PRESSUREMETER TESTING IN MIOCENE STIFF CLAYS

by

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(ABSTRACT)

In geotechnical engineering, it has proven difficult to obtain reliable soil parameters for stiff clays. Laboratory testing results are often scattering due to the fissures and slickensides in these soils. Alternatively, in situ techniques offer a means to test the soils in place. This study focuses on in situ testing in Miocene stiff clays using an advanced nine strain arm self-boring pressuremeter (SBPM). This device was used to test the soils in both a self-boring and a simpler, non-boring mode (pre-bored or PBPM tests). The Miocene stiff clay was unique in that was sensitive and lacked fissures and slickensides. The pressuremeter results could be compared to a range of other tests previously performed at the site.

It is concluded that the SBPM provides accurate values of modulus and strength. Minor soil disturbance was found to have little effect when it could be anticipated. The simpler PBPM tests were not successful because of disturbance caused by borehole preparation.
Shapes of the pressuremeter membrane expansion are accessible by using the new nine strain arms pressuremeter. It is concluded that the assumption of expanding under plane strain condition is reasonable. Also, any initial minor soil disturbance in SBPM tests does not affect the magnitudes of membrane expansion at final loading stages.

The issue of anisotropic in situ lateral stress fields is studied analytically. Finite element methods are applied to assess the effects of anisotropic in situ stresses on the interpreted in situ stresses from the SBPM test. It is concluded that the pressuremeter lift-off measurements would exaggerate the degree of in situ stress anisotropy due to stress concentration effects. Procedures for interpreting actual anisotropic in situ stresses using the lift-off measurements are proposed.
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CHAPTER I  INTRODUCTION

Design for geotechnical problems in stiff clays follows largely the same process as that used for other soils. Where loading is involved, strength and stiffness values for the soil have to be determined, with the latter parameter usually defined in terms of one or more modulus values. Determination of strength and stiffness is always challenging, but special difficulties exist for stiff clays in that these materials commonly contain ubiquitous fissures and slickensides. The presence of such features often produces a wide scatter in laboratory test results. Because they can dominate the behavior of the small specimens that are typically used in the tests. The problems involved in choosing design parameters for stiff clays have long been recognized. In the end, the only solution for conventional practice is to use conservative parameters based on local experience.

In situ testing offers an alternative to laboratory testing for deriving soil parameters. The most popular types of in situ tests are the standard penetration test (SPT) and the cone penetration test (CPT), but these tools are primarily used for soil classification and as indirect means of determining soil strength. They are not effective in obtaining the modulus or in situ stress values, parameters often important in design in stiff clays. As a result, some geotechnical engineers advocate use of the pressuremeter to help determining parameters for stiff clays. The pressuremeter is an in situ testing device that involves expansion of a flexible membrane against the sides of a borehole. During the test, the pressure applied to the borehole and the amount of cavity expansion are recorded, and soil strength and stiffness are back-calculated from cavity expansion theory. In theory, the pressuremeter test is well-suited for solving problems created by the presence of fissures and slickensides in stiff clays, since the test is performed while the soil is fully confined and the probe is large enough to stress a broad area of soil mass. However, in practice, concerns about the test have limited its potential.
The conventional approach to pressuremeter testing involves drilling a hole in the ground, removing the drilling equipment, inserting a pressuremeter into the hole, and performing the test. This method is commonly known as the Menard pressuremeter (MPM) test after Louis Menard, the individual who invented the modern version of this approach to pressuremeter testing. The MPM test is widely used in France (Gambin, 1990), where it is integrated into geotechnical practice. It is much less used in the U.S. because of skepticism about the influence of the disturbance introduced by the drilling process and the reduction of stress on the sides of the borehole as the drilling equipment is removed from the hole. In response to these concerns, in 1972 a self-boring pressuremeter (SBPM) was developed simultaneously in France and England. This device drills itself into the ground using a cutting process, and hence theoretically minimizes disturbance as well as maintains contact with the sides of the hole at all times. Experience has shown that achieving the desired result in the self-boring process is difficult, and that self-boring in stiff clays is not cost-effective for most geotechnical projects. However, experience shows that modulus values from properly conducted SBPM tests are almost an order of magnitude higher than those from laboratory tests (Clough et al., 1990). Such improvements in modulus values can be important to the cost of a project since design in stiff clays is often dictated by predicted deformations, not conditions that might occur at failure where strength governs.

The objectives of this study address both fundamental and practical issues related to using the pressuremeter for determining key design parameters for stiff clays. Principal goals include:

1. Assessment of the effects of an anisotropic in-situ stress field on the interpreted in situ stresses from the pressuremeter test (recent pressuremeter tests in stiff clays have indicated the presence of an anisotropic stress field).
2. Performance of SBPM and pre-bored pressuremeter tests. So results from alternative forms of the pressuremeter test can be compared to themselves and to other types of tests, and determine the accuracy of the results.

3. Study of the shape of the expanding pressuremeter membrane in the presence of a stiff clay to understand how closely the membrane follows the commonly assumed cylindrical shape.

4. Sorting out the relative degree of sensitivity and insensitivity of soil parameters to disturbance and other test-dependent factors in the pressuremeter test.

5. Assessing alternative procedures for performing the pressuremeter test that might be used to make it more efficient, while at the same time providing good quality test results for geotechnical practice.

The first of these objectives was primarily focused on a theoretical study using finite element analyses. The remainder involved an experimental effort using a new pressuremeter that was instrumented to allow insight into the fundamentals of pressuremeter loading in the field.

The issue of the effects of an in situ anisotropic stress field was first raised in the early 1980's, when an SBPM was equipped so that it could measure the response of individual points around the membrane. Subsequently, several investigators reported measurements indicating non-uniform lateral stress fields. While it is now understood that such conditions exist, the issue of how this condition might affect the interpretation of the pressuremeter is not well understood. This subject was of particular importance to this work in that the field testing effort was to be accomplished in a soil deposit where some test data suggested the soil mass to have anisotropic conditions.
The experimental phase of this test program focused on a unique stiff clay found in the downtown Richmond, Virginia area. The soil, known locally as the Miocene clay, serves as the bearing layer for most deep foundation projects in Richmond. It is an unusual stiff clay in that it is sensitive, and it possesses few, and very widely spaced slickensides. The reason for the lack of slickensides and fissures is not fully understood, but this characteristic is important in that laboratory tests can be performed on this soil without problems associated with wide data scatter seen in other stiff clays. Thus, a sound laboratory data base can be obtained for comparison with in situ tests. In addition to the lack of fissures and slickensides, the Miocene clay provides a useful test medium in that a number of test programs have been performed, including several research investigations (Mayu, 1987, and Mullen, 1991). Extensive results are available from high quality laboratory tests and a variety of in situ tests, including self-bored pressuremeter tests, pressuremeter tests in holes opened by driving a split-spoon sampler, CPT, SPT, and dilatometer tests (DMT).

The field investigation for this research utilized a new type of SBPM which was designed with nine strain arms to allow the membrane shape to be monitored during all phases of the test. The probe also included an electronic system which provided near-continuous monitoring of the loading and response to the loading. Tests were performed in both a self-boring mode and in holes opened by pushing thin-walled Shelby tubes. These tests complemented those which had been conducted earlier, allowing for a comparison of data from a full range of testing methods.

In addition to the introduction, this document includes five chapters and one appendix. Chapter II presents a background review of pressuremeter testing and the procedures for reducing the data from the test. Information about measurements of membrane shape and the available test data on the Miocene clay
are also provided. The study of the effects of anisotropic stress fields using the finite element method is discussed in Chapter III. New procedures to interpret in situ lateral stresses in both isotropic and anisotropic in situ stress fields are proposed. Chapter IV covers the pressuremeter testing program and the characteristics of the test site used for the field tests. Chapter V presents the results of the pressuremeter testing program including membrane shapes, the soil parameters, and effects of soil disturbance and methods of testing. The results are also compared to those available from other testing techniques. Finally, Chapter VI contains the summary and conclusions of this study. Documentation of the pressuremeter testing data is provided in the Appendix.
CHAPTER II  BACKGROUND

2.1 General Characteristics of Stiff Clays

A clay with a strength greater than 49 KN/M² (7 psi) is classified as stiff. Stiff clay has a high strength in large part from the effects of overconsolidation. This occurs through the process of loading the clay to one effective stress, and subsequently unloading it to a lower stress. The initial loading compresses the clay to a lower void ratio. When unloading, the clay does not rebound to its original condition because of a hysteretic process. At the lower void ratio generated by overconsolidation, the soil has a strength higher than what would be commensurate with the present overburden conditions. If the void ratio is low enough, the clay will develop adequate strength to be classified as stiff.

In nature, the overconsolidation process normally involves several cycles. As the process continues, it typically leads to development of fissures and slickensides in the clay. Fissures and slickensides can have a major impact on behavior in geotechnical engineering problems such as open cuts and natural slopes. In these cases, the soil mass is unconfined and the fissures or slickensides provide avenues for local failure. However, in situations where the soil mass remains confined during loading, the effects of fissures and slickensides are of marginal influence. This latter type of problem includes pier and pile foundations and positively supported walls such as pre-stressed anchored walls. Geotechnical testing programs for stiff clays ideally will reflect the nature of the problem for which the results will be used.

In geotechnical engineering, it is common practice to conduct laboratory tests on small cylindrical specimens to obtain engineering parameters for clays. The
specimens are usually available from drilling and sampling procedures. The laboratory tests typically involve applying compression to induce a shear failure through the body of the specimen. If a fissure intersects the shear plane, then the testing results may be different from a test where the fissures have no influence on the test outcome. Testing results of stiff clays are often scattered due to the presence of fissures and slickensides (Lo, 1970). Therefore, laboratory tests for stiff fissured clays may have little value, particularly for problems of loading the soil in a confined state.

An alternative for testing stiff clays is in situ testing. Testing the clay in place, without altering the confined environment, keeps the influence of fissures or slickensides in proper perspective. Among many types of in situ tests available, pressuremeter testing provides an attractive tool for defining design parameters of stiff clays under the confined environment.

2.2 Pressuremeter testing

Although Kogler (1937) introduced an early version of the pressuremeter, it was Menard (1957) who brought this technology into conventional practice. As shown in Figure 2.1, the Menard pressuremeter (MPM) is a cylindrical probe with a length to diameter ratio about 6.5 (including guard cells). The central portion of the probe consists of a flexible membrane that can be expanded against the sides of a borehole. The basic procedure of an MPM test involves inserting the probe into a prebored hole, hydraulically pressurizing the measuring cell, and expanding the membrane against the soil. The intent of the pressuremeter testing is to create an environment similar to an infinitely long cylinder expanding in a semi-infinite medium.
FIGURE 2.1  Menard pressuremeter (after Mitchell et al., 1978)
2.2.1 Menard Pressuremeter Testing

The MPM probe consists of a measuring cell and two guard cells. In the basic approach of an MPM test, the probe is inserted into a prebored hole. Then, using water to pressurize the measuring cell and expand the cell membrane against the surrounding soils, the expansion process is monitored by measuring the applied pressure and the amount of water flowing into the probe. The MPM uses two pressurized guard cells immediately above and below the expanding membrane. The guard cells function as stabilizing devices within the borehole and help to develop radial plane strain conditions at mid-height of the probe. The length to diameter ratio of the measuring cell is about three to one.

An MPM testing result as shown in Figure 2.2 is typically called the pressuremeter curve. The curve presents the soil response in terms of the applied pressure and the volumetric strain. Typically, the pressuremeter curve exhibits three phases of behavior: (1) Phase I - reloading of the disturbed soil; (2) Phase II - linear, pseudo-elastic compression of the soil, where unload-reload cycles also may be performed; and (3) Phase III - a strongly nonlinear response where the probe expands with little increase in pressure. Engineering parameters that can be derived from the pressuremeter curve include:

1. In situ lateral pressure, \( P_o \), taken as the pressure corresponding to the kink in the curve between the phases I and II.
2. Pressuremeter modulus, \( E_p \), determined from the straight line portion in the phase II.
3. Limit pressure, \( P_L \), taken as the pressure at which the volume increase equals the initial volume of the pre-bored borehole.
4. Undrained shear strength, \( S_u \), interpreted using semi-empirical methods or cavity expansion theories.
FIGURE 2.2  Menard pressuremeter test pressure-volume curve
These parameters are often applied in design in an empirical or semi-empirical way. Figure 2.3 shows an example of a correlation between parameters derived from an MPM test and the information needed for spread footing design. The correlation was developed by plotting the pressuremeter parameters against results from observed prototype and model test behavior.

