.1. The difference is that for a self-boring pressuremeter test curve, the beginning is not a "straight line". The following procedures can be used to obtain the modulus G_{M_SPB} from the self-boring pressuremeter test, corresponding to the modulus G_M from the Menard's method.

A typical self-boring pressuremeter test curve is shown in Figure 7.3(a). The stress - strain relationship can be derived (Palmer, 1972; Ladanyi, 1972; Baguelin et al., 1972) from the pressuremeter curve by:

Figure 7.3 SBP and FDP test curves and the stress strain relationships

$$\tau = \epsilon_c \frac{dp}{d\epsilon_c} \quad \text{or} \quad \tau = \frac{dp}{d[\log_e(\Delta V/V)]} \quad (7.1)$$

where τ is the shear stress, ϵ_c is the cavity strain, p is the pressure and V stands for the volume change in a pressuremeter test.

The stress strain curve is illustrated in Figure 7.3(b). The results of the self-boring pressuremeter tests conducted indicated that the range of line AD can be determined by strains from 0% to 2%. Thus, the modulus G_{M_SBP} can be obtained by the equation:

$$G_{M_SBP} = \frac{p_D - p_A}{0.04} \quad (7.2)$$

Values of modulus G_{M_SBP} were calculated from the self-boring pressuremeter tests conducted at the Sackville site. The results are shown in Table 7.1. The values of G_{M_SBP} were compared with those of G_{ur_SBP} , as presented in Chapter 3. The average ratio of G_{M_SBP}/G_{ur_SBP} is 0.48 (Table 7.1). Although this is an average value, it gives a good representation of the comparison results, as indicated by the data in Table 7.1. The data

Table 7.1 Ratios of modulus G_{M_SBP} to G_{ur} for SBP tests

Test#	Depth (m)	G_{ur} (MPa)	G_{M_SBP} (MPa)	G_{M_SBP}/G_{ur}	G_{M_SBP} (average) (MPa)
s89t6	2.4	1.57	0.97	0.616	0.754
s89t2	3.9	2.04	1.23	0.603	0.979
s89t7	3.9	2.04	1.46	0.716	0.979
s89t3	5.4	2.81	0.81	0.287	1.349
s89t8	5.4	2.81	0.76	0.272	1.349
s89t4	6.9	3.87	1.95	0.505	1.858
s89t5	12.1	9.86	3.75	0.381	4.733
average values =			1.56	0.483	1.714

of the last column in Table 7.1 are values of G_{M_SBP} calculated from the ratio 0.48. The difference of the average value of that column (1.7 MPa) with the average of the actual G_{M_SBP} values (1.56 MPa) is less than 10%.

For the FDP tests, the method for obtaining G_{new} has been proposed in Chapter 3. However, G_{M_FDP} cannot be estimated directly from the test curve. As examined in Chapter 3, the extent of soil disturbance around the FDP probe is much more extensive than that for the SBP or the Menard pressuremeter tests. The resulting pressure - strain curve from the FDP tests would be affected by the soil disturbance, and the slope of the beginning portion of the pressure - strain curve would be much smaller than for the undisturbed soil. The effects of soil disturbance for the initial part of a FDP test curve is complex, as it involves both elastic and plastic behaviours. The ratio of the initial modulus to the unload-reload modulus can be used to indicate the extent of disturbance effect on the beginning portion of the FDP test curve. Results of the ratios of initial modulus to the unload-reload modulus for the FDP tests are shown in Figure 7.4.

Since the values of G_{new} from the FDP tests at the three sites were all in good agreement with those from the SBP tests, as indicated in Figure 6.12 to 6.14 in chapter 6, the ratio of G_{M_SBP}/G_{ur_SBP} from the SBP tests can be used to estimate the values of G_{M_FDP} for the FDP tests:

Figure 7.4 Values of ratio G_{ur_FDP}/G_o for the FDP tests

$$G_{M_FDP} = \frac{G_{M_SBP}}{G_{ur_SBP}} G_{new_FDP} \quad (7.3)$$

Table 7.2 Modulus G_{M_FDP} from the FDP tests

Test# pen1			pen2			pen3		
Depth	G_new	G_M_FDP	Depth	G_new	G_M_FDP	Depth	G_new	G_M_FDP
m	MPa	MPa	m	MPa	MPa	m	MPa	MPa
1.5	2.23	1.07	1.5	4.25	2.04	5.50	111.51	53.53
2.0	2.25	1.08	2.0	5.92	2.84	6.00	182.34	87.53
2.5	3.72	1.79	2.5	5.84	2.81	6.50	107.64	51.66
3.0	3.14	1.50	3.0	6.28	3.02	7.00	108.69	52.17
3.5	2.33	1.12	3.5	2.86	1.37	7.50	45.09	21.64
4.0	2.45	1.18	4.0	2.78	1.33	8.00	4.21	2.02
4.5	2.44	1.17	4.5	2.83	1.36	8.50	34.17	16.40
5.0	2.12	1.02	5.0	2.99	1.44	9.00	25.98	12.47
5.5	4.06	1.95	5.5	2.95	1.42	9.50	28.96	13.90
6.0	3.24	1.55	6.0	2.97	1.42	10.00	23.93	11.49
6.5	2.56	1.23	6.5	3.39	1.63	10.50	13.94	6.69
7.0	3.08	1.48	7.0	3.53	1.69	11.00	4.43	2.13
7.5	3.64	1.74	7.5	2.10	1.01	11.50	17.90	8.59
8.0	3.52	1.69	8.0	3.62	1.74	12.00	15.56	7.47
8.5	2.53	1.21	8.5	2.45	1.18	12.50	14.70	7.06
9.0	4.00	1.92	9.0	4.02	1.93	13.00	18.28	8.77
10.0	3.80	1.82	10.0	5.41	2.60	14.00	28.73	13.79
11.0	6.53	3.13	11.0	7.15	3.43	14.50	17.92	8.60
12.0	9.43	4.53	12.0	6.83	3.28	15.00	18.01	8.64
13.0	7.66	3.68	13.0	8.84	4.24	15.50	19.65	9.43
14.0	46.12	22.14	14.0	9.77	4.69	16.00	89.20	42.82
15.0	5.38	2.58	15.0	20.07	9.63	16.50	7.03	3.37
16.0	12.29	5.90	16.0	41.95	20.13			
17.0	17.28	8.29	17.0	4.86	2.33			
18.0	21.45	10.30	18.0	15.96	7.66			
19.0	6.32	3.03	20.0	14.89	7.15			

For the FDP tests conducted at the Sackville site and the Diplomatic Hotel site, values of G_{M_FDP} are calculated with the above equation. The results are shown in Table 7.2. Apparently, the values of the modulus G_{M_FDP} are smaller than those of the elastic shear modulus. Thus, if the elastic shear modulus is used for estimating settlement, the result would lead to serious underestimation.

In summary, the Menard type modulus G_{M_FDP} can be used to estimate the final settlement by following the French method. The modulus G_{M_FDP} is obtained from the elastic modulus G_{new_FDP} with the relationship of G_M to G_{ur} from the SBP results.

In the following sections, two case studies of settlement calculations are presented. Values of the modulus G_{M_FDP} obtained from the full displacement pressuremeter tests were used to estimate the settlements. The first case is a multistorey building on a raft foundation. The second case is a test embankment.

The method for the final settlement calculation used is proposed by Baguelin et al. (1978). This method is applicable to raft foundations and embankments:

$$s = \sum \frac{\alpha_i \beta \Delta \sigma_w}{E'_i} \Delta z_i \quad (7.4)$$

Where α is the Menard factor, which is determined by soil type and overconsolidation ratio. E'_i is the elastic modulus which can be calculated from the values of shear modulus:

$$E' = 2G(1 + \mu') \quad (7.5)$$

where μ' is the effective Poisson's ratio. β is a function of the safety factor, F :

$$\beta = \frac{2}{3} \times \frac{F}{F-1} \quad (7.6)$$

F is the ratio of the ultimate bearing capacity to the pressure applied by the foundation.