Recent publications of Briaud (1992) and Gambin (1990) provide detailed discussions of the development of the MPM equipment and its application to geotechnical problems. In France, where Menard exerted considerable influence on geotechnical practice, the MPM approach has been widely accepted (Baguelin et al., 1986; Gambin, 1990). However, in the United States, there have limited applications in selected types of problems and geographic areas (Clough et al., 1990). Reluctance in adopting the MPM approach may be attributed to:

1. Concern over the effect of soil disturbance induced by preboring the testing hole for the MPM.
2. Lack of understanding of the basis for the MPM design rules.
3. Growth of competitive technologies, e.g., cone penetration or dilatometer testing.

2.2.2 Alternative pressuremeter techniques

In an attempt to overcome the problems associated with soil disturbance during pre-boring, English and French teams simultaneously developed a self-boring idea for the pressuremeter in 1972. The English probe is called the Camkometer (Wroth and Hughes, 1973), while the French version is termed the PAF probe (Baguelin et al., 1972). Figures 2.4 and 2.5 depict the Camkometer and PAF devices respectively.
FIGURE 2.3 New design curve for spread footings (after Briaud, 1990)
FIGURE 2.4 Cambridge self-boring pressuremeter, Camkometer (after Arnold, 1983)
FIGURE 2.5  Self-boring pressuremeter; PAF-76 probe (after Jezequel and Lemehaute, 1984)
In a self-boring pressuremeter (SBPM) test, a downward thrusting force provided by a reaction system at the ground surface pushes the probe into the ground. Meanwhile, the soil enters the open face at the bottom of the probe through a sharp-edged cutting shoe. Immediately inside the cutting shoe, there is a rotating cutting blade, or chopper. The cutting blade chops the entering soil into small fragments or cuttings. To minimize soil disturbance, the cutting blade also exerts light pressure on the soil to help to prevent soil from moving into the shoe on its own. Drilling mud or water is jetted down from the open central annulus of the cutter rod to the shoe. This fluid carries the soil cuttings to the ground surface through the annulus between the drill rods and the cutter rods. Using this process, the probe self-bores itself to the test depth. If all goes according to the plan, the surrounding soils remain essentially undisturbed. Finally, the membrane is expanded and the applied pressure and the corresponding membrane movements are recorded. Besides the sensors for measuring pressure and membrane movements, other types of sensors, such as pore pressure, inclinometer, and temperature sensors can be incorporated into the SBPM.

Besides the self-boring process, the SBPM procedure also differs from the MPM procedure in several other ways. In all SBPM's, the expansion pressures and the membrane movements are monitored by using electronically instrumented pressure cell and strain arms. Depending upon the pressuremeter type and design, the number of strain arms ranges from two to nine. If included, other types of electronic sensors may be used. These types of instrumentation allow the use of a computer controlled data acquisition system to automate the data recording process.

The Camkometer type of SBPM has several special features that distinguish it from other SBPM's. An important one is the use of a rigid probe body formed from a stiff cylinder of bronze and surrounded by a membrane. During probe insertion, the membrane is pulled tight to the body by applying a vacuum inside the
probe. During testing, gas pressure is applied inside the probe to expand the membrane. The use of the rigid probe body is to help to maintain cylindrical membrane shape during testing.

Theoretically, the SBPM insertion introduces little disturbance to the surrounding soils. As a result, it has the potential of deriving more reliable soil parameters and in situ lateral stresses than the MPM does. Other advantages of the SBPM test are:

1. A microcomputer can electronically monitor and record the test data.
2. There are well-developed theories for derivation of soil parameters.
3. Assuming minimal soil disturbance, the test results will reflect effects of aging and stress-strain history of the soil deposit.
4. Direct measurement of in situ lateral stresses is possible.
5. The SBPM has a longer length to diameter ratio of the measuring probe than in the MPM, better simulating plane strain conditions.

While the SBPM offers advantages over the MPM, it remains of limited use in practice because the procedures required to perform the test are not efficient. To date, it has found most value in research to allow comparison to other testing procedures.

In an attempt to address the practical problems posed by the SBPM, investigators have developed other forms of pressuremeters. Examples include the OYO monocell probe (Figure 2.6), used in a prebored hole; and the full-displacement pressuremeter (Figure 2.7) and the stressprobe pressuremeter (Figure 2.8), which are inserted into the ground by pushing. Most of the new probes were developed in the late 1970's and early 1980's. However, none of them has yet found wide acceptance by the practicing community.
FIGURE 2.6  Schematic diagram of the OYO monocell probe (from Bergado and Khaleque, 1986)
FIGURE 2.7  Full displacement pressuremeter (from Withers et al., 1986)
FIGURE 2.8 Stressprobe (push-in) pressuremeter (from Fyffe et al., 1986)
Most recent improvements in pressuremeter technology have focused on electronic monitoring systems and data acquisition devices. For example, there are new SBPM's with nine strain arms and advanced downhole data acquisition systems for monitoring pressure-deformation behavior during the test (Clough et al., 1990). The advances in monitoring and data acquisition have improved the accuracy and efficiency of the SBPM test.

2.3 Interpretation Procedures for Pressuremeter Results

If the probe volume change is measured during a pressuremeter test, as in the MPM test, the pressuremeter curve presents the relationship between the applied pressure and the corresponding soil volumetric strain. If the membrane movements are measured by strain arm instrumentation, the pressuremeter curve presents the relationship between the applied pressure and the corresponding soil cavity strain. The cavity strain is the ratio of the observed membrane expansion to the initial probe radius. The applied pressure has to be corrected for the strength and stiffness of the membrane. Soil parameters and design information derived from the pressuremeter curve include in situ lateral stress in the ground, soil stiffness or modulus, and soil strength. The following sections discuss the procedures for obtaining these parameters.

2.3.1 In Situ Lateral Stresses

Knowledge of in situ lateral stress is useful for many geotechnical engineering problems, including the following (Baguelin and Jezequel, 1974; Mitchell et al., 1978):
1. Laboratory testing, where lateral stress is needed to recreate in situ consolidation conditions.

2. Settlement prediction, where soil deformation parameters depend upon the magnitude of in situ effective stress level.

3. Design of retaining structures that restrain natural soil masses where the lateral stress acts on the structure.

4. Finite element analyses, in which the initial lateral stress is an important parameter to define the initial stress state in the soil.

Introducing probes in the earth mass to measure in situ stress field has historically been more commonly conducted in rock than in soil engineering. In rock engineering, Obert and Duvall (1967) note that measuring the in situ stress field requires a partial or complete strain-relief. Testing methods include flat-jack and borehole deformation methods. In both cases, to arrive at the in situ stress requires knowledge of the stress-strain relationship of the material. In rock masses, it is common to assume that the stress field is non-isotropic even in a horizontal plane.

In soil engineering, there are three approaches to obtain in situ lateral stresses: empirical correlations with index properties, laboratory procedures, and in situ techniques. Empirical correlations provide relationships between $K_0$ values and parameters determined in basic soil testing. Table 2.1 gives examples of such correlations. Because these methods actually rely on in situ stress measurements from other test results, they do not provide an independent means of measuring the lateral in situ stress. Mayne and Kulhawy (1990) developed the first such correlation which relies exclusively on SBPM data. Using results of 56 tests in soft and stiff clays, the relationship between $K_0$ and the overconsolidation ratio, OCR, is:

$$K_0 = 0.52 \text{ OCR}^{0.51}$$
TABLE 2.1 Index correlation methods for estimating the coefficient of earth pressure at rest, \( K_o \) of clays

<table>
<thead>
<tr>
<th>Method</th>
<th>Formula</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>For normally consolidated (NC) clays:</strong></td>
<td>( K_o = 0.4 + 0.007(\text{PI}) ) (for ( 0 &lt; \text{PI} &lt; 40 ))</td>
<td>( \text{Brooker and Ireland (1965)} )</td>
</tr>
<tr>
<td></td>
<td>( K_o = 0.64 + 0.001(\text{PI}) ) (for ( 40 &lt; \text{PI} &lt; 80 ))</td>
<td></td>
</tr>
<tr>
<td><strong>For over-consolidated (OC) clays:</strong></td>
<td>( (K_o)<em>{\text{OC}} = (K_o)</em>{\text{NC}} \sqrt{\text{OCR}} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \text{where OCR = Overconsolidation ratio} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( (K_o)<em>{\text{OC}} = (K_o)</em>{\text{NC}} \cdot (\text{OCR})^m )</td>
<td>( \text{Schmidt (1966)} )</td>
</tr>
<tr>
<td></td>
<td>( \text{where (}K_o\text{)_{NC} and m values are functions of PI} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( (K_o)_{\text{Lowerbound}} = 1 - \sin \phi' )</td>
<td>( \text{Tavenas (1975)} )</td>
</tr>
<tr>
<td></td>
<td>( (K_o)_{\text{Upperbound}} = (1 - \sin \phi') \cdot (P_c'/P_o') )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( K_o = (1 - \sin \phi') \cdot (P_c'/P_o')^{0.42} )</td>
<td>( \text{Schmertmann (1975)} )</td>
</tr>
<tr>
<td></td>
<td>( \text{where } P_c' = \text{Pre-consolidation pressure, and} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( P_o' = \text{Effective insitu overburden pressure} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( K_o = (1 - \sin \phi') \cdot (P_c'/P_o')^{0.50} )</td>
<td>( \text{Meyerhoff (1975)} )</td>
</tr>
<tr>
<td><strong>For clays with sensitivity ranging from 1 to 10:</strong></td>
<td>( K_o = (1 - \sin \phi') \cdot \left( \frac{P_o}{\sigma_{w0}'} \right)^{\sin \phi'} \cdot 10^{(1.10 - 1.62 LI)} )</td>
<td>( \text{Kulhawy et al. (1989)} )</td>
</tr>
<tr>
<td></td>
<td>( \text{where } P_o = \text{Atmospheric pressure} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \sigma_{w0}' = \text{Effective overburden pressure} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( LI = \text{Liquidity index of the soil} )</td>
<td></td>
</tr>
</tbody>
</table>
However, the correlation assumes that the clay has been subjected to only one cycle of unloading. Unfortunately, this is typically not true in most stiff clays.

Laboratory procedures to obtain the in situ stress usually involve using special devices with remolded samples. This approach is useful in providing a general context for trends in $K_o$ values, and obtaining an estimate of $K_o$. However, Wroth (1975) concluded "... there is no satisfactory method of determining $K_o$ for a natural soil by laboratory testing."

Direct measurement of in situ stress is theoretically possible by using in situ techniques. From the early days of the development of the pressuremeter, this device was thought to be capable of measuring the in situ lateral stress. Menard proposed that the in situ lateral stress was the stress at the point where the MPM pressuremeter curve reverses from its initial curved portion to a linear phase. However, experience has shown that due to soil disturbance induced by preboring, the MPM approach cannot yield consistent values of the in situ lateral stress (Tavenas, 1975; Mair and Wood, 1987).

Workers using the SBPM have reported success in measuring lateral stress in soft to medium clays (Powell, 1990; Lacasse et al., 1983), although results in stiff clays have been mixed (Clough, et al., 1990).

Arnold (1983) attempted to explain the difficulty of measuring in situ stress in the pressuremeter test in terms of the large number of factors that influence the measurement:

1. Soil conditions
2. Hole preparation technique
3. Cutter geometry
4. Hydraulic fracture
5. Relaxation time before starting the test
6. Soil anisotropy
7. Effect of soil cementation, and
8. Compliance effect of the device.

In the best of circumstances, a carefully conducted SBPM test is required to obtain reliable in situ lateral stress measurements.

2.3.1.1 Methods of Interpretation

There are several methods to interpret in situ horizontal stresses from the result of an SBPM test. Lacasse and Lunne (1983) summarized some of these as shown in Figure 2.9. They suggested that the following approaches provided satisfactory estimates of in situ horizontal stresses in two Norwegian soft clay deposits: (1) the initial excess pressure method (Wroth and Hughes, 1973); (2) the modified lift-off pressure method (Lacasse et al., 1981); (3) the graphical iterative method (Marsland and Randolph, 1977); and (4) the hyperbolic approaches of Denby (1978) and Arnold (1981). Among the available methods, visual inspection of a pressuremeter curve, called "lift-off" method [see Figure 2.9 (a)] is the most straightforward method and gives reasonable results (Mair and Wood, 1987; Powell, 1990; Hawkins et al., 1990).

The "lift-off" pressure is the pressure at which the movement of the membrane initiates during an SBPM test. Measurement of the lift-off pressure can be enhanced by using a computer data acquisition process and intensifying the number of data reading during the initial stage of a test. In stiff soils, complications occur since the lateral pressures can be high enough to lead to compliance
FIGURE 2.9 Interpretation of insitu horizontal stress using self-boring pressuremeter test results (from Lacasse and Lunne, 1983)
problems in the pressuremeter system (e.g., membrane compression or reverse deflection of the strain arm). Further, the lift-off stress is sensitive to system behavior. So, any disturbance in the probe drilling process can make it difficult to decide the lift-off pressure.

To convert the lift-off pressure to the lateral stress in the soil, membrane inertia effect must be subtracted out. The membrane inertia effect can be defined by performing calibrations on the SBPM in the laboratory. Since membranes and strain arms typically do not have uniform properties, it is useful to have calibrations for each strain arm. A computer data acquisition system can store membrane inertia effects in the computer memory, and automatically make corrections to the field data.

2.3.1.2 Measurements of Anisotropic Lateral Stress Distribution

In soil engineering, in situ horizontal stresses are usually assumed to be equal in all directions if the ground surface is level. This assumption is likely true only in a practical sense, since geologic processes for deposition or overconsolidation of soil will not be uniform. Wroth (1984) states that different lateral stresses in all horizontal directions are not likely. However, in rock masses, anisotropic stress measurement is a common practice (Ribacchi, 1977).

The Camkometer has three or more strain arms oriented in different radial directions to measure membrane movements. Thus, for each test, different strain arms may yield different pressuremeter curves. This capability is useful in checking the strain arm operation. It also allows assessment of nonuniform membrane deformation, which may be caused by natural causes, such as presence of an
anisotropic lateral stress field in the soil. It is not uncommon to record and process
thousands of data points from each strain arm. This is only possible with the
application of computer-based data acquisition systems.

Table 2.2 presents cases where investigators have reported anisotropic
horizontal stress measurements in soil. In each of these cases, the investigators
provided reasons for the presence of the anisotropic stresses, and could confirm that
no instrument error influenced the data. Associated with the existence of an
anisotropic horizontal stress field, determination of the magnitudes and directions
of the principal horizontal stresses becomes a problem. Because the pressuremeter
strain arms are not necessarily aligned with the principal stresses. Dalton and
Hawkins (1982) proposed a method to interpret principal lateral stresses using a
Mohr’s circle stress representation (see Figure 2.10). Mayu (1987) proposed
another interpretation method based on a stress ellipse (see Figure 2.11). Both
methods will be further discussed in Chapter 3.

2.3.2 Determination of Undrained Shear Strength, $S_u$

During a pressuremeter test, soil around the pressuremeter is gradually
sheared as the membrane expands. The following sections discuss methods for
determining the undrained shear strength, $S_u$, from pressuremeter testing in clays.
All methods assume that the soil is saturated, and is sheared under undrained, no
volume change conditions.
<table>
<thead>
<tr>
<th>REFERENCE</th>
<th>TYPE OF CLAY; LOCATION</th>
<th>COMMENTS</th>
</tr>
</thead>
</table>
| Dalton and Hawkins (1982) | Guilt stiff clay in the Cambridgeshire countryside, England | (1) Different lift-off pressures are obtained from three individual strain arms of the Camkometer without involving any mechanical defects.  
(2) In situ horizontal stress measurements should rely on readings of independent strain arms instead of taking an average reading.  
(3) A method of Mohr's circle stress representation is applied to interpret the magnitudes and directions of principal horizontal stresses.  
(4) The ambiguity of the directions of the major horizontal stress in different boreholes may be due to the boreholes interfere with each other. |
| Ghionna et al. (1983)     | Taranto stiff clay, Italy                                   | (1) Anisotropic horizontal stress measurements are obtained from thee strain arms of the Camkometer. The maximum $K_s$ value based on the measured lift-off pressures ranges from 3 to 6, which is much higher than the $K_s = 2.4$ obtained from the Brooker and Ireland method.  
(2) The anisotropic horizontal stress measurements were found to be due to mechanical defects of the strain arms system and uneven soil disturbance around the probe. |
| Benoit and Clough (1986a) | Young Bay Mud (soft clay) at Hamilton Air Force Base         | (1) Anisotropic horizontal stress measurements are obtained from thee strain arms of the Camkometer without involving any mechanical defects.  
(2) Use Mohr's circle stress representation to determine the magnitudes and directions of principal horizontal stresses. The difference between the major and minor principal stresses is as high as 20%. |
| Mayu (1987)               | Miocene stiff clay, Richmond, VA.                           | (1) Anisotropic horizontal stress measurements are obtained from thee strain arms of the Camkometer without involving any mechanical defects.  
(2) The magnitudes and directions of principal horizontal stresses are interpreted by using a stress ellipse assumption.  
(3) The existence of anisotropic horizontal stresses may be due to local topographic effect. |
| Clarke and Allan (1990)   | Boom over-consolidated clay; 200 m below ground level at Mol, Belgium | (1) Different in situ horizontal stresses are obtained by using a high pressure self-boring pressuremeter, where an oversized cutter is applied.  
(2) Mohr’s circle representation is applied to interpret stresses in different directions. |
If \( a \) and \( b \) are the minor and major stresses respectively, and \( r_1, r_2 \) and \( r_3 \) are the three stress readings at 120°, then

\[
\begin{align*}
  r_1 &= \frac{1}{2} (b+a) + \frac{1}{2} (b-a) \cos 2\theta \quad (1) \\
  r_2 &= \frac{1}{2} (b+a) + \frac{1}{2} (b-a) \cos 2(\theta + \frac{2\pi}{3}) \quad (2) \\
  r_3 &= \frac{1}{2} (b+a) + \frac{1}{2} (b-a) \cos 2(\theta + \frac{4\pi}{3}) \quad (3)
\end{align*}
\]

rewrite in terms of \( \theta \)

\[
\begin{align*}
  r_1 &= \frac{1}{2} (b+a) + \frac{1}{2} (b-a) (\cos^2 \theta - \sin^2 \theta) \quad (4) \\
  r_2 &= \frac{1}{2} (b+a) + \frac{1}{2} (b-a) \left( \frac{1}{2} \sin^2 \theta - \frac{1}{2} \cos^2 \theta + \sqrt{3} \sin \theta \cos \theta \right) \quad (5) \\
  r_3 &= \frac{1}{2} (b+a) + \frac{1}{2} (b-a) \left( \frac{1}{2} \sin^2 \theta - \frac{1}{2} \cos^2 \theta - \sqrt{3} \sin \theta \cos \theta \right) \quad (6)
\end{align*}
\]

add equations (4), (5) and (6) gives

\[
\frac{1}{2} (b+a) = \frac{1}{3} (r_1 + r_2 + r_3) \quad (7)
\]

Subtracting equation (6) from equation (5) gives

\[
r_2 - r_3 = \frac{\sqrt{3}}{2} (b-a) \sin 2\theta \quad \text{Hence} \quad \sin 2\theta = \frac{2(r_2 - r_3)}{\sqrt{3}(b-a)} \quad (8)
\]

Rewrite equation (1) using (7)

\[
\cos 2\theta = \frac{2(2r_1 - r_2 - r_3)}{3(b-a)} \quad (9)
\]

Using the relationship \( \sin^2 \theta + \cos^2 \theta = 1 \) eliminate \( 2\theta \)

\[
\frac{1}{2} (b-a) = \frac{2}{3} \sqrt{r_1^2 + r_2^2 + r_3^2 - r_1r_2 - r_2r_3 - r_3r_1} \quad (10)
\]

adding equations (7) and (10) gives

\[
b = \frac{1}{3} (r_1 + r_2 + r_3) + \frac{2}{3} \sqrt{r_1^2 + r_2^2 + r_3^2 - r_1r_2 - r_2r_3 - r_3r_1}
\]

subtracting equation (10) from equation (7) gives

\[
a = \frac{1}{3} (r_1 + r_2 + r_3) - \frac{2}{3} \sqrt{r_1^2 + r_2^2 + r_3^2 - r_1r_2 - r_2r_3 - r_3r_1}
\]

\( \theta \) may be obtained from equation (8), but as the right hand side of equation (10) can be kept in the calculator store, it is perhaps better obtained from

\[
\sin 2\theta = \frac{1}{\sqrt{3}} \sqrt{\frac{2}{3} \sqrt{r_1^2 + r_2^2 + r_3^2 - r_1r_2 - r_2r_3 - r_3r_1}}
\]

FIGURE 2.10 Interpretation of principal lateral stresses proposed by Dalton and Hawkins (1982)
(x, y) coordinate system with x chosen arbitrarily parallel to Arm 1.
(X, Y) coordinate system with X and Y respectively parallel to \( \sigma_1 \) and \( \sigma_2 \), the major and the minor principal total lateral soil pressures in the horizontal plane.

Equation of the ellipse in the (X, Y) coordinate system

\[
\frac{X^2}{(\sigma_1)^2} + \frac{Y^2}{(\sigma_2)^2} = 1
\]

Equation of the ellipse in the (x, y) coordinate system

\[
x^2 \left[ \frac{\cos^2 \alpha}{(\sigma_1)^2} + \frac{\sin^2 \alpha}{(\sigma_2)^2} \right] + y^2 \left[ \frac{\sin^2 \alpha}{(\sigma_1)^2} + \frac{\cos^2 \alpha}{(\sigma_2)^2} \right] + 2xy(\cos \alpha)(\sin \alpha)\left[ \frac{1}{(\sigma_1)^2} - \frac{1}{(\sigma_2)^2} \right] = 1
\]

or

\[
Ax^2 + By^2 + Cxy = 1
\]

\[
\begin{bmatrix}
  x_1^2 & y_1^2 & x_1y_1 \\
  x_2^2 & y_2^2 & x_2y_2 \\
  x_3^2 & y_3^2 & x_3y_3
\end{bmatrix}
\begin{bmatrix}
  A \\
  B \\
  C
\end{bmatrix}
= 
\begin{bmatrix}
  1 \\
  1 \\
  1
\end{bmatrix}
\]

where \((x_1, y_1), (x_2, y_2)\) and \((x_3, y_3)\) are the coordinates of the total lateral pressures measured by Arm 1, Arm 2 and Arm 3 respectively. Solve for \(A\), \(B\) and \(C\). Then, solve for \(\alpha\), \(\sigma_1\), and \(\sigma_2\) with

\[
\tan 2\alpha = \frac{C}{A-B}
\]

\[
\frac{1}{(\sigma_1)^2} = A\cos^2 \alpha + C(\sin \alpha)(\cos \alpha) + B\sin^2 \alpha
\]

\[
\frac{1}{(\sigma_2)^2} = A\sin^2 \alpha - C(\sin \alpha)(\cos \alpha) + B\cos^2 \alpha
\]

FIGURE 2.11 Ellipse approach for determining total lateral pressures (from Mayu, 1987)
2.3.2.1 Menard Method

Menard (1957) proposed the first method for estimating $S_u$ from pressuremeter testing results as follows:

$$S_u = \frac{(P_L - P_o)}{N}$$

where $P_L$ and $P_o$ are the limit pressure and in situ lateral stress, respectively, and $N$ is called the "pressuremeter constant" (Marsland and Randolph, 1977). The $N$ value was originally determined empirically, and was thought to range from 5 to 7.5 (Baguelin et al., 1978). Using the theory of cavity expansion in an ideal elasto-plastic material, Marsland and Randolph (1977) showed the $N$ value as:

$$N = 1 + \ln\left(\frac{G}{S_u}\right)$$

where $G$ is the soil shear modulus. If the shear modulus is available, this approach can obtain the $S_u$ value by iteration. To obtain the shear modulus, Mair and Wood (1987) recommended performance of a small unload-reload cycle during the pressuremeter test. Discussion of the shear modulus determination will be presented in a later section.

2.3.2.2 The Gibson and Anderson (GA) Interpretation

Based on the analogy of an expanding cylinder, Gibson and Anderson (1961) provided the first rigorous method to obtain $S_u$ from the pressuremeter test. The assumptions include:
1. The soil behaves as an elastic-perfectly plastic material characterized by an undrained Young's modulus, E and an undrained shear strength, $S_u$.

2. The shearing occurs under undrained and radial plane strain conditions.

3. Cavity expansion theory applies; i.e., the pressuremeter probe behaves as an infinitely long circular cylinder expanded under uniform inner pressure in a semi-infinite medium.