7.2 SETTLEMENT CALCULATION OF A MULTISTOREY BUILDING

7.2.1 Description of the building

The building in this case study is the Bank of Montreal building located in the centre of downtown Fredericton, New Brunswick. It is on the northeast corner of the intersection of King and Carleton Streets as shown in Figure 7.5. The soil profile for this site was given in Chapter 6. The building rises nine stories above one level of underground parking and is founded on a raft foundation. The plan of raft foundation is shown in Figure 7.6 and the elevation is shown in Figure 7.7.

The building is a reinforced concrete structure with precast panel cladding. The elevator shafts and the basement perimeter walls provides the only shear walls. As shown in Figure 7.6, the raft foundation under the nine storey portion of the building is approximately rectangular with dimensions 50.3 by 39.6 m. The raft is 1.06 m thick with two way reinforcement top and bottom.

The construction was divided into two phases. The first phase included excavation of the site, construction of the raft foundation, the parking level and the ground floor. The second phase included the construction of the nine stories and all the finishing work. During the construction, well points were used to dewater the site. The water level was controlled at elevation +4.4 m from original elevation of +6.1 m. Special measures were taken to minimize the disturbance during construction. The raft was constructed in small sections from which the foundation soil was unloaded only over small areas for a short duration.

Figure 7.5 Location of Bank of Montreal Building (Alkins, 1983)

Figure 7.6 Plan of Raft Foundation (Alkins, 1983)

Figure 7.7 Elevation of the Bank of Montreal Building

7.2.2 Compressibility from FDP test results

Results of shear moduli measured from the full displacement pressuremeter tests together with the self-boring pressuremeter test data, were shown in Figure 6.14. The results of the piezocone tests were shown in Figure 6.11, which were the pore pressure and the corrected tip resistance measured. For soils from depths between 6 m to about 21 m, the piezocone data showed that there were thin sand layers in the silty clay, indicated by high tip resistance and low pore water pressure. The values of shear modulus from self-boring pressuremeter tests for the top layer (6 - 16 m), which contained thin sand layers, were much higher than values for the lower layer (below 16 m). The high values of shear modulus (in the horizontal direction) from the self-boring pressuremeter tests reflected the overall effects of silty clay mixed with thin sand layers.

For soil settlement, the soil is stressed in the vertical direction instead of the horizontal direction. The effects of the sand layers should be taken into account in the application of the moduli which were measured in the horizontal direction.

The measured modulus values from the FDP tests represented the characteristics of the silty clay, where there were no thin sand layers. Combining the data of the tip resistance from the piezocone tests and the data from the FDP tests, the moduli for the whole silty clay layer were obtained and are shown in Figure 7.8 by the dotted line.

7.2.3 Settlement calculation

The construction progress was monitored and the actual weight and arrival dates of

construction materials, delivered to the site were obtained. The plot of net building pressure versus time is contained in Figure 7.9., where the net pressure was calculated as the actual building pressure less the pressure decrease due to the excavation of the soil. The final net pressure was taken as 50 kPa as shown in Figure 7.9.

The stress increase in the subsoil because of the net pressure increase was calculated by Newmark's method. The loading was treated as a uniformly loaded rectangular area and the stress increase calculated is along the centreline under the raft foundation. Newmark's influence values were used (Lambe & Whitman, 1969). The results are shown in Figure 7.10.

The settlement is calculated using Equation 7.4. According to Alkins (1983), the soil is overconsolidated, hence the value of α is 1.0. The calculated final settlement was 24.5 mm from the elastic shear modulus and 51 mm from the modulus G_{M_FDP} . A detailed calculation is shown in Table 7.3.

Table 7.3 Settlement calculation with shear modulus from FDP tests

Depth (m)	G (MPa)	$\Delta\sigma'_v$ (kPa)	E' (MPa)	E' _{M_FDP} (MPa)	Settlement	
					(from E') (mm)	(from E' _{M_FDP}) (mm)
7.00	25.00	49.90	62.50	30.00	1.60	3.33
9.00	25.00	49.50	62.50	30.00	1.58	3.30
11.00	25.00	49.40	62.50	30.00	1.58	3.29
13.00	25.00	49.00	62.50	30.00	1.57	3.27
15.00	25.00	48.40	62.50	30.00	1.55	3.23
17.00	25.00	48.10	62.50	30.00	1.54	3.21
19.00	25.00	47.40	62.50	30.00	1.52	3.16
21.00	25.00	47.00	62.50	30.00	1.50	3.13
23.00	27.69	45.80	69.23	33.23	1.32	2.76
25.00	16.15	45.00	40.38	19.38	2.23	4.64
27.00	18.46	44.00	46.15	22.15	1.91	3.97
29.00	18.00	42.60	45.00	21.60	1.89	3.94
31.00	20.08	41.40	50.19	24.09	1.65	3.44
33.00	20.08	39.80	50.19	24.09	1.59	3.30
35.00	20.08	38.00	50.19	24.09	1.51	3.15
(layer thickness = 2 m)				Total:	24.54	51.12

Figure 7.8 Modulus used for settlement calculation

Figure 7.9 Net building pressure versus time (Alkins, 1983)

Figure 7.10 Stress increase in the subsoil

The following equation for settlement calculation was proposed by Menard and Rousseau (1962). In this method, the settlement is divided into two parts: the deviatoric settlement and the spherical settlement. The equation is as the following:

$$s = \frac{2}{E_d} q B_o \left(\lambda_d \frac{B}{B_o} \right)^{\alpha} + \frac{\alpha}{9E_c} q \lambda_c B \quad (7.7)$$

where E_d = modulus within the zone of influence of the deviator tensor, E_c = modulus within the zone of influence of the spherical tensor, q = footing net bearing pressure, B_o = reference width of 60 cm, α = rheological factor, λ_d and λ_c are the shape factors. Figure 2.28 shows the decomposition of the soil into layers for this method.

For this case, we have the following parameters:

$$q = 50 \text{ kPa}$$

$$B = 40 \text{ m}$$

$$B_o = 60 \text{ cm}$$

$$\alpha = 1.0$$

$$\lambda_d = \lambda_c = 1.0$$

As the values of $E_{3/4/5}$, $E_{6/7/8}$, $E_{9/16}$ can be treated as very high. Then E_d is calculated as:

$$\frac{1}{E_d} = \frac{1}{4} \left(\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3/4/5}} + \frac{1}{2.5E_{6/7/8}} + \frac{1}{E_{9/16}} \right) = \frac{1}{4} \left(\frac{1}{E_1} + \frac{1}{0.85E_2} \right) \quad (7.8)$$

And:

$$E_c = E_1 \quad (7.9)$$

From Table 7.3, therefore:

$$E_c = E_1 = 30 \text{ MPa}$$

$$E_d = 55.1 \text{ MPa}$$

The final result gives:

$$s = \left[\frac{2}{9 \times 55.1 \times 1000} 50 \times 0.6 \left(1.0 \frac{40}{0.6} \right)^{1.0} + \frac{1.0}{9 \times 30 \times 1000} \times 50 \times 1.0 \times 40 \right] \times 1000 \text{ mm} \quad (7.10)$$

That is:

$$s = 7.2 \text{ (deviatoric settlement)} + 6.5 \text{ (spherical settlement)} = 13.7 \text{ mm}$$

7.2.4 Comparison with values measured and calculated from laboratory tests

The actual settlement of the building was monitored during and after the construction. The values of pressure increase and the actual settlement are shown in Figure 7.11. At the time of the completion of construction, the monitored settlement was 16 mm. Settlement data shown by Landva et al. (1987) indicates that the final settlement reaches about 40 mm in 1987, which is about four years after the construction. The data is shown in Figure 7.12.