Based on these assumptions, the GA interpretation leads to the following equation:

$$P = P_L + S_u \cdot 1n \left[ \frac{\delta V}{V} - 3 \left(1 - \frac{\delta V}{V}\right) \left(\frac{\sigma_H}{E}\right) \right]$$

where $P$ = applied pressure in the cavity  
$P_L$ = limit pressure; pressure corresponding to an infinite expansion  
$S_u$ = undrained shear strength of the soil  
$\sigma_H$ = total lateral pressure in the soil  
$E$ = Young's modulus of the soil  
$V$ = current volume of the pressuremeter probe  
$\delta V$ = difference between the initial probe volume and the current probe volume corresponding to the applied pressure $P$.

For pressuremeters equipped with strain arm instrumentation, the volumetric strain, $\delta V/V$ can be approximated as:
\[
\frac{\delta V}{V} = 1 - \frac{1}{(1 + \epsilon_c)^2}
\]

where \(\epsilon_c\) = cavity strain, the observed membrane expansion divided by initial probe radius.

Usually, the \(\sigma_H/E\) value is comparably smaller than \(\delta V/V\). So, if plotting the pressure, \(P\), versus \(\ln(\delta V/V)\), the undrained shear strength can be determined from the slope of the final straight line portion of the curve (see Figure 2.12). Wroth (1984) contended that the GA interpretation gave a conservative \(S_u\) value for design in geotechnical engineering.

### 2.3.2.3 The Baguelin, Ladanyi and Palmer (BLP) Interpretation

Interpretation methods proposed by Baguelin et al. (1972), Ladanyi (1972), and Palmer (1972) are essentially the same. The methods yield the undrained shear strength and the stress-strain curve. Unlike the GA interpretation, the BLP interpretation method does not require a pre-defined soil stress-strain response. The assumptions include:

1. Soil is saturated and deformed under undrained conditions.
2. A radial plane strain condition exists.
3. All points of the soil around the pressuremeter probe follow the same stress-strain relationship.
4. The vertical stress remains as the intermediate principal stress.
FIGURE 2.12 Determination of undrained shear strength using the Gibson-Anderson interpretation
Under small strain conditions, the final equation of the BLP interpretation is:

\[
\frac{(\sigma_r - \sigma_\theta)}{2} = \epsilon_r \cdot \Phi'(\epsilon_r)
\]

where \( \sigma_r = \) total radial stress  
\( \sigma_\theta = \) total tangential stress  
\( \epsilon_r = \) radial strain  
\( \Phi'(\epsilon_r) = \) slope of the pressuremeter curve

The equation shows that the undrained shear strength is obtained by multiplying the slope at any point on the pressuremeter curve to the corresponding radial strain. Amar et al. (1975) proposed a graphical procedure to derive a complete stress-strain curve (see Figure 2.13). For strain hardening materials, the GA and the BLP interpretations give similar results, because the soil behavior is close to that of an elastic-perfectly plastic material. However, for brittle materials with strain softening behavior, the BLP interpretation can identify the peak strength, whereas this is not possible with the GA method. The disadvantage of the BLP interpretation is that the \( S_u \) value is sensitive to the selection of a strain datum. The strain datum selection may become difficult when there exists soil disturbance caused by pressuremeter probe insertion (Wroth, 1984). The BLP method is known to lead to over-estimation of the \( S_u \) value in the presence soil disturbance (Baguelin, et al., 1978; Denby and Clough, 1980).

2.3.2.4 Other Methods

Besides the Menard, GA and BLP methods, there are some other methods to interpret the \( S_u \) value (Prevost and Hoeg, 1975; Denby and Clough, 1980).
FIGURE 2.13  BLP method for constructing stress-strain curve (from Baguelin et al., 1978)
However, there has been a general tendency toward using a simpler procedure (e.g., the Menard or GA method) because: (1) Complex methods do not automatically lead to better $S_u$ values; (2) Soil $S_u$ values are often scattered in nature, which rules out the need for what are reported as more accurate procedures (Mair and Wood, 1987).

2.3.2.5 Experience of Interpreting $S_u$ Values From Pressuremeter Tests

Commonly, $S_u$ values interpreted from SBPM tests are higher than those from conventional testing methods (Mair and Wood, 1987; Clough, et al., 1990). Explanations of this have focused on effects of soil disturbance around the probe, influence of finite length of the probe, and drainage of excess pore pressures during the shearing process. In summary, factors said to lead to higher $S_u$ values in SBPM testing versus that from conventional testing are:

1. Presence of a remolded soil annulus around the pressuremeter (Baguelin et al., 1978; Denby and Clough, 1980).
2. Use of an incorrect in situ horizontal stress in interpreting the undrained shear strength (Baguelin et al., 1978).
3. Effects of a higher strain rate applied in pressuremeter testing than that used in laboratory testing (Anderson et al., 1987).
4. Effects of the stress path applied to the soil in the pressuremeter test as opposed to that used in laboratory testing (Anderson et al., 1987).
5. The finite length to diameter ratio of the probe (Yeung and Carter, 1990).
6. Partial drainage effect during the expansion (Fukagawa et al., 1990).
7. Experimental errors (Lacasse et al., 1990).
When considering these arguments, note that $S_u$ values obtained from laboratory tests using conventional standards often under-represent the actual strength of a soil mass. This is particularly true for stiff fissured clays (Baguelin, et al., 1978). Thus, it should not inherently be assumed that the SBPM gives an incorrect value of $S_u$ when it is higher than the laboratory $S_u$.

There is no certain answer about how to view the results of $S_u$ interpreted from the pressuremeter test. In stiff fissured clays, the undrained shear strengths obtained from most conventional testing methods are unreliable. From experience in London Clay, Wroth (1984) concluded that the $S_u$ value derived from the GA interpretation of SBPM testing data was consistent with observed behavior. A problem remaining in most stiff clays is the uncertainty introduced by the effects of fissures and slickensides. This study has the advantage of dealing with a stiff clay that does not contain fissures or slickensides to any degree.

### 2.3.3 Determination of Modulus Value

The soil modulus, which represents the soil stiffness, is useful for prediction of soil movements in geotechnical problems. When plotting the pressuremeter curve in terms of pressure versus radial strain, the slope of the curve is twice the value of the shear modulus. Note that defining soil shear modulus from the SBPM testing results is based on the principal stress difference, and is independent of the drainage conditions (Windle and Wroth, 1977). Thus, problems concerning drainage and volume change that arise with definition of shear strength do not arise in defining shear modulus.

Referring to Figure 2.14, different definitions of the shear modulus include initial shear modulus, $G_i$, and unload-reload shear modulus, $G_{ur}$. The $G_i$ value is
(a) PRESSUREMETER CURVE WITH WELL DEFINED LIFT-OFF PRESSURE

(b) PRESSUREMETER CURVE WITH UNDEFINABLE LIFT-OFF PRESSURE

FIGURE 2.14 Determination of shear moduli from pressuremeter testing
derived from the initial linear portion of a pressuremeter curve. The $G_{ur}$ value is available from the unload-reload loop of a pressuremeter curve. A secant modulus also can be measured at a given strain level. However, it is not commonly used and will not be discussed further.

The initial shear modulus has the disadvantage of being sensitive to disturbance introduced during pressuremeter installation. However, the unload-reload shear modulus is relatively insensitive to these effects (Robertson and Hughes, 1986; Bellotti, et al., 1986; Withers et al., 1986; and Pappas, 1993). It has also been observed that the unload-reload shear modulus is independent of the strain rate applied in the unload-reload cycle (Windle and Wroth, 1977), and is insensitive to the pressure where the unload-reload cycle starts (Hughes, 1983). These findings show that the modulus derived from the unload-reload process is much less subject to test effects than others. One factor that can significantly influence the soil unload-reload modulus is the magnitude of unloading. Wroth (1982) was the first one to note that the unloading should not exceed the elastic limit of the clay, and recommended that the unloading amplitude be kept to less than twice the magnitude of the undrained shear strength.

Table 2.3 shows comparisons of modulus values derived from pressuremeter tests in stiff clays to those obtained from other methods. The unload-reload shear modulus value from the pressuremeter is typically much higher than the results of laboratory triaxial compression tests. Experience suggests that movement predictions based on the $G_{ur}$ modulus from SBPM testing is consistent with field behavior (Grant, 1985).

A recent study by Clarke and Allan, 1990 showed the $G_{ur}$ values from SBPM testing tend to decrease as cavity strain amplitude increases (see Figure 2.15). Powell (1990) suggests that the $G_{ur}$ value should be presented with the
TABLE 2.3 Review of pressuremeter testing in stiff soils

<table>
<thead>
<tr>
<th>REFERENCE</th>
<th>TYPE OF CLAY; LOCATION</th>
<th>SOME PROPERTIES OF THE CLAY</th>
<th>PROBE USED; No. OF TESTS</th>
<th>REMARKS</th>
<th>AT REST PRESSURE COEFFICIENT, $K_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windle, D. and Wroth, C.P. (1977)</td>
<td>Heavily overconsolidated fissured clay</td>
<td>Gault clay</td>
<td>Camkometer (English SBPM)</td>
<td>Gault clay marked effects of loading rate on peak shear strength interpreted from SBPM tests</td>
<td>Gault clay $E_u = 40 - 100$ MPa, $K_o = 3 - 4$</td>
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<tr>
<td></td>
<td>Two sites:</td>
<td>Gault clay, London clay</td>
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<td></td>
<td>Gault clay, Cambridge</td>
<td>$w_i = 25 - 30%$</td>
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<td></td>
<td>London clay</td>
<td>$w_i = 26 - 30%$</td>
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<td></td>
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<td>$L_L = 70 - 75$</td>
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<td>$P_I = 40 - 53$</td>
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<td></td>
<td></td>
<td>$S_u = 70 - 160$ kPa</td>
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<td>$w_i = 60 - 80$</td>
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<td>$P_I = 34 - 50$</td>
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<td>$S_u = 200 - 350$ kPa</td>
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<td>$S_u = 220 - 350$ kPa</td>
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<td></td>
<td></td>
<td>$S_u = 3.4$</td>
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<tr>
<td>Janiokowski, M. and Lancellotta, R. (1977)</td>
<td>Very stiff, slightly fissured clay</td>
<td>Montalto di Castro (Italy)</td>
<td>PASFOR (French SBPM) 12 tests</td>
<td>Values of strength interpreted from SBPM tests are 2.0 - 2.5 times larger than values from UU triaxial compression tests</td>
<td>London clay $E_u = 40 - 100$ MPa, $K_o = 1.6 - 3.6$</td>
</tr>
<tr>
<td>Law, K. T. and Eden, W. J. (1980)</td>
<td>Stiff sensitive clay</td>
<td>Leda clay, NRCC, Ottawa</td>
<td>Camkometer (English SBPM)</td>
<td>Oversized cutting shoe leads to overestimate shear strength by 80%</td>
<td>Oversized cutting shoe leads to overestimate modulus by 30%</td>
</tr>
</tbody>
</table>

* from laboratory tests

(Note): This table is an update from the literature review of Mayu (1987)
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<th>MODULUS</th>
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<th>AT REST PRESSURE COEFFICIENT, K₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>stiff sensitive clay, NRCC, Ottawa</td>
<td>Wₐ = 80%</td>
<td>Camflexometer (English SBPM)</td>
<td>80%</td>
<td>shear strength interpreted from SBPM tests at 20% - 40% strain</td>
<td>Good agreement between K₀ values from SBPM test and laboratory ( K₀ = 0.8 - 1.3 ) unit F ( K₀ = 1.6 - 2.2 )</td>
</tr>
<tr>
<td>stiff to hard overconsolidated clay, with fissures, sickle-shaped shear zone</td>
<td>wₐ = 42%</td>
<td>Camflexometer (English SBPM)</td>
<td>24%</td>
<td>shear strength interpreted from SBPM test 1 in the upper range of the OCR strength data</td>
<td>( S₀ = 100 - 200 ) kPa O.C. ( = 4.6 ) unit F ( K₀ = 1.6 - 2.2 )</td>
</tr>
</tbody>
</table>
### TABLE 2.3 (Cont'd)  Review of pressuremeter testing in stiff soils