The settlement was also calculated by the one dimensional method based on the

Table 7.4 Summary of settlement values for Bank of Montreal Building

Method	Settlement (mm)	Remarks
1-D E'	20.45	convert G_{new} to M
* 1-D E'_{M_FDP}	51.12	convert G_{M_FDP} to E'_{M_FDP}
Menard	13.7	convert G_{M_FDP} to E'_c and E'_d
Measured	16	Right after construction (Alkins, 1983)
* Measured	40	Four years later (Landva et. al. 1987)
Lab test	163	From consolidation test #2 (Alkins, 1983)
Lab test	104	From consolidation test # 3 (Alkins, 1983)

laboratory consolidation tests by Alkins (1983). Results of two consolidation tests were used and the values of settlement obtained were 163 mm and 104 mm respectively.

Comparison can be made between the results obtained from these different methods. This was done in bar graphs which are presented in Figure 7.13 and Figure 7.14.

Figure 7.11 Monitored data of settlement and building pressure increase (Alkins, 1983)

Figure 7.12 Monitored values of settlement at the centre of raft (Landva et al., 1987)

Figure 7.13 Settlements calculated from FDP tests data and the measured values

Figure 7.14 Settlements calculated from laboratory data and the measured values

The values of settlements obtained from these methods are summarised in Table 7.4.

The comparison showed that the settlement calculated from the results (G_{M_FDP}) of the full displacement pressuremeter tests agreed well with the values of settlement from measurement. The calculation from laboratory consolidation test results however, grossly overestimated the settlement by 2.5 to 4 times. Also, it is shown that if the elastic shear modulus was used for settlement calculation, the settlement would be under estimated and on the unsafe side.

As well, the comparison results indicated that the equation proposed by Menard and Rousseau is not suitable for settlement calculation of raft foundations. The resulting value of settlement was much smaller than the measured value.

7.3 SETTLEMENT CALCULATION OF AN EMBANKMENT

7.3.1 Introduction of the embankment project

The embankment in this case was built in September/October 1989 in a place between Sackville and Aulac in the Province of New Brunswick, Canada. The detailed location and soil profile were shown in Chapter 6 in Figure 6.1 and Figure 6.2. The objective of this project was to provide a case record of deformation and progressive failure for both reinforced and unreinforced embankment sections constructed on a soft compressible soil. In this section, the settlement of the embankment was calculated, based on data of the laboratory tests and the full displacement pressuremeter tests. Comparison was made between the calculated to the observed.

Both in-situ and laboratory tests were conducted for this site. In-situ tests performed consisted of the cone penetration tests, the piezocone tests, the self-boring pressuremeter tests and full displacement pressuremeter tests. The laboratory tests consisted of the

oedometer consolidation tests, the triaxial tests and the simple shear tests.

The design of the test embankment configuration was based on limit equilibrium analysis. The limit equilibrium analysis program (REAP) used for the design can take into consideration of both reinforced and unreinforced embankment behaviour (Rowe et al., 1991)

The test embankment consisted of a 25 m long unreinforced section and a 25 m long reinforced section, connected by a reinforced transition. Berms were provided in the east, west and the southern sides. The overall fill range was about 40 m x 50 m combined the two section and the berms provided.

The embankment was instrumented before and during the construction. The instrumentation included piezometers for measuring excess pore pressures, settlement plates, augers, heave plates, and inclinometer casing for measuring soil movement, and a total pressure cell for monitoring pressure increase due to construction. Data from the instrumentation are very useful for future analysis and interpretations of the embankment behaviour.

7.3.2 Compressibility from FDP tests and laboratory tests results

In Chapter 6, it was shown that the results of the shear modulus measured from the full displacement pressuremeter tests, interpreted using the method proposed, agreed with the results from the self-boring pressuremeter tests (Figure 6.14). This means that the modulus obtained from the full displacement pressuremeter tests can reflect the behaviour of the silty clay. In Chapter 5 the results of the full displacement pressuremeter tests were compared with those from laboratory tests (Figure 5.33). The values of converted shear moduli from these figures are shown again in Figure 7.15. For the full displacement pressuremeter tests, the average values of shear modulus from tests PEN1

and PEN2 were used for settlement calculation.

7.3.3 Settlement calculations

Construction was carried out in several stages. For the unreinforced section, it was performed in four stages. During the first, second and third stages, the thickness of fill was raised to 1.3 m, 2.25 m and 3.4 m respectively. From the fourth stage of the construction, the fill was rapidly loaded to failure. Figure 7.16 shows the sequence of construction for the unreinforced section. Similarly the reinforced section was also built in stages. Figure 7.17 shows the construction sequence for the reinforced section. The total pressure applied on the silty clay due to the increase of fill can be calculated according to the fill height. The total pressure together with the embankment thickness are shown in Figure 7.17.

According to the measurements, settlement of the embankment began to develop rapidly after the fill thickness reached 3.4 m high and then it started to fail. When the fill thickness is equal to 3.4 m, the total pressure was about half the failure pressure, with a safety factor of about 2. The settlement will be calculated under this height. The reason for choosing this height is that the total pressure is much less than the foundation capacity. It is believed that the proposed method of settlement estimation is not applicable to loading close to a general failure.

According to Figure 7.17, the calculated total pressure was 70 kPa, and the measured total pressure was 80 kPa when the embankment height was 3.4 m. The measured value of total pressure was used in the calculation. The stress increase in the subsoil was calculated by Newmark's method. The loading was treated as a uniformly loaded rectangular area and the stress increase calculated is along the centreline under the embankment. The results of the stress increase are shown in Figure 7.18.

Figure 7.15 Comparison of shear modulus from PEN1 to laboratory test results

Figure 7.16 Sequence of construction for the unreinforced section

Figure 7.18 Stress increase in the subsoil

Table 7.5 Settlement calculation from laboratory consolidation tests for the embankment