<table>
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<tr>
<th>REFERENCE</th>
<th>TYPE OF CLAY; LOCATION</th>
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<th>PROBE USED; No. OF TESTS</th>
<th>REMARKS</th>
<th>AT REST PRESSURE COEFFICIENT, $K_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Johnson, G.W. (1982)</td>
<td>Stiff clay, 2 sites in the vicinity of University of Texas at Austin</td>
<td>$W_0 = 16 - 25%$ other data not available Texas self-boring pressuremeter 4 tests</td>
<td></td>
<td></td>
<td>Good correlations between initial shear modulus interpreted from SBPM tests and Modulus values determined from seismic cross-hole tests</td>
</tr>
</tbody>
</table>

* from laboratory tests

**NOTE:** This table is an update from the literature review of Mayu (1987)
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</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beaumont formation closely spaced fissures and slickensides depth = 0 - 26 ft</td>
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<tr>
<td></td>
<td>Montgomery formation fissures, slickensides and sand seams depth &gt; 26 ft</td>
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<td></td>
<td>Huston, Texas</td>
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<tr>
<td></td>
<td>Beaumont formation</td>
<td>W_n = 20 - 30%</td>
<td>Texas self boring</td>
<td>Beaumont form, value of shear strength from SBPM are same order as UU</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LL = 40 - 80</td>
<td>pressuremeter</td>
<td>triaxial but 1.5 times larger than values from NSP (1) tests and</td>
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<tr>
<td></td>
<td></td>
<td>P_i = 30 - 60</td>
<td>8 tests</td>
<td>1.1 to 2.0 times smaller than values from CPT (2) tests</td>
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<td></td>
<td></td>
<td>S_i' = 30-150 kPa</td>
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<td></td>
<td></td>
<td>sensitivity = 0.5 - 1.5</td>
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<td>For both formations, values of E_u from UU tests random and scattered.</td>
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<tr>
<td></td>
<td></td>
<td>OCR &gt; 6</td>
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<td>values from cross-hole technique are 5.0 - 10.0 times larger than values</td>
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<tr>
<td></td>
<td>Montgomery formation</td>
<td>W_n = 15 - 25%</td>
<td></td>
<td>from SBPM. Values from NSP and CIU (3) tests are 2 - 3 times smaller</td>
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<tr>
<td></td>
<td></td>
<td>LL = 25 - 30</td>
<td></td>
<td>than values from SBPM</td>
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<tr>
<td></td>
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<td>P_i = 5 - 15</td>
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<td>S_i' = 50 - 220 kPa</td>
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<td></td>
<td></td>
<td>sensitivity = 2.0</td>
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<td></td>
<td>OCR = 4.0 - 6.0</td>
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<td>total unit weight =</td>
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<td>126pcf</td>
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* from laboratory tests

[NOTE]: This table is an update from the literature review of Mayu (1987)
TABLE 2.3 (Cont’d)  Review of pressuremeter testing in stiff soils

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<tr>
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<th>SHEAR STRENGTH</th>
<th>MODULUS</th>
<th>REMARKS</th>
<th>AT REST PRESSURE COEFFICIENT, $K_o$</th>
</tr>
</thead>
</table>
| Clarke and Allan (1990) | Boom clay; overconsolidated clay (can be treated as weak rock) | $W_p = 19.5\%$  
$L_I = 55$  
$P_L = 28$ | High pressure self-boring pressuremeter;  
73.88 mm in diameter and 1.9 m long;  
capacity up to 20 MPa | | | Shear modulus decreases as cavity strain amplitude increases | Inward movement of the soil due to the compression of the membrane may reduce the in situ total stress |
| Lacasse, S. et al. (1990) | Medium-stiff Haga clay | OCR = 2 - 30 | Self-boring pressuremeter  
(Cambridge In Situ Mark IV device);  
PIP (stress-probe) | | | Self-boring pressuremeter tests obtain much higher undrained shear strength than those from triaxial compression and field vane tests. | $G_{ur} = 10$ MPa from SBPM test. Consider unload-reload shear modulus as the most reliable parameter. However, its application in design remains complex. |

* from laboratory tests

[NOTE]: This table is an update from the literature review of Mayu (1987)
### TABLE 2.3 (Cont’d)  Review of pressuremeter testing in stiff soils

<table>
<thead>
<tr>
<th>Reference</th>
<th>Type of Clay; Location</th>
<th>Some Properties of the Clay</th>
<th>Probe Used; No. of Tests</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| Powell, J. J. M. (1990) | Stiff Gault clay (heavily overconsolidated clay) in Madingley test site; | $W_o = 28 - 32\%$  
$LL = 26 - 32$  
$PL = 72 - 80$  
$PI = 40 - 50$ | Self-boring pressuremeter (Camkometer); MPM; PIP; full displacement cone pressuremeter | The best available interpretation methods depend on the type of pressuremeter testing. All types of pressuremeter test obtain similar profile of limit pressure, which agree with the undrained shear strength obtained by triaxial and dilatometer test, but exist small deviation due to rate effect. | Unload-reload shear modulus is a function of the experienced cavity strain amplitude. Rate of testing may affect the modulus value.  
Using self-boring pressuremeter testing and interpreting by using lift-off and Marsland Randolph methods give reasonable results than the other testing methods and interpretation methods do. |
| Grant, W. P. (1985) | Seattle stiff fissured clays | $W_e = 20 - 40\%$  
$LL = 55 - 75$  
$PL = 23 - 30$  
SPT-N = 40 - 70 | SBPM | CU triaxial tests have  
$c' = 57.5$ kPa,  
$\phi' = 31^\circ$ (peak)  
$c' = 47.9$ kPa,  
$\phi' = 13^\circ$ (residual) | $E_0 = 145$ MPa from SBPM test, which is used to predict ground movements and matches the observed movements. |

* from laboratory tests

[NOTE]: This table is an update from the literature review of Mayu (1987)
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<th>AT REST PRESSURE COEFFICIENT, $K_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bergado and Khalil (1986)</td>
<td>stiff sandy clay, Bangkok marine clay (9m to 15m depth)</td>
<td>$W_p = 20 - 30%$</td>
<td>OYO LLT pressuremeter</td>
<td>$S_u = 0.8 - 2.2$ kgf/cm² from GA interpretation.</td>
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<tr>
<td></td>
<td></td>
<td>$LL = 50 - 70$</td>
<td></td>
<td>$E_u = 60 - 240$ kgf/cm² from linear part of the pressuremeter curve.</td>
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<td>$PI = 30 - 50$</td>
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<td>correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$</td>
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</tbody>
</table>

| Ohye et al. (1983)          | Highly overconsolidated alluvial clay              | $W_p = 32\%$               | OYO LLT pressuremeter    | $S_u = 0.8 - 2.2$ kgf/cm² from GA interpretation. |
|                            |                                                   | $PI = 30 - 40$             |                          | $E_u = 60 - 240$ kgf/cm² from linear part of the pressuremeter curve. |
|                            |                                                   | $OCR = 5 - 20$             |                          | $E_u = 60 - 240$ kgf/cm² from linear part of the pressuremeter curve. |
|                            |                                                   | $S_u \ast = 30 - 40$ kgf/cm² |                          | $E_u = 60 - 240$ kgf/cm² from linear part of the pressuremeter curve. |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
|                            |                                                   |                             |                          | correlation with CPT: $S_u = q_c/19.4$ $f_i/0.53$ |
| Clarke, B.G. (1979)         | Heavily overconsolidated clay. Gault clay at Madingley, Cambridge | $S_u = 70 - 300$ kPa       | SBPM (Cascometer); i4 tests | $S_u = 70 - 300$ kPa |
|                            |                                                   |                             |                          | Shear modulus is 8 - 60 MPa (Avg. = 30 MPa) |
|                            |                                                   |                             |                          | $K_o = 2 - 3$ |

* from laboratory tests

[NOTE]: This table is an update from the literature review of Mayu (1987)
TABLE 2.3 (Cont'd) Review of pressuremeter testing in stiff soils

<table>
<thead>
<tr>
<th>REFERENCE</th>
<th>TYPE OF CLAY: LOCATION</th>
<th>SOME PROPERTIES OF THE CLAY</th>
<th>PROBE USED; No. OF TESTS</th>
<th>SHEAR STRENGTH</th>
<th>MODULUS</th>
<th>AT REST PRESSURE COEFFICIENT, $K_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Houlsby, G.T. and Withers, N.J. (1988)</td>
<td>Heavily overconsolidated clay. Gault clay at Madingley, Cambridge</td>
<td>Fugro pressuremeter cone (FPC); full-displacement pressuremeter. Length to diameter ratio = 10</td>
<td>7 tests</td>
<td>$S_u = 80 - 192$ kPa, in good agreement with the SBPM results from Clarke (1979), and the results from triaxial tests using undisturbed samples.</td>
<td>Shear modulus is 12 - 69 MPa, in good agreement with the SBPM results from Clarke (1979)</td>
<td>Full-displacement pressuremeter does not give reliable result due to soil disturbance effect.</td>
</tr>
</tbody>
</table>

* from laboratory tests

[NOTE]: This table is an update from the literature review of Meyu (1987)
FIGURE 2.15  Variation of shear moduli versus different cavity strain ranges (after Clarke and Allan, 1990)
corresponding strain or stress amplitude experienced during the unload-reload cycle. This issue will be further investigated in this study.

2.3.4 Shape of Pressuremeter Membrane During Expansion

The shape of the pressuremeter membrane during expansion is an issue from both theoretical and practical perspectives. The theoretical aspect relates to whether the membrane follows the assumptions of a right, circular cylinder expansion. From a practical aspect, especially in stiff soils, one question is whether local stiff pockets can lead to deviations from the ideal expanding shape.

When applying cavity expansion theory to derive soil parameters from a pressuremeter test, it is assumed that the pressuremeter is infinitely long and expands cylindrically, and that the soil around the probe deforms under a radial plane strain condition. Schmertmann (1975) undertook an analytical study of the pressuremeter expanding in an elastic medium, and found that if the pressuremeter had a length to diameter ratio of about 6, it would come close to assuring a radial strain condition in the mid-plane of the probe.

Using a standard Menard tricell pressuremeter and a monocell pressuremeter, Ohya et al. (1983) carried out radiography studies to visualize the shape of pressuremeter membranes during testing. Figure 2.16 shows the vertical profile of the expanded probes. The results showed a largely cylindrical expansion at the yield pressure level except at the ends of the probes. When reaching the limit pressure, diameters of the probes tended to increase toward the bottom. Ohya et al. (1983) explained the bulging was due to effects of the water pressure difference between the water in the probe and the water in the borehole.
FIGURE 2.16  Vertical profiles of an expanded OYO probe (from Ohya et al., 1983)
Yeung and Carter (1990) used the finite element method to study cases of pressuremeter testing performed at shallow depths in soils with a non-homogeneous strength profile. They concluded that an irregular deformation pattern could result in some cases due to differences in stress confinement and strength profile (Figure 2.17). However, their problems are not relevant to conventional geotechnical cases.

The discussion to this point focused on the shape of the pressuremeter membrane in vertical profile. There is less knowledge about the shape of the membrane in the horizontal plane, but some data are available from field tests where individual strain arm measures are recorded. Using pressuremeters with three strain arms, Dalton and Hawkins (1982) and Benoit and Clough (1986) observed nonuniform horizontal membrane movements. Mayu (1987) obtained similar results when testing in the Richmond Miocene (Figure 2.18). He argued that the nonuniform membrane expansion was due to anisotropic stresses in the horizontal plane.

2.4 Miocene Clay: Previous Testing Results

The stiff soil of interest to this study is a clay located in Richmond, Virginia known locally as the Miocene clay. The Miocene deposits consist of dark gray marine clays and silts that are estimated to be 11,000,000 to 25,000,000 years old. They serve as the support layer for deep foundations of major structures in the downtown Richmond area. Casagrande (1966) conducted the first study in Miocene clays from a geotechnical perspective. Investigations by using PBPM and SBPM tests were reported by Martin and De Stephen (1983), Martin and Drahos (1986), and Mayu (1987). Mayu (1987) performed six SBPM tests in the Miocene clays,
FIGURE 2.17 Pressuremeter membrane deflection shapes of testing at a shallow depth (after Yeung and Carter, 1990)
MEASUREMENT OF ARM 1 OF TEST R1

NOTE: The deflection shown above is not to scale

FIGURE 2.18  Horizontal membrane profiles based on Mayu (1987) SBPM testing data in Miocene clays
and his work was followed by Mullen (1991) who conducted CPT, SPT, and DMT as well as MPM in holes formed by driving a split-spoon sampler.

The Miocene clays are unique in two respects: (1) Few, if any, fissures are present; and (2) the soil is sensitive, and subject to a loss of strength if disturbed. It was the latter characteristic that first brought the Miocene clay to the attention of Casagrande. At the time of Casagrande's work, design parameters for the Miocene clay were often based on blow counts obtained by standard penetration testing. Because of the sensitivity of the Miocene clay, blow counts were abnormally low. Accordingly, the design strengths and modulus values were excessively conservative. With Casagrande's study, this effect was recognized and he adopted new approaches to take the sensitivity into account.

The Miocene clay posed an ideal choice for this investigation. It is stiff and largely free from fissures, which allow for better comparison between results of different types of tests. The clay is also interesting in its sensitivity and possible presence of high in situ lateral stresses.

2.4.1 Laboratory Test Results

The Richmond Miocene deposit is a medium gray, stiff and generally non-fissured clay or silt of marine origin. Mayu (1987) obtained samples by both conventional procedures and undisturbed block sampling so that the effect of disturbance could be assessed in comparing different investigations.

Total unit weight of the Miocene clay varies from 17.2 kPa to 18.2 kPa. Typically the Miocene soil is more of a silt than a clay, but is commonly called "the Miocene clay." According to the Unified Soil Classification System, the Miocene
clay has the Atterberg limits placed on the border of MH-OH and CH soils (see Figure 2.19). Based on different Atterberg Limits, Casagrande (1966) characterized the Miocene soils as upper Miocene and lower Miocene. Plasticity indices for the upper Miocene are higher than those of the lower Miocene. At the site tested by Mayu (1987) (hereafter called the Schnabel site), the depth of separation between the upper Miocene and the lower Miocene occurs at a depth about 7 meters (Mullen, 1991).

Results of oedometer tests show that the over-consolidation ratio, OCR for the Miocene clay varies between 4 and 5. Both Mayu (1987) and Casagrande (1966) have shown that soil sample disturbance can lead to incorrect assessment of the OCR because of soil sensitivity.

Table 2.4 presents a summary of the $S_u$ values from undrained-unconsolidated laboratory triaxial testing. Mayu (1987) was the only investigator to test undisturbed block samples of the Miocene clays. The $S_u$ values of the block samples are higher than those of the Shelby tube samples. The block samples yielded an average $S_u$ values of 500 kPa, and typically failed at strains about half those observed when testing the Shelby tube samples. It was believed that the Shelby tube sample behaviors reflected the soil disturbance effects. The disturbance might be induced by pushing in the Shelby tube, and/or extracting the sample from the tube. Testing on the completely remolded samples showed that the soil sensitivities ranged from five (5) to eight (8).

2.4.2 Self-Boring Pressuremeter Test Results

Using a three strain arm Camkometer, Mayu (1987) performed six SBPM tests in the Miocene clay deposit in downtown Richmond, Virginia. The SBPM
FIGURE 2.19 Plasticity index of the Miocene clay (from Mayu, 1987)
TABLE 2.4  \( S_u \) values from UU triaxial tests and SBPM tests in the Miocenes

<table>
<thead>
<tr>
<th>Investigators</th>
<th>Sample Type</th>
<th>Undrained Shear Strength (KN/M²), ( S_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>\begin{tabular}{l} \text{UU Triaxial} \ \text{SBPM GA Method} \ \text{SBPM BLP Method} \end{tabular}</td>
</tr>
<tr>
<td>Casagrande (1965)</td>
<td>Shelby Tube</td>
<td>\begin{tabular}{l} \text{Upper Miocene} \ \text{Lower Miocene} \ \text{Remolded} \ \text{Upper Miocene} \ \text{Lower Miocene} \end{tabular}</td>
</tr>
<tr>
<td></td>
<td></td>
<td>\begin{tabular}{l} \text{220-245} \ \text{319-441} \ \text{40} \ \text{10-30} \end{tabular}</td>
</tr>
<tr>
<td>Martin and De Stephen (1983)</td>
<td>Shelby Tube</td>
<td>\begin{tabular}{l} \text{Lower Miocene} \end{tabular}</td>
</tr>
<tr>
<td></td>
<td></td>
<td>\begin{tabular}{l} \text{240-442} \end{tabular}</td>
</tr>
<tr>
<td>Mayu and Clough (1990)</td>
<td>Remolded</td>
<td>\begin{tabular}{l} \text{Lower Miocene} \end{tabular}</td>
</tr>
<tr>
<td></td>
<td>Shelby Tube</td>
<td>\begin{tabular}{l} \text{Lower Miocene} \end{tabular}</td>
</tr>
<tr>
<td></td>
<td>Block</td>
<td>\begin{tabular}{l} \text{Upper Miocene} \ \text{Upper Miocene} \ \text{Lower Miocene} \end{tabular}</td>
</tr>
<tr>
<td></td>
<td></td>
<td>\begin{tabular}{l} \text{274-500} \ \text{265} \ \text{215-320} \end{tabular}</td>
</tr>
<tr>
<td></td>
<td></td>
<td>\begin{tabular}{l} \text{265-340} \ \text{340-470} \end{tabular}</td>
</tr>
</tbody>
</table>
tests included five tests at the Schnabel Center site (tests R1 to R5), and one in a parking lot of the Richmond Coliseum site (test R6). Each test included a primary loading stage, unload-reload cycle, and a final reloading. Figure 2.20 presents a typical test result, which illustrates well-defined lift-off pressures.

2.4.2.1 In Situ Lateral Stress Distribution

Table 2.5 shows the in situ horizontal stresses and corresponding $K_o$ values interpreted by Mayu from his SBPM tests. Since he defined the lateral stresses by the lift-off pressure measurements of the three strain arms pressuremeter, two trends stood out in his results: (1) The lateral stresses are always larger than the corresponding vertical stresses at all testing depths; and (2) The three strain arm measurements result in different lateral stresses at a given depth. The $K_o$ values estimated by Mayu (1987) ranged from 3 to 19, with the lower values occurring at greater depths. The Mayu $K_o$ values are much larger than those normally associated with soils. Using the correlation proposed by Kulhawy and Mayne (1990), with the OCR values ranging from 4 to 5, it would be argued that the $K_o$ value of the Miocene clay should be about only 1.05 to 1.20.

The reason for the high $K_o$ values in the Miocene clays is not obvious. Of course, it is possible that there was some type of experimental error. However, all Mayu's self-boring pressuremeter testing results were consistent, and he checked all calibrations before and after testing. One reasonable explanation for the high $K_o$ values is the existence of a natural drained cohesion in the Miocene clay. Note that if no drained cohesion exists, the highest possible $K_o$ value is about the soil passive pressure coefficient, or about 3 for most clays and silts. However, a small amount of drained cohesion can account for much higher $K_o$ values.
FIGURE 2.20 Typical result of Mayu (1987) self-boring pressuremeter testing in Miocene clays
TABLE 2.5 Lateral principal stress in the Miocene clays interpreted from the SBPM results (from Mayu, 1987)

<table>
<thead>
<tr>
<th>Test</th>
<th>$\sigma_h'$ (KN/M²)</th>
<th>$\sigma_v'$ (KN/M²)</th>
<th>$K_o = \frac{\sigma_h'}{\sigma_v'}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Arm1</td>
<td>Arm2</td>
<td>Arm3</td>
</tr>
<tr>
<td>R1</td>
<td>353 (51.2)</td>
<td>891 (129.1)</td>
<td>484 (70.1)</td>
</tr>
<tr>
<td>R2</td>
<td>413 (59.9)</td>
<td>1034 (149.9)</td>
<td>565 (81.9)</td>
</tr>
<tr>
<td>R3</td>
<td>390 (56.5)</td>
<td>997 (144.5)</td>
<td>542 (78.6)</td>
</tr>
<tr>
<td>R4</td>
<td>472 (68.4)</td>
<td>1176 (170.4)</td>
<td>624 (90.4)</td>
</tr>
<tr>
<td>R5</td>
<td>498 (72.2)</td>
<td>1443 (209.1)</td>
<td>705 (102.2)</td>
</tr>
<tr>
<td>R6</td>
<td>915 (132.6)</td>
<td>1184 (171.6)</td>
<td>1467 (212.6)</td>
</tr>
</tbody>
</table>
Besides the SBPM testing results, there is peripheral evidence supporting the existence of the significant high lateral stresses in the Miocene. This includes: (1) A borehole in the stiff Miocene clay begins to close quickly after opening; (2) When extruding the clay from a sample tube, the clay often splits along horizontal planes into small discs. This phenomenon, called "discing" is common when removing rock core from highly stressed rock deposits; and (3) The Miocene clay rarely contains fissures or slickensides. The last evidence is especially unusual when compared to most stiff clays. Existence of slickensides is an indication of a failure because the lateral stresses are high enough to induce shear stresses exceeding the strength of the soil. The lack of fissures or slickensides in the Miocene clay suggests the presence of enough drained strength to prevent the failure from occurring. If this is the case, then the lateral stresses remain "locked" in the soil.

As noted previously, Mayu (1987) reported different lift-off pressures measured from the three strain arms (see Table 2.5). He attributed this behavior to an anisotropy lateral stress field, and explained it as regional topographic effects that existed in the test areas.

2.4.2.2 Undrained Shear Strengths

Mayu (1987) applied the GA and the BLP interpretations to obtain $S_u$ values from his SBPM testing results in the Miocene clay. Table 2.4 summarizes the average undrained shear strength values of his work, including the results from laboratory tests on block samples and those from the SBPM tests. The table also includes results from other investigators. Conclusions are:

1. The strengths from the GA interpretation are about 20% to 35% lower than those from the BLP interpretation.
2. The strengths from the BLP interpretation are in good agreement with those from laboratory testing on the undisturbed block samples.

2.4.2.3 Modulus Values

As discussed earlier, the soil shear modulus can be derived directly from the SBPM testing results. Most other testing methods lead to the Young's modulus, not the shear modulus. Based on theory of elasticity, the Young's modulus and the shear modulus can be related if known the Poisson's ratio value for the soil. For testing in saturated clays under undrained conditions, it is reasonable to use a Poisson's ratio of 0.5. Using this assumption, Mayu (1987) compared the Young's modulus values of the Miocene clay obtained by the SBPM and MPM tests. The results are shown in Figure 2.21. Note that the MPM tests in comparison were not conducted at the Schnabel site, but were performed for consulting projects in the same general vicinity. The comparison shows that the MPM tests yielded the Young's modulus values about half those obtained from the SBPM tests. It was concluded that the lower values from the MPM tests might be due to disturbance when opening the test hole in the presence of the sensitive Miocene clay.

Mullen (1991) was the first to conduct pre-bored pressuremeter tests to obtain unload-reload shear modulus values of the Miocene clay at the Schnabel site. He opened a testing hole by driving a 3-inch diameter sampler that was subsequently removed. He then inserted a 2.75-inch diameter, nine strain arms pressuremeter into the hole for testing. Note that the pressuremeter used was the same as the one used in this investigation. Mullen's objective was to find if reasonable unload-reload modulus values could be obtained although the test hole was opened crudely. If satisfactory results could be obtained, then it would be possible to improve the efficiency of the pressuremeter test. The $G_{ur}$ values

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FIGURE 2.21  Elastic modulus values versus depth from SBPM and Menard pressuremeter tests in the Miocene clay (from Mayu, 1987)
obtained by Mullen were about 18 MN/M² at 3 meters depth, and ranged from 36 MN/M² to 42 MN/M² at depths from 5 to 6 meters. These $G_u$ values are about one-half of those obtained from the SBPM tests of Mayu (1987).

Since unload-reload process can be repeated during an SBPM test, Figures 2.22 and 2.23 show the effects of cyclic loading on the shear moduli as reported by Mayu (1987). It shows that the modulus value obtained from the second unload-reload cycle was about 20% lower than that from the first unload-reload cycle.

2.5 Summary

The existence of fissures and slickensides in stiff clays makes it difficult to obtain representative parameters from laboratory testing. In situ techniques can test stiff clays in the confined state where the influence of fissures and slickensides is minimized. Among the in situ techniques, the pressuremeter test appears to be promising for determining soil parameters for design. Types of pressuremeter device range from simple Menard pressuremeter to sophisticated self-boring pressuremeter. This study focuses on the self-boring pressuremeter (SBPM) technique, an approach that the probe drills itself into the ground. Under ideal circumstances, this procedure introduces minimal disturbance and leads to determination of reliable soil parameters.