Depth (m)	C _c	P' _o (kPa)	$\Delta\sigma'_z$ (kPa)	σ'_{zo} (kPa)	P _c (kPa)	e _c	Settlement (m)
1.00	0.30	8.50	80.00	88.50	200.00	1.02	0.0
1.50	0.36	12.75	79.98	92.73	130.50	1.13	0.0
2.00	0.42	17.00	79.97	96.97	60.30	1.23	0.019
2.50	0.48	21.25	79.95	101.20	56.60	1.34	0.026
3.00	0.54	25.50	79.94	105.44	52.90	1.45	0.033
3.50	0.61	29.75	79.75	109.50	49.20	1.56	0.041
4.00	0.67	34.00	79.57	113.57	45.50	1.67	0.050
4.50	0.61	38.25	79.38	117.63	46.88	1.56	0.048
5.00	0.55	42.50	79.20	121.70	48.25	1.45	0.045
5.50	0.49	46.75	78.84	125.59	49.63	1.33	0.042
6.00	0.42	51.00	78.48	129.48	51.00	1.21	0.038
6.50	0.40	55.25	78.12	133.37	55.25	1.16	0.035
7.00	0.37	59.50	77.76	137.26	59.50	1.11	0.032
7.50	0.35	63.75	77.28	141.03	63.75	1.09	0.029
8.00	0.33	68.00	76.80	144.80	68.00	1.07	0.026
8.50	0.31	72.25	76.32	148.57	72.25	1.05	0.024
9.00	0.29	76.50	75.84	152.34	76.50	1.02	0.021
9.50	0.28	80.75	75.13	155.88	80.75	1.00	0.020
10.00	0.26	85.00	74.42	159.42	85.00	0.97	0.018
10.50	0.25	89.25	73.71	162.96	89.25	0.95	0.016
11.00	0.23	93.50	73.00	166.50	93.50	0.92	0.015
11.50	0.23	97.75	71.95	169.70	97.75	0.92	0.014
12.00	0.23	102.00	70.90	172.90	102.00	0.92	0.014
12.50	0.25	106.25	69.85	176.10	106.25	0.93	0.014
13.00	0.26	110.50	68.80	179.30	110.50	0.94	0.014
13.50	0.28	114.75	68.00	182.75	114.75	0.96	0.014
14.00	0.30	119.00	67.20	186.20	119.00	0.97	0.015
14.50	0.32	123.25	66.40	189.65	123.25	0.99	0.015
15.00	0.33	127.50	65.60	193.10	127.50	1.01	0.015
15.50	0.35	131.75	64.72	196.47	131.75	1.03	0.015
16.00	0.37	136.00	63.84	199.84	136.00	1.04	0.015
16.50	0.37	140.25	62.96	203.21	140.25	1.04	0.015
17.00	0.37	144.50	62.08	206.58	144.50	1.04	0.014
17.50	0.37	148.75	60.94	209.69	148.75	1.04	0.014
18.00	0.37	153.00	59.79	212.79	153.00	1.04	0.013
18.50	0.37	157.25	58.65	215.90	157.25	1.04	0.012
19.00	0.37	161.50	57.50	219.00	161.50	1.04	0.012
19.50	0.37	165.75	56.73	222.48	165.75	1.04	0.012
20.00	0.37	170.00	55.95	225.95	170.00	1.04	0.011
20.50	0.37	174.25	55.18	229.43	174.25	1.04	0.011
21.00	0.37	178.50	54.40	232.90	178.50	1.04	0.010
21.50	0.37	182.75	54.40	237.15	182.75	1.04	0.010
22.00	0.37	187.00	54.40	241.40	187.00	1.04	0.010
Total settlement =							0.868 m

Settlement Calculation from Consolidation Tests

As described in Chapter 5, consolidation tests were performed on soil samples

obtained from the test embankment site. Compression coefficients of the soil were obtained from these tests. The initial void ratios were also measured. Values of compression coefficient and initial void ratio are shown in Figure 7.19. The values of preconsolidation pressure were also estimated from the consolidation curves expressed as void ratio - $\log(P)$. The values of the initial vertical effective stress and the stress increases, because of the construction of the embankment, together with the preconsolidation pressure, are shown in Figure 7.20. Terzaghi's (1925) equation was used for the calculation:

$$s = \sum \Delta z_i \frac{C_c}{1 + e_o} \log \left(\frac{\sigma'_o + \Delta \sigma'_z}{\sigma'_o} \right) \quad (7.11)$$

Where:

C_c = compression coefficient;

e_o = initial void ratio;

σ'_o = initial effective vertical stress;

$\Delta \sigma'_z$ = vertical stress increase caused by extra loading;

z_i = thickness of each layer;

The detailed numerical values of the calculation are presented in Table 7.5. The final settlement calculated from this method was 868 mm.

Settlement Calculation from FDP Tests

The full displacement pressuremeter tests were performed at two locations at the test embankment site numbered as test PEN1 and PEN2. The values of shear modulus resulted from these two series tests were presented in Chapter 7. The average values

Figure 7.19 Compression coefficient and initial void ratio

Figure 7.20 Values of vertical stresses for settlement calculation

Figure 7.21 Values of shear modulus averaged from tests PEN1 and PEN2

(Figure 7.21) from the two tests were used for calculating the values of modulus G_{M_FDP} , which are shown in Table 7.6.

The settlement was again calculated by Equation 7.3. The Menard factor α was taken as 1.0 for top 5 meters and 0.67 for the rest of the soil. As mentioned before the safety factor is about 2, hence the factor β can be calculated by Equation 7.6:

$$\beta = \frac{2}{3} \frac{F}{F-1} = \frac{4}{3} \quad (7.12)$$

The Young's modulus E' was calculated by Equation 7.4. The final settlement resulted from this method was 255 mm (see Table 7.6).

7.3.4 Comparison with values measured and calculated from laboratory tests

The actual settlements were monitored for both the reinforced section and the unreinforced section. The settlement plates were placed along the bottom of the embankment. The value of the maximum settlement was obtained from a settlement plate at the centre of the bottom line. The settlement readings at the centre of the unreinforced section are shown in Figure 7.22. In this graph, the changes of embankment thickness for both reinforced and unreinforced sections versus time are displayed, as well. At about 400 hour to 420 hour, the value of measured settlement under the unreinforced section increased sharply, while the thickness of the unreinforced section remained the same. The reason was that the unreinforced section was affected by the continuous increase of thickness of the reinforced section.

If there was no increase of the thickness of the reinforced section, the settlement curve would most likely be as projected in Figure 7.22. At the time when the thickness of the embankment reached 3.4 m, which was far from the failure height, the settlement

Table 7.6 Settlement calculation results from FDP tests for the embankment

Depth (m)	G_{new} (MPa)	$\Delta\sigma_v$ (kPa)	E' (MPa)	$E'_{M, FDP}$ (MPa)	α (from E')	Settlement	
						(from E') (mm)	(from $E'_{M, FDP}$) (mm)
1.50	3.24	80.00	8.09	3.89	1.00	4.93	10.30
2.00	4.09	79.98	10.21	4.90	1.00	3.92	8.16
2.50	4.78	79.97	11.96	5.74	1.00	3.34	6.97
3.00	4.71	79.95	11.77	5.65	1.00	3.40	7.07
3.50	2.59	79.94	6.49	3.11	1.00	6.16	12.84
4.00	2.62	79.75	6.54	3.14	1.00	6.10	12.70
4.50	2.64	79.57	6.59	3.16	1.00	6.04	12.58
5.00	2.56	79.38	6.39	3.07	1.00	6.21	12.93
5.50	3.50	79.20	8.76	4.20	1.00	4.52	9.42
6.00	3.10	78.84	7.75	3.72	1.00	5.08	10.59
6.50	2.97	78.48	7.43	3.57	0.67	3.54	7.37
7.00	3.30	78.12	8.26	3.97	0.67	3.17	6.60
7.50	2.87	77.76	7.16	3.44	0.67	3.64	7.57
8.00	3.57	77.28	8.93	4.29	0.67	2.90	6.04
8.50	2.49	76.80	6.23	2.99	0.67	4.13	8.61
9.00	4.01	76.32	10.03	4.81	0.67	2.55	5.31
9.50	4.31	75.84	10.77	5.17	0.67	2.36	4.91
10.00	4.61	75.13	11.52	5.53	0.67	2.19	4.55
10.50	5.72	74.42	14.30	6.87	0.67	1.74	3.63
11.00	6.84	73.71	17.09	8.20	0.67	1.44	3.01
11.50	7.49	73.00	18.71	8.98	0.67	1.31	2.72
12.00	8.13	71.95	20.34	9.76	0.67	1.19	2.47
12.50	8.19	70.90	20.48	9.83	0.67	1.16	2.42
13.00	8.25	69.85	20.63	9.90	0.67	1.13	2.36
13.50	18.10	68.80	45.25	21.72	0.67	0.51	1.06
14.00	27.95	68.00	69.86	33.53	0.67	0.33	0.68
14.50	20.33	67.20	50.84	24.40	0.67	0.44	0.92
15.00	12.72	66.40	31.81	15.27	0.67	0.70	1.46
15.50	19.92	65.60	49.80	23.91	0.67	0.44	0.92
16.00	27.12	64.72	67.80	32.54	0.67	0.32	0.67
16.50	19.09	63.84	47.74	22.91	0.67	0.45	0.93
17.00	11.07	62.96	27.67	13.28	0.67	0.76	1.59
17.50	14.89	62.08	37.22	17.87	0.67	0.56	1.16
18.00	18.71	60.94	46.77	22.45	0.67	0.44	0.91
18.50	14.68	59.79	36.71	17.62	0.67	0.55	1.14
19.00	10.66	58.65	26.65	12.79	0.67	0.74	1.54
19.50	10.63	57.50	26.59	12.76	0.67	0.72	1.51
20.00	10.61	56.73	26.52	12.73	0.67	0.72	1.49
20.50	10.60	55.95	26.50	12.72	0.67	0.71	1.47
21.00	10.60	55.18	26.50	12.72	0.67	0.70	1.45
21.50	10.60	54.40	26.50	12.72	0.67	0.69	1.43
Total: (mm)						122.55	255.31

was about 100 mm, indicated by "D" in Figure 7.23. Measurements from the settlement plate under the reinforced section showed close results.

Figure 7.22 Readings from settlement plate under the unreinforced section

Figure 7.23 Comparison of settlements estimated from different methods

The long term settlement data for the embankment at height 3.4 m is not available. It is attempted to be deduced from the following examinations.

From observation of the Bank of Montreal Building at Fredericton, the total settlement four years after the construction was 40 mm which was about 2.5 times of the value when the building just completed (16 mm). There are two similarities between the embankment project and the Bank of Montreal project. The first one is that the soil formation for the silty clay in the embankment site is the same as the silty clay in the Bank of Montreal site. The second similarity is that the total loading pressures applied were within half of the failure pressures. Therefore, the best estimation for the final settlement of the embankment at 3.4 m height, can be made according the data of the Bank of Montreal building, that is, the final settlement was 2.5 times of the initial settlement. Consequently, the final settlement was taken as 250 mm for the embankment shown as "E" in Figure 7.23.

Values of the final settlement estimated from the FDP tests, the laboratory consolidation tests and the settlement plates are presented for comparison in Figure 7.23.

Data in Figure 7.23 indicated that the final settlement ("B" in Figure 7.23) estimated from the modulus G_{M_FDP} obtained from full displacement pressuremeter tests

Table 7.7 Summary of settlement estimation for embankment height of 3.4 m

	Method	Settlement (mm)	Remarks
	1-D E'	122.5	convert G_{new} to E'
*	1-D E'_{M_FDP}	255.3	convert G_{M_FDP} to E'_{M_FDP}
	Measured	100	right after construction
*	Projected	250	final settlement
	Lab test	868	Consolidation test

was in excellent agreement with the value of the projected final settlement. The settlement result from the laboratory consolidation tests ("F" in Figure 7.23) however, overestimated the actual settlement seriously, with a magnitude of about 3.5 times. It was demonstrated as well, that the settlement calculated from the elastic modulus ("A" in Figure 7.23) was much smaller than the final settlement. The values of these settlements are summarized in Table 7.7.

7.4 SUMMARY AND DISCUSSIONS

For silty soils, it has been shown that laboratory tests lead to overestimation of soil settlements. This is demonstrated both from the literature (chapter 2) and the applications of the laboratory tests in the two cases presented. A reliable method is needed to overcome this difficulty. Attempt was made to use the results from the FDP tests to calculate the final settlements.

There is a limitation in applying the shear modulus from the FDP tests directly in the settlement calculation. Outside France, development has been made in direct measurement of the fundamental soil parameters. For soil deformation the parameter is the shear modulus, which is obtained from the unload-reload curve. The reason for using the unload-reload curve is that the effects of soil disturbance on the modulus can be reduced to the minimum, which leads to more reliable estimation. The shear modulus is an elastic modulus, which represents the soil behaviour only inside the yield surface, as described by Figure 7.1. But in actual situations, only a small portion of the loading in the beginning can be considered as elastic behaviour. After that small portion, plastic deformation will occur. Obviously, the plastic deformation cannot be reliably modelled by the elastic modulus.

The modulus G_p proposed by Menard is a representation of the overall behaviour of both elastic and plastic deformation. It is not an elastic modulus. The application of this method in France has been very successful.

For the self-boring pressuremeter, the same procedure can be used for obtaining the Menard modulus. Because of its ability to reduce soil disturbance, good quality results can be achieved from the self-boring pressuremeter tests.

In contrast with the self-boring pressuremeter, the use of full displacement pressuremeter is simple and economical. Hence, in engineering applications, it is much more practical than both the Menard pressuremeter and the self-boring pressuremeter.

For a FDP test, the unload-reload curve is inside the yield surface in the stress space, disturbance can be studied by elastic theory. Values of the elastic shear modulus can be obtained based on the analysis result. The calculated results were in good agreements with those from the self-boring pressuremeter tests.

However, it is difficult to obtain the Menard modulus directly from the FDP tests, because both elastic and plastic behaviours are involved. It is complex to evaluate the

effects of soil disturbance under such conditions. Hence, the approach of the correlation relationship from the SBP tests was used.

The Menard modulus G_{M_FDP} for the FDP test can be estimated from the shear modulus G_{FDP} , using the correlation result from the self-boring pressuremeter tests. The ratio, G_{M_SBP}/G_{ur_SBP} , the equivalent Menard shear modulus to the elastic shear modulus, was obtained from the self-boring pressuremeter tests. Since the shear modulus G_{FDP} was comparable to the shear modulus G_{SBP} , values of G_{M_FDP} can be obtained the ratio of G_{M_SBP}/G_{ur_SBP} .

Almost no case records are available outside France of pressuremeter applications in settlement calculation. Two case studies were presented. These studies is useful in understanding several aspects. First, the direct use of the elastic modulus would lead to under-estimation of the settlement. The values of elastic shear modulus were applied in both cases in settlement calculations. The resulted settlement values were significantly lower than those of the final settlements. In contrast, the use of the Menard modulus G_{M_FDP} from the FDP tests led to good settlement estimations.

Secondly, it was demonstrated that the equation proposed by Menard & Rousseau is not suitable for raft footings, but should be used for individual isolated footings. The results from the Bank of Montreal building exhibited that the settlement was seriously underestimated by this equation. This equation is applicable in two dimensional stress conditions, while for raft foundation or embankment, the loading processes can be best approximated by one dimensional stress conditions.

Besides the Menard modulus, three parameter were utilized in the settlement calculations. They were the Menard's α factor, the drained Poisson's ratio and the coefficient β . Each of these factors had their effects on the final settlements calculated.

The Menard's α factor is an empirical factor. Its value depends on the soil type and the ratio of E_M/P_L . With clayey soils, α equals to 1 for overconsolidated condition (E_M/P_L

> 16) and 2/3 for normally consolidated condition ($E_M/P_L = 9-16$). Usually for overconsolidated soils, larger load can be applied, hence, $\alpha=1$ to account for larger settlements. The value of α equal to 1.0 was used for the Bank of Montreal building, 0.67 to 1.0 were used for the embankment.

The coefficient β tends to take into account the effect of the pre consolidation pressure. If the safety factor is 3, the bearing pressure is likely to be close to or smaller than the preconsolidation pressure, and β is 1 in this case. If the safety factor is less than 3, the bearing pressure is likely to exceed the preconsolidation pressure, hence, the value of β increases accordingly. The safety factor for the Bank of Montreal building was 3, and 1 was used for the value of β (Equation 7.6). The safety factor was 2 for the embankment with a height of 3.4 m. Therefore, the value of β was taken as 4/3.