Results of pressuremeter testing can be used to obtain shear strength, shear modulus, and initial lateral stress. There are a number of methods to obtain each parameter, but practice has led to relatively simple methods which provide reliable results.
FIGURE 2.22  Shear modulus values from primary loading and the first unload-reload cycle (from Mayu, 1987)
FIGURE 2.23 Shear modulus from unload-reload and reload-unload cycles (from Mayu, 1987)
There is greater experience with the SBPM in soft clays than in stiff clays. One of the problems in testing in stiff clays is the lack of reliability of the laboratory results due to fissures and slickensides. The Miocene clay in the Richmond area offers an ideal candidate for study since it is a stiff clay remarkably free of fissures and slickensides. A number of previous test programs have been performed on the Miocene clay, providing a sound data base for the test program proposed herein.
CHAPTER III  FINITE ELEMENT ANALYSES: SELF-BORING PRESSUREMETER TESTING IN CLAY WITH ANISOTROPIC HORIZONTAL STRESS FIELD

3.1 Introduction

In Chapter 2, Mayu's (1987) self-boring pressuremeter testing results for the Miocene clays were presented. Mayu used an SBPM with three strain arms that measured lift-off pressures independently. Testing results showed different lift-off measurements from the arms at any given depth. Mayu believed this indicated anisotropic lateral stresses, and to interpret the results, he assumed the lift-off pressure distribution around the probe varied in an elliptical pattern.

It was felt that the issue of anisotropic lateral stresses needed further study because: (1) There was no evidence provided that these conditions could not somehow change the process by which in situ stresses should be interpreted from a pressuremeter test (i.e., does the simple liftoff procedure work); and (2) The assumption of an elliptical variation of stresses did not allow for other possibilities. The finite element method was used to study the problem. This was followed by a reassessment of the Mayu tests as well as another well-documented set of SBPM tests.

3.2 Finite Element Studies - Codes Used

The finite element studies focused on saturated clays that do not drain under the short-term loading conditions obtained in pressuremeter testing. The analyses were performed with two finite element programs. The first one, known as
SOILSTRUCT, was developed by Dr. G. W. Clough and his colleagues. The program employs a nonlinear elastic soil stress-strain model that can represent the behavior of saturated soils reasonably so long as large scale failure of the soil mass or large strains does not occur. This restraint was acceptable for some of the cases considered herein since conditions focused on the liftoff of the membrane in the initial phases of the pressuremeter test. All the analyses using SOILSTRUCT in this study assumed the soil behaved in a linear-elastic mode.

As the study progressed, interest developed in how the presence of anisotropic lateral stresses would affect the behavior of an SBPM test under a full loading condition. Since this involves phases where the soil around the pressuremeter fails and undergoes large strains, it was necessary to use a code that could better deal with these conditions. To this end, the code known as EXCAVATE (Mana, 1978) was employed. EXCAVATE uses an elastic-perfectly plastic model and a classic plasticity formulation to handle the failure condition. For the undrained type failure considered in this study, the model provides an accurate accounting for soil behavior. EXCAVATE is also designed to allow for the effects of large strains.

3.3 Procedures For Pressuremeter problem Simulation - SOILSTRUCT and EXCAVATE Analyses

Simulation of a pressuremeter testing requires careful modeling of the testing procedures. Basic assumptions used in the finite element analyses of this study include:
1. The pressuremeter is inserted into a semi-infinite, homogeneous soil mass without disturbance.
2. During the probe expansion, the surrounding soils undergo plane strain deformation.
3. The soils respond symmetrically to the probe installation and expansion.

The latter assumption suggests that a finite element mesh consisting only one quarter of the horizontal cross section is adequate to model the problem.

The finite element meshes represent a horizontal cross section through the middle height of a self-boring pressuremeter (see Figure 3.1 and Figure 3.2). The outer soil boundaries, located distant from the probe, are subjected to a stress regime not influenced by the probe insertion or the membrane expansion process. The actual location of the soil outer boundaries was determined by a trial process based on the idea that different magnitudes of boundary pressure applied on the outer boundary should have little influence on the stresses near the central area around the probe. Based on the trials, the outer boundaries were set at a distance twenty-one times the radius of the probe.

As shown in Figure 3.1, the finite element mesh used for the SOILSTRUCT consists of 400 soil elements and 20 interface elements. Both the two-dimensional soil elements and the one-dimensional interface elements have four nodal points. All the interface elements are located around the periphery of the probe. Figure 3.2 shows the mesh used for the EXCAVATE analyses. Notably, the two-dimensional soil elements in the EXCAVATE program have eight nodes, and the one-dimensional interface elements have six nodes.
FIGURE 3.1 The finite element mesh used for SOILSTRUCT analyses
FIGURE 3.2  The finite element mesh used for EXCAVATE analyses
In the finite element simulation, the interface elements represent the boundary between the self-boring pressuremeter, membrane, and the surrounding soils. Notably, the one-dimensional interface elements have two distinct sides with two or three nodes per side, which represent different parts of the system. In reality, the rigid body of the SBPM is surrounded by a membrane which separates the probe from the soil. In the beginning of the test, the membrane is tight against the body of the SBPM. During the probe expansion, pressure is applied to the inside of the membrane, which, in turn, passes the stress on to the soil. Assuming for a moment that the membrane has zero stiffness when the pressure in the probe reaches the lateral pressure, the membrane will leave the probe and begin to push out into the soil. In the real problem, the membrane has some stiffness, but the stiffness effect is eliminated from the test results using a calibration technique. To simulate the process of the membrane expansion, the interface element plays two roles. The inner boundary of the interface element represents the rigid periphery of the probe and is unmovable. The outer boundary of the interface element represents the membrane and is expected to move in a normal direction outward from the probe. Membrane expansion is mobilized when the inside pressure exceeds the in situ lateral stress acting on the probe.

Procedures used in the finite element analysis to simulate the SBPM test are:

1. Initialize the soil and interface elements with zero stress.
2. Assign extremely high normal stiffness and shear stiffness to the interface elements. This is to simulate the perimeter of the stiff SBPM-membrane system as it is acted upon by the compressive lateral stresses of the soil during probe insertion.
3. Apply boundary pressures, $P_x$ and $P_y$ to the extremities of the finite element mesh to create the in situ horizontal stress effect. This step creates a stress field for the soil and interface elements. The normal
compressive stress in the interface element is then defined as the required lift-off pressure at the element location.

4. Apply incremental radial pressures to the outer boundary of the interface elements. This simulates the pressure application from within the probe. In the phase prior to membrane lift-off, the membrane will not move due to the extremely high interface stiffness and the applied pressures are not transferred into the soil elements.

5. As the radial pressures acting on the outer boundary of the interface element increase, the interface normal stresses are eventually reduced to near zero. This represents the condition just prior to membrane lift-off in the pressuremeter test. Upon reaching this condition, the normal stiffness and shear stiffness of the interface element are reduced to extremely low values. The outer boundary of the interface element then starts to move away from the inner boundary. This may be viewed as "opening" of a gap between the membrane and the probe. Numerically speaking, the finite element analysis cannot reach a precisely zero interface normal stress. Each interface element has to wait to reach a small tensile stress before it realizes the lift off should have occurred. A special process was implemented in the code to allow this condition to be achieved. Note that under anisotropic lateral stress condition, interface elements at different locations do not open simultaneously. However, if the in situ stresses are isotropic, all interface elements open at the same stress level.

6. Transferring of the probe pressure into the adjacent soil elements starts after the interface normal stiffness and shear stiffness are reduced to zero. The outside boundary of the interface moves into the soil as the probe pressure increases. Thereafter, the soil pressure-deformation response is initiated and the shear strength of the soil is mobilized.
3.4 Parameters Used for SOILSTRUCT and EXCAVATE

The Miocene clays have an overconsolidation ratio ranging from 4 to 5, and undrained shear strength ranging from 200 to 400 KN/M² (Mayu, 1987). The Young's modulus of the Miocene clay used for the finite element analyses was determined from the correlation shown in Figure 3.3. Since the soils were deformed under undrained condition, a Poisson's ratio close to 0.5 was used. For an isotropic stress condition with \( K_o \) equal to 1.0, the in situ lateral stresses \( P_X \) and \( P_Y \) are set to be 100 KN/M², which correspond to a testing depth of 5 meters. For anisotropic in situ lateral stress conditions, it was assumed that \( P_X \) was greater than \( P_Y \), with \( P_Y \) equal to 100 KN/M².

3.5 Illustrative Results of the Finite Element Analyses

In the finite element analysis, interface elements were used to simulate the lift-off mechanism of an SBPM test. Initially, the interface element experiences compression under in situ stress field. The computer simulation is modified to be "reactive," so an interface element has to reach a tension normal stress before it knows the lift-off has occurred. After the lift-off, the pressure increase is completely transferred through the interface element to displace the adjacent soil elements. For isotropic initial soil stress condition, all the interface elements lift-off simultaneously, and the lift-off pressure has the same magnitude as the actual in situ stress. An example of the pressuremeter curve is shown in Figure 3.4.

Figure 3.5 shows an example of the pressuremeter curves under an initially anisotropic in situ lateral stress condition. The computer program was modified to control the pressure increments during the initial pressurization stage so the
\[ E_u = KS_u \]

\[ E_u = \text{UNDRAINED MODULUS OF CLAY} \]

\[ K = \text{FACTOR FROM CHART ABOVE} \]

\[ S_u = \text{UNDRAINED SHEAR STRENGTH OF CLAY} \]

**FIGURE 3.3** Correlation for determining undrained Youn’s modulus of clay
(from Duncan and Buchignani. 1976)
Finite element analysis using SOILSTRUCT
Apply insitu boundary pressures
Px = Py = 100 kPa (assume linear elastic material)

Lift-off Pressure = 100 kPa
for all probe/soil interface

FIGURE 3.4 An example of a pressuremeter curve under an initially isotropic insitu lateral stress condition
Finite element analysis using EXCAVATE
Apply insitu boundary pressure Py = 100 kPa
and an anisotropic ratio Px/Py = 1.5

Lift-off Pressure
Pmax = 196 kPa

Lift-off Pressure
Pmin = 53 kPa

FIGURE 3.5 An example of pressuremeter curves under an initially anisotropic insitu lateral stress condition
interface elements lift-off progressively, with only one element lifting-off in each loading increment.

3.6 Discussions of Finite Element Analysis Results

For purposes of discussion, it is useful to delineate between two factors: an actual in situ anisotropic stress ratio in terms of the $P_X$ and $P_Y$ applied in the finite element analysis (assuming $P_X > P_Y$), and an interpreted in situ anisotropic stress ratio in terms of the lift-off pressure response of the SBPM test; i.e.

$$\text{Actual In Situ Anisotropic Stress Ratio (as exists at the distant boundary)} = \frac{P_X}{P_Y}$$

$$\text{Interpreted In Situ Anisotropic Stress Ratio (as exists at the probe boundary)} = \frac{P_{\text{max}}}{P_{\text{min}}}$$

where $P_X$ and $P_Y$ are the in situ lateral stresses applied to the extremities of the finite element mesh, and $P_{\text{max}}$ and $P_{\text{min}}$ are the maximum and minimum lift-off pressures in the interface elements, respectively.

Figure 3.6 shows that the interpreted and actual in situ stress ratios are in agreement only when the stress regime is isotropic. Where the in situ stresses are anisotropic, the interpreted anisotropic stress ratio is higher than the actual one. The difference between the two ratios increases as the in situ stresses become more anisotropic. The higher interpreted anisotropic stress ratio is caused by a stress concentration effect created by the presence of the pressuremeter probe in the stressed soil medium. This is an important finding and shows that the probe itself exaggerates the degree of the anisotropic stress field.

Figure 3.6 also shows a correlation between the actual and the interpreted anisotropic stress ratios. It is found that the Young’s modulus of soil has little
FIGURE 3.6 Correlation between the actual anisotropic stress ratio and the interpreted anisotropic stress ratio
effect on the correlation. Using the correlation, the SBPM results can be corrected to obtain the actual in situ lateral stresses. However, before using the correlation, the following question needs to be answered first: "How does one obtain the maximum and minimum lift-off pressures based on the strain arm lift-off measurements from an SBPM test?" Note that the strain arm directions may not align with those of the in situ principal lateral stresses.