The choice of the drained Poisson's ratio value affects the final settlement through Menard modulus, as shown in equation 7.5. The Poisson's ratio for both cases was taken as 0.25, which was close to the drained test result described in section 5.4.2. The effect of Poisson's ratio is not significant. For silty clay soils, the reasonable range of the Poisson's ratio can be considered as from 0.15 to 0.25. According to equation 7.5, the resulting difference of the Menard modulus is only 8%. The corresponding difference of the settlement is 8% as well. For the Bank of Montreal building, the variation of the final settlement because of this was only 4 mm. For the embankment, the variation was 13 mm. These variations were very insignificant in comparison with the calculated final settlements of 51 mm and 255 mm.

Chapter 8

SUMMARY AND CONCLUSIONS

8.1 SUMMARY

Silty soils are widely distributed all over the world. Usually, they are found in coastal areas. With the prominent increase of offshore investigation for natural resources and new land development along the coasts, it is essential to be able to obtain reliable geotechnical data for these silty soils.

Several cases in the literature were presented to demonstrate the difficulties in obtaining good quality samples in silty soils. Also, the literature shows that when soil samples are taken, calculations based on the laboratory consolidation tests significantly overestimate the settlements. The overestimation ratio could be as high as seven times. In-situ test seems to be the alternative for obtaining reliable soil data.

The development and interpretation of several in-situ testing methods, including the dilatometer test, cone penetration test and pressuremeter test were briefly reviewed. The procedures of settlement calculation from the consolidation test results and the pressuremeter test results were presented as well.

The Pressuremeter has great potential in obtaining basic soil parameters under well defined and controlled boundary conditions. With the use of the self-boring pressuremeter, the disturbance effect can be reduced substantially and good quality tests can be performed. However, the use of the self-boring pressuremeter is time consuming and expensive. In terms of practical engineering application, the full displacement pressuremeter has many advantages over the self-boring pressuremeter.

The installation effects of the full displacement pressuremeter probe were examined in detail. When the probe is pushed into the ground, the surrounding soil is displaced and disturbed. During this process the soil is being expanded cylindrically. Together with the subsequent expansion of the probe, the cylindrical expansion during the installation can be considered as part of the whole cylindrical expansion process. The evidence illustrating this phenomena is the measured initial pressure after the probe installation being higher than the in-situ initial horizontal stress.

In the study of the installation effects on the interpretation of the shear modulus, the change of the modulus magnitude at different positions around the probe was assumed, based on a model test data and a qualitative analysis. The generalized Lamé's solution is used for solving this cylindrical annulus problem with n layers.

Simple shear tests were performed to determine the parameter, α , which is needed in the numerical analysis. A new controlling system was developed for the simple shear apparatus. The new system was controlled by a computer program written in the ASYST language. Thus, prescribed directions of principal stresses were able to be achieved and maintained during testing.

The disturbance index for calculating the shear modulus was obtained from the numerical analysis. Values of shear modulus were calculated from FDP tests conducted at three sites based on the analysis results. The results of the shear modulus calculated by the proposed methods were compared to values calculated by Houlsby and Withers' method (1988). It was shown that Houlsby and Withers' method does not always lead to repeatable results for tests performed on a uniform site. For all the three sites, the new method led to repeatable results of shear moduli. When the Houlsby and Withers's method does yield repeatable values, the values of shear modulus from their method agree with those obtained from the new method.

Five methods of calculating undrained strength were used for the interpretation of

the FDP tests. Computer programs were developed for the analysis using these five methods. The limit pressure method and Lemee's method (1973) lead to consistent and repeatable results of undrained strength for all sites. Houlsby and Withers' method gives repeatable results of undrained strength for the soft clay at the Sackville site, and these values agreed well with those from the limit pressure and Lemee's method. However, Houlsby and Withers' method did not always yield consistent and repeatable results as shown in the case of test nrc1 and nrc2 for the NRC site.

Four possible types of strain - pressure or volume - pressure relationships were assumed, based on the examination of installation of the FDP probe. These four types of strain - pressure curves were obtained from all the test data by a curve fitting method. Three methods for Su_{PI} , Su_{Lemee} and Su_{hw} were applied to these curves to evaluate the undrained strength. The results of undrained strength show that Su_{hw} was not affected by the assumptions proposed. Su_{Lemee} leads to scattered results of undrained strength. The results obtained from the four assumptions covered all the possible forms of volume - pressure relationships for a FDP test, although the exact form was not known. The resulting values of undrained strength from these volume - pressure relationships by the limit pressure method agreed with one another and agreed with the results from the unmodified volume - pressure relationships. This indicates that the effects of installation of FDP on the undrained strength are small, when the limit pressure method is used. For soft clay, the method proposed by Houlsby and Withers can be used as a good reference for comparison with values obtained from the limit pressure method.

Soil samples were taken from the site of the Sackville site. Two types of samples were obtained. One was the regular tube sample of a diameter of 7.5 cm. The other type was the large sample with a sample diameter of 20 cm. A series of laboratory tests, including oedometer test, the triaxial test and the modified simple shear test were performed to evaluate the compressibility and undrained shear strength.

Values of the compression index and the constrained modulus were obtained from the oedometer tests. From the triaxial tests, values of undrained strength and undrained Young's modulus were estimated. Undrained shear strength and shear modulus were computed from the curves of shear strain - shear stress, from the simple shear tests.

In order to be able to compare the compressibility measurements of these tests, the values of constrained modulus and undrained Young's modulus were converted to shear modulus. It was shown that the values of converted shear modulus from the oedometer tests were much smaller than those from the FDP tests (approximately 10 times smaller). As well, modulus from the triaxial tests were smaller than those from the FDP tests.

One exception of compressibility measurement is the simple shear test. The average values of shear modulus from the simple shear tests were slightly smaller than those from the FDP tests. It is believed that it is the high loading rate of the simple shear test that causes the higher values of the shear modulus.

For the values of the undrained strength, the comparison of the FDP results to the simple shear tests and triaxial tests showed good agreement. At a depth of about 6.5 m, the difference of the average value from simple shear tests fell within 10% of the averaged value from the FDP tests. For the triaxial tests, with one exception at a depth of 0.6 m, all the values of undrained strength were comparable to those from the FDP tests.

Besides the full displacement pressuremeter test, other in-situ tests were conducted, such as the piezocone tests and self-boring pressuremeter tests. As well, data of the field vane tests on these sites were available.

The tip resistance, the friction ratio and the pore pressure measured from the piezocone tests were used together for soil classifications. Also, the data of the piezocone were used in examining the site uniformity, which serves as the basis for comparison studies.

Parallel tests were conducted at two sites. Results showed that the repeatability of the measured shear modulus was excellent for both sites. As well, close results were found for the undrained shear strength values from these parallel tests.

Results of shear modulus from the self-boring pressuremeter tests were obtained at the NRC site and the Sackville site. They compared very well to those from the full displacement pressuremeter tests. Values of shear modulus from the self-boring pressuremeter tests at downtown Fredericton were compared to those measured with the FDP tests at the Diplomatic Hotel site. Very close agreement was found between values of these two test methods.

The undrained strength from the self-boring pressuremeter tests were calculated by two methods. The first method was the limit pressure method. The other one was the pressure : $\log_e(\Delta V/V)$ slope method (log-slope). It was found that the log-slope method gave values of strength which were about twice those of the limit pressure method. Only values of undrained strength from the self-boring pressuremeter tests calculated by the limit pressure method were compared with those from the FDP tests. Excellent agreement was found for all three sites between the SBP and the FDP tests.

The method proposed by Konrad and Law (1987) for calculating undrained strength from the piezocone test was used in this study. At all three sites, values of the undrained strength from the piezocone tests agreed with those from the FDP tests very well.

Data of the field vane tests were available at both the Sackville and the NRC sites. Values of the vane tests at downtown Fredericton were used for comparison with data at the Diplomatic Hotel site. At the NRC site, values from the vane test were slightly higher than those from the FDP tests. Generally, values of undrained strength from the FDP values all agreed very well with those measured from the field vane tests.

Two cases of settlement calculations are presented in this study. The first one is a multistorey building founded on a raft foundation. The underlying subsoil was 30 m thick

silty clay. The second case was a test embankment constructed on about 20 m of silty clay.

For the multistorey building, two methods were used for the settlement calculation based on the measured Menard modulus from FDP tests. The first method used was proposed by Baguelin et al., which is suitable for raft foundations or embankments. The estimated settlement was 51 mm. The second method is the French method proposed by Menard and Rousseau. The settlement estimated from this method was 13.7 mm. The values of elastic modulus were also used for calculation by the first method. The resulted settlement value was 20 mm.

The actual data indicated that the settlement right after the completion of the construction was 16 mm. The final settlement four years after the construction was about 40 mm. The values of settlement calculated from two laboratory consolidation tests were 104 mm and 163 mm.

In comparison, the estimated settlement of the multistorey building based on the Menard modulus from the FDP data closely agreed with the measured settlement. The settlement calculated from laboratory consolidation tests was several times higher than the observed value. The value of settlement calculated from the elastic modulus was much lower than the actual settlement.

For the test embankment, the same methods as for the Bank of Montreal building were used for settlement calculation for an embankment height of 3.4 m, which is reasonably far from that of a general failure. The value of settlement estimated was 255 mm, based on the Menard modulus from FDP tests. The settlement value was 140 mm right after the embankment reached 3.4 m height with the projected final settlement being 250 mm at this height. The settlement calculated from the laboratory consolidation tests was 868 mm. In comparison, the settlement estimated from the FDP tests agreed closely with the projected final settlement. In contrast, the value calculated from the

consolidation tests grossly overestimated the projected final settlement, by a ratio of about 3.5 times. The settlement calculated directly from the values of elastic modulus was 122 mm, which was less than half of the projected final settlement.

8.2 CONCLUSIONS

The following conclusions are drawn from this study:

1. For silty soils, the soil behaviour cannot be measured reliably by laboratory testing. As well, the settlements may be seriously overestimated based on the laboratory test results as illustrated by the two case studies reported in this thesis.
2. The in-situ test program demonstrates that the full displacement pressuremeter is suitable and has many advantages over the self-boring pressuremeter for daily engineering practice.
3. At three silty clay sites, the values of shear modulus obtained by the proposed method from the FDP tests agree well with those from the SBP tests.
4. Detailed examination of all the methods for interpreting the FDP test results to obtain the undrained strength clearly indicates that the limit pressure method yields the best results.
5. Excellent agreement has been found in the comparisons of the undrained strength results from the FDP tests with those from the other in-situ tests, including the self-boring pressuremeter, the piezocone and the field vane tests.
6. The case studies demonstrate that the elastic shear modulus from the pressuremeter test cannot be directly used for the estimation of the final settlement. However, the Menard modulus obtained from the FDP test using the proposed method can lead to good estimation of the final settlement.

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Appendix A

FULL DISPLACEMENT PRESSUREMETER TEST

1 INTRODUCTION

A full displacement pressuremeter (FDP) is a pressuremeter device fitted with a conical tip to ease the pushing of the pressuremeter probe into the soil. Its name reflects the method of how it is installed. Unlike the Menard or the self-boring pressuremeter devices, no borehole is formed for the test. The device is pushed into the ground in the same way as pushing a static cone device. Actually a static cone penetrometer can be combined with the full displacement pressuremeter, which leads to another name called cone pressuremeter device. The particular FDP device described in this chapter is not combined with a static cone penetrometer. There is, however, no difference in the methods for interpretations for both cone pressuremeter test and the full displacement pressuremeter test. The principle and components of the device and the standard testing

procedure will be briefly described first. Then discussions will be given in terms of improving the testing procedure.

2 FULL DISPLACEMENT PRESSUREMETER DEVICE

The FDP device used in this study is composed of two parts, a 60° conical tip which is similar to the standard static penetrometer cone, and a pressuremeter probe. The device is shown in Figure 1.

The probe consists of a cadmium plated steel tube on which is mounted a rubber

Figure 1 The FDP Probe

membrane with a protective inflatable metallic sheath. The membrane with the sheath is held in place using two tapered metal rings and two lock nuts. The probe is fitted with quick connectors at both ends. One end accepts the tubing leading from the pressure-volume control unit and the other, the saturation tubing. The conical tip is screwed onto the base of the probe. The top of the probe is fitted with an adaptor which accepts EX drill rod. The total length of the probe is 580 mm and its maximum deflected diameter is 35.2 mm which is slightly less than the diameter of a standard static penetrometer cone.

The device is designed to enable the execution of pressuremeter tests up to 2500 kPa using light weight drilling equipment on sites where the use of heavy drill rigs is impractical or is economically prohibitive.

Figure 2 shows the schematic diagram for the whole unit. The control unit for the full displacement pressuremeter contains the following components:

- a piston cylinder assembly;
- a -100 to +2500 kPa pressure gauge;
- a counter (for volume change) graduated in 1/10 of cm³ and,
- the valves and tubing fittings.

These components are housed within a fibreglass case which is supported by a detachable aluminum tripod. A tubing is used for linking the probe and the control unit. The outside diameter for the tubing is 6.35 mm and the inside diameter is 2 mm. The upper end is terminated by a 15 mm O.D. quick connector and the lower end by an 18 mm O.D. quick connector.

3 PREPARATION BEFORE TESTING

The unit must be filled with clean water and fully saturated first. A one litre

Figure 2 Schematic diagram for FDP

capacity container is required for this operation. If the temperature is below freezing point or the risk of freezing exists, the water must be replaced by anti-freeze mixture comprised of 50% water and 50% ethylene glycol.

The unit can be saturated by means of the following steps:

1. Check that the pressure gauge reads zero;
2. Fill up the cylinder and the controlling unit by rotate the crank-handle in the deflate direction;
3. Remove any air trapped in the cylinder by transferring 100 cm³ from the cylinder back into the container;
4. Refill the cylinder;
5. Fill the tube and the probe by repeating:
 Rotate the crank-handle to inflating direction in testing position;
 Refill the cylinder and repeat above action;
6. Close the end of the probe and screw the conical tip onto the probe;

The degree of saturation needs to be checked after the above steps. Place valves #3 and #4 to "Test" position (refer to Figure 2). Pressurize the cylinder until the pressure gauge shows a value of 2500 kPa. Verify that the volume necessary for this operation is less than 5 cm³ and also that after about two minutes the pressure stabilizes between 2000 and 2500 kPa. If the volume is more than 5 cm³, the saturation of the system is inadequate and saturation process is to be repeated.

When the probe is dilated under atmospheric pressure, the pressure reading indicates the inertia of the membrane and sheath. Also there will be certain loss of volume in the tube, control system and the probe. For correct readings, both aspects have to be accounted for by proper calibrations before test.

The pressure calibration can be conducted after the whole system is saturated. The probe can be placed vertically on the ground next to the control system. With the valves

Figure 3 Example of pressure calibration curve

#3 and #4 on "Test" position, the probe is inflated and deflated five times by injection of 90 cm³ in order to kneed the sheath. The probe is then inflated up to 90 cm³ at an injection speed of the order of 1/3 cm³/second. The pressures are noted for each increment of 5 cm³ injected. The pressures that have been read are then corrected by taking into account the head of water between the pressure gauge and the centre of the probe. The pressure correction curve is then drawn based on the injected volumes. An example is given in Figure 3.

Volume calibration consists of two steps. In the first step, the probe is placed in a stiff wall tube with an interior diameter of around 34 mm. The probe inserted in the tube is then inflated by injecting water at a rate of 1/3 cm³/second until the pressure reaches 2000 kPa, then deflated by bringing the volume counter back to zero. The pressures should be noted for every 5 cm³ injected. The second step needs disconnection of the probe and closing of the free end of the tubing. The pressure is increased progressively up to 2000 kPa, then the volume counter is brought back to zero. The pressure

Figure 4 Example of volume calibration curve

corresponding to each cm^3 injected is noted. Example data of these two steps are shown in Figure 4.

4 PROBE INSERTION AND TEST EXECUTION

The FDP probe is designed for positioning in place by pushing or gentle hammering. The probe is connected to hollow drill rods of EW size, either steel or aluminum, with an external diameter of 34.9 mm. In dense soils, the probe could be damaged by the driving. In such case, it is recommended that a hole be pre-drilled with a diameter of 35 mm.

Once the probe has been pushed to the desired depth for testing, set valves #3 and #4 to "Test" position, testing can then be carried out in increments of equal volumes. The increment of increasing volume can be set to 5 cm^3 and the corresponding pressure should be noted 30 seconds later. The maximum volume injected is 90 cm^3 . The unload-

Figure 4 Example of volume calibration curve

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4 PROBE INSERTION AND TEST EXECUTION

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Figure 5 Example of recorded data

reload cycle can be performed between steps of volume. The whole test takes about 10 minutes. It is also possible to use the method of constant loading rate. The recommended speed is $1/3 \text{ cm}^3/\text{second}$ which is equivalent to 1 crank revolution in 9 seconds. An example of testing data are shown in Figure 5.

5 DISCUSSIONS ON CALIBRATION AND VOLUME CORRECTION

It is essential to perform accurate calibrations in order to obtain corrected pressure-volume curve. The values of pressure and volume from calibration serve as reference points and the interpretation of parameters critically depends on these reference points. To ensure accurate calibration, one must understand the reason for every step in the calibration and the effects of calibration on the interpretation of each parameter. Also for different types of soil, the degree of sensitivity of parameters to pressure or volume correction is different. So that special care should be given to the more sensitive aspect

d. d to b was drawn by hand and the volume correction curve C was obtained by moving a-b-c parallel to the origin.

Figure 7 Example of negative unload-reload curve

This standard procedure was applied in the data correction. Unusual results were found in the corrected data. It was found in some of the data sets that the slope of the unloading-reloading curve is negative. One example is shown in Figure 7 and Figure 8. Figure 7 shows the corrected data following the standard procedure. Figure 8 shows the zoomed-in unload-reload curve.

By examining the volume calibration curves in Figure 6, one can see that for curve A the linear portion starts at point b when the pressure is around 700 to 800 kPa. The system compliance only affects curve A after point b, which is the linear portion. On the other hand, it is found from the recorded test data that the pressures applied in the tests were mostly under 800 kPa. Combining these, we know that the system compliance did not affect the test data. In another word, the applied pressure was not high enough to cause the deformation of the rubber membrane.

Figure 8 Zoomed graph of negative unload-reload curve

Based on these observations, it is concluded that only the tube and control unit flexibility need to be taken into consideration for volume correction. The tube and control unit flexibility is shown by curve B in Figure 6.

As the slope of unload-reload will be used to calculate the shear modulus, which is the most important parameter from pressuremeter test, the above considerations for volume correction is especially meaningful for obtaining reliable results.

The above discussion also helps us to determine in which stage the unloading and reloading should be carried out, in order to minimize the possible error from volume correction. From curve A in Figure 6, we can see that the increase of volume loss in the tube and the control unit becomes very small after certain pressure (say 400 kPa). If the unload-reload is conducted after that pressure, then when the slope is being calculated, the possible error from volume correction could be minimized.

6 RAW DATA PROCESSING BY CURVE FITTING METHOD

A curve fitting method has been introduced to process pressuremeter data (Huang et al., 1986; Arnold, 1981). The main reason for using a curve fitting method is to offer an alternative method for analysing pressuremeter data instead of using a graphical procedure. It was also used for analysing pressuremeter data by an empirical method (Arnold, 1981). A curve fitting method was used in this study for processing the recorded data from the full displacement pressuremeter tests. The intention is to reduce the fluctuation of the data and offer an easier means for further manipulation of the data. Also the curve fitting technique is combined in a new method proposed later in this study to evaluate the pressure-strain relationship from full displacement pressuremeter tests.

The simplex algorithm is used for the curve fitting method (Caceci & Cacheris 1984; Huang et al. 1986). It has advantages such as: no divergence is possible; no derivatives or numerical differentiation is involved which avoids rounding-off errors and allows the handling of non-continuous functions.

Consider a set of N data points (x_i, y_i) to be fitted by a series of functions $f_1(x), f_2(x) \dots f_n(x)$. The purpose of optimization is to determine a set of function coefficients and x values where f_i intersects f_{i+1} which correspond to the smallest sum of the squares of residuals SSR , defined as

$$SSR = \sum_{i=1}^{n1} W_1(f_1(x_i) - y_i)^2 + \sum_{i=n1+1}^{n2} W_2(f_2(x_i) - y_i)^2 \dots + \sum_{i=n_{k-1}+1}^N W_k(f_k(x_i) - y_i)^2 \quad (1)$$

where W_k is a weight (optional) assigned to each data point and N is the total number of data points.

The basic concept is to consider each set of variables as a vector in a space of dimension M (M is the total number of variables to be optimized, i.e. coefficients of the

functions and the x values as stated earlier). The vector is called a vertex and a simplex is a geometrical figure which consists of $M+1$ vertexes.

The curve fitting method can handle any type of nonlinear functions. For the full displacement pressuremeter test, the hyperbolic equation (Arnold, 1981) is used to describe the stress-strain relationship of recorded data:

$$p = \sigma_{ho} + \frac{\varepsilon_c}{a + b\varepsilon_c} \quad (2)$$

where:

p = loading pressure,

ε_c = cavity strain,

a, b = constants,

σ_{ho} = initial horizontal stress.

As the primary data are volume and pressure instead of strain and pressure, the volume has to be converted to strain. By definition, strain is calculated as:

$$\varepsilon = \frac{\rho - \rho_0}{\rho_0} \quad (3)$$

where:

ρ = radial displacement,

ρ_0 = initial probe radius.

Since for unit length of cylindrical column:

$$\rho = \sqrt{\frac{V}{\pi}} \quad (4)$$

where V = the volume. From the above two equations, we have:

$$z = \sqrt{1 + \frac{\Delta V}{V_0}} - 1 \quad (5)$$

where ΔV = the volume change reading, V_0 = the initial volume of the probe.

A computer program was developed in C language for the curve fitting method using the simplex algorithm. It is written and compiled in Turbo C++ for IBM PC compatibles. All the expressions were standard ANSI C except for the functions regarding screen graphic output. Although the hyperbolic equation is used as the analytical function for curve fitting, with slight modification, this program can be used for any type of functions, and only the section which calculates the sum of squared residuals need be changed.

The input parameters for the simplex algorithm needs to be stored in a data file. This file contains the following information:

1. Maximum number of iterations
2. Initial values of parameters to be evaluated
3. Values of increment for each parameter
4. The maximum errors allowed for each parameter

Figure 9 shows the graph of recorded test data at depth of 3.5 m. Figure 10 shows the resulted graph after the pressure and volume corrections. It can be seen that the data fluctuation becomes more severe after pressure and volume corrections. Figure 11 shows the recorded data after curve fitting and Figure 12 shows the resulted graph after pressure and volume corrections based on Figure 11. These results indicate that the data fluctuation problem is resolved by the method of curve fitting. Results of the all the test data after curve fitting showed that the curve fitting method served the intended purposes very well.

Figure 9 Recorded testing data of FDP

Figure 10 After pressure and volume correction without curve fitting.

Figure 11 Recorded data of FDP after curve fitting

Figure 12 After pressure and volume correction with curve fitting first.

END

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FIN