The above question is essentially related to the lift-off pressures distribution around the probe. The self-boring pressuremeter used in this study has three sets of three strain arms 120 degrees apart. Using the lift-off pressure measurements, there are several methods to interpret the lift-off pressure distribution around the probe. Dalton and Hawkins (1982) proposed a method applying strain rosette theory as shown in Figure 2.10. Mayu (1987) applied a stress ellipse approach as shown in Figure 2.11. To find out which method is appropriate, the lift-off pressure distributions derived from both approaches are compared to those obtained from a finite element analysis. Figure 3.7 suggests that the interpretation using the strain rosette theory agrees with that from the finite element analysis. The stress ellipse approach results are not consistent with either of the two. In the remaining of this study, the strain rosette theory is used.

3.7 Determination of In Situ Lateral Stresses from an SBPM Test in an Anisotropic Stress Regime

Based on the previous discussions, the stress concentration effect should be considered in interpreting actual anisotropic in situ lateral stresses from an SBPM test in anisotropic clays. New procedures for the interpretation are defined as following:
FIGURE 3.7 Comparisons of different interpretation methods for normal stress distribution along the probe-soil interface
1. Obtain lift-off pressure measurements from individual strain arms.
2. Apply the strain rosette theory (Figure 2.10) to derive the maximum and minimum lift-off pressures around the probe.
3. Compute the interpreted in situ anisotropic stress ratio as the ratio of the maximum lift-off pressure to the minimum one.
4. Obtain the actual in situ anisotropic stress ratio, \( P_X/P_Y \) from the correlation shown in Figure 3.6.
5. Having the \( P_X/P_Y \) ratio, obtain the values of \( P_X \) and \( P_Y \) from the superposition procedures shown in Table 3.1. The coefficient of lateral earth pressure at rest, \( K_o \) can be estimated if the pore pressure and effective overburden pressure are available.

Dalton and Hawkins (1982) and Mayu (1987) provided lift-off pressure measurements of SBPM tests in soils where the in situ lateral stresses were believed to be anisotropic. The following sections re-evaluate the anisotropic in situ horizontal stress using the above new procedures.

### 3.7.1 Gault Clay: Interpretation of Dalton and Hawkins (1982) Test Results

Dalton and Hawkins (1982) carried out a series of SBPM tests in the Gault clay using a three-arm Camkometer. The testing site was a gently sloping grass meadow with over 40-meter thick of the Gault clay, and overlain by 3 meters of glacial till. The SBPM tests showed different lift-off pressure measurements in different radial directions. Dalton and Hawkins (1982) directly took the lift-off pressures as the in situ horizontal stresses, and applied the strain rosette theory to interpret in situ principal horizontal stresses. The results suggested an anisotropic


TABLE 3.1 Procedures of calculating actual insitu major and minor lateral stresses based on the lift-off pressure measurements of SBPM tests

<table>
<thead>
<tr>
<th>(P_y/P_y)</th>
<th>Probe/Soil Interface Pressures (Applying (P_y = 100) kPa and a given (P_y/P_y) ratio)</th>
<th>(P_{\max}/P_{\min})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>((P_{\max})_{\gamma = 100 \text{ kPa}} (\text{unit: kPa}))</td>
<td>((P_{\max})_{\gamma = 100 \text{ kPa}} (\text{unit: kPa}))</td>
</tr>
<tr>
<td>1.0</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>1.1</td>
<td>90</td>
<td>119</td>
</tr>
<tr>
<td>1.2</td>
<td>81</td>
<td>138</td>
</tr>
<tr>
<td>1.3</td>
<td>72</td>
<td>158</td>
</tr>
<tr>
<td>1.4</td>
<td>62</td>
<td>177</td>
</tr>
<tr>
<td>1.5</td>
<td>53</td>
<td>196</td>
</tr>
<tr>
<td>1.6</td>
<td>43</td>
<td>216</td>
</tr>
<tr>
<td>1.7</td>
<td>34</td>
<td>235</td>
</tr>
<tr>
<td>1.8</td>
<td>25</td>
<td>254</td>
</tr>
<tr>
<td>1.9</td>
<td>15</td>
<td>274</td>
</tr>
<tr>
<td>2.0</td>
<td>6</td>
<td>293</td>
</tr>
</tbody>
</table>

Procedures:

1. Based on the lift-off pressure measurements, interpret major and minor lateral stresses, \(P_{\max}\) and \(P_{\min}\) using the strain rosette theory.
2. Calculate the interpreted insitu anisotropic stress ratio, \(P_{\max}/P_{\min}\).
3. Referring to the table shown above, find the corresponding actual insitu anisotropic stress ratio, \(P_y/P_y\), and \((P_{\max})_{\gamma = 100 \text{ kPa}}\) (use linear interpretation, if necessary).
4. Define a scaling factor, \(F\) as
   \[
   F = \frac{P_{\min}}{(P_{\min})_{P_y = 100 \text{ kPa}}}
   \]
5. Calculate the actual insitu minor principal stress, \(P_y\), as
   \[
   P_y = F \cdot 100 \quad (\text{unit: kPa})
   \]
6. Calculate the actual insitu major principal stress, \(P_x\), as
   \[
   P_x = P_y \cdot \left(\frac{P_x}{P_y}\right) \quad (\text{unit: kPa})
   \]

where the \((P_y/P_y)\) ratio is obtained in the Step 3.
in situ lateral stress field with a $K_o$ value of 2.0 in the minor principal stress direction, and 5.5 in the major principal stress direction.

Using the lift-off pressure measurements of Dalton and Hawkins (1982), and applying the new interpretation procedures that account for the stress concentration effect, the interpreted $K_o$ values are reduced. As shown in Figure 3.8, the $K_o$ value becomes 3.0 in the minor principal stress direction, and 4.0 in the major principal stress direction.

3.7.2 Miocene Clay: Interpretation of Mayu (1987) Test Results

Mayu (1987) also took lift-off pressure measurements as in situ horizontal stresses without considering stress concentration effects, and used a stress ellipse assumption to interpret the in situ principal horizontal stresses. However, as discussed in the previous section, the strain rosette theory should be used for the principal horizontal stress interpretation. Table 3.2 shows the interpreted principal in situ lateral stresses from both the stress ellipse approach and the strain rosette theory. The strain rosette theory led to a much smaller major principal stress and a slightly higher minor principal stress. Accordingly, the interpreted anisotropic stress ratio is reduced.

Following the results from the strain rosette theory, Figure 3.9 shows the $K_o$ estimates based on the new interpretation procedures. Note the maximum $K_o$ values are now as much as 50% below than those interpreted by Mayu (1987). The re-evaluated $K_o$ values range from 4 to 14, which are still higher than what is normally expected. The $K_o$ values obtained from other testing or interpretation methods will be discussed in a later section.
FIGURE 3.8 Re-evaluation of Dalton & Hawkins (1982) SBPM testing data
TABLE 3.2 Interpretations of Mayu (1987) SBPM testing data using stress ellipse approach and strain rosette theory

<table>
<thead>
<tr>
<th>TEST</th>
<th>DEPTH (m.)</th>
<th>Lift-off Measurements (kPa)</th>
<th>Stress Ellipse Approach</th>
<th>Strain Rosette Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Arm1</td>
<td>Arm2</td>
<td>Arm3</td>
</tr>
<tr>
<td>R1</td>
<td>2.7</td>
<td>353</td>
<td>891</td>
<td>484</td>
</tr>
<tr>
<td>R2</td>
<td>3.6</td>
<td>413</td>
<td>1034</td>
<td>565</td>
</tr>
<tr>
<td>R3</td>
<td>4.2</td>
<td>390</td>
<td>997</td>
<td>542</td>
</tr>
<tr>
<td>R4</td>
<td>5.1</td>
<td>472</td>
<td>1176</td>
<td>624</td>
</tr>
<tr>
<td>R5</td>
<td>6.0</td>
<td>498</td>
<td>143</td>
<td>705</td>
</tr>
<tr>
<td>R6</td>
<td>15.4</td>
<td>915</td>
<td>1184</td>
<td>1467</td>
</tr>
</tbody>
</table>
FIGURE 3.9  Re-evaluation of Mayu (1987) SBPM testing data in the Miocene clays
3.7.2.1 Direction of the Principal Lateral Stresses

Mayu (1987) attributed the presence of the anisotropic in situ horizontal stress field in the Miocene clays to local topography effect. It was found that direction of the minor principal stress followed the slope at the site, pointing toward the James River. This argument seems reasonable, since the James River down cut through the region, removing soil and relieving stresses in the direction of the slope. Figure 3.10 shows the orientations of the major and minor horizontal stresses that were re-evaluated in this investigation. Note that the directional argument made by Mayu (1987) remains valid even after re-evaluating the magnitudes of the in situ principal stresses.

3.7.2.2 Comparison of $K_o$ From Different Testing Methods in the Miocene Clays

Following the Mayu (1987) testing program, Mullen (1991) performed dilatometer tests (DMT) in Miocene clays at the Schnabel center site. The dilatometer is a tool that relies on empirical correlations to interpret soil parameters. It is not assured that the dilatometer gives reliable results in the Miocene clays, because the database for developing the empirical correlations did not include unique soils, like the Miocene clays. However, it is useful to consider the results in a qualitative sense.

Figure 3.11 compares the $K_o$ values obtained from the DMT to those re-evaluated in this study. The DMT suggested the $K_o$ in the Miocene clay is about 1.0 at a depth of 1 to 2 meters. The $K_o$ increased to about 5.5 at a depth of 6 meters, and then linearly decreased to 2.0 at a depth of 9 meters. The $K_o$ values from the DMT and the SBPM tests at depths above 4 meters are not consistent in
FIGURE 3.10 Directions of the interpreted principal insitu lateral stresses at the Schnabel Center site (Based on Mayu (1987) SBPM testing results)
FIGURE 3.11 Comparisons of Ko values from SBPM testing and DMT in the Miocene clays
either magnitudes or trends. The SBPM tests yielded higher $K_o$ values than the dilatometer did, and the magnitude of $K_o$ decreased with depth. At the depths between 5 to 6 meters, the $K_o$ values from the DMT agreed with the minimum $K_o$ values estimated from SBPM testing. The only point of general agreement between the SBPM and dilatometer results is that the $K_o$ at the site is large relative to conventionally accepted values.

3.7.2.3  **Explanations of High $K_o$ Values in the Miocene Clays**

High $K_o$-value measurements are commonly reported in rock masses. Figure 3.12 presents $K_o$ values compiled from tests in rocks with different fault formations (Jamison and Cook, 1979). $K_o$ values greater than 10 are not unusual in rock formation. On the other hand, it is commonly assumed that the $K_o$ values in soils can be no greater than the passive earth pressure coefficient determined with zero cohesion. The rationale for this assumption is that if soil has no natural cohesion in the long term with full drainage, it will fail if the lateral stress reaches the passive pressure. Given the typical drained friction angles of most soils, the limiting lateral earth pressure coefficient is about 3. This tends to be supported by empirical data for typical soils (Mayne and Kulhawy, 1990).

How then can the high values of measured $K_o$ for the Miocene clays be explained? The possibilities are as follows:

1. Test error.
2. Problems in interpreting the test results.
3. An unusual condition which exists in the Miocene clays.
FIGURE 3.12  Ko measurements in rock masses with different faulting conditions (data from Jamison and Cook, 1979)
Test error is always an element of concern, but there are two reasons to believe this was not a major element. First, the SBPM tests yielded consistent results. There were no unusual jumps or perturbations in the data and all three arms suggested high lateral stresses. Second, Mayu (1987) calibrated his SBPM before and after testing, and the calibration was consistent.

As to problems of interpreting the test results, any errors due to the use of the stress ellipse theory and in accounting for stress concentration were accommodated. However, Hawkins et al. (1990) suggested that the SBPM tests may overestimate the $K_o$ value in stiff clays based on the lift-off pressure measurement. It is argued that the initial high tangent stiffness of the soil decreases substantially with strain. The strain level marking the end of the initial truly elastic region may be of the same order as the pressuremeter membrane and transducer corrections. As a result the pressuremeter test can not detect the true lift-off and reveals higher lift-off measurements.

Finally, the possibility of an unusual condition in the Miocene clay deserves full consideration. The Miocene clay differs from the norm in several aspects:

1. When a hole is opened in the clay, it closes rapidly enough to cause problems in withdrawal of equipment such as the pressuremeter.
2. Although the clay has a high OCR, it only rarely exhibits fissures. The fissures are typically only seen at large scale as in an open excavation. This suggests the clay has some form of natural cohesion that allows it to avoid passive failure.
3. If samples of the soil are extruded from a tube, it will "disc" and break into small cylindrical pieces as occurs in a highly stressed rock sample that is removed from its stressed environment.
Notably, one only needs to assume a drained cohesion of 40 KN/M² to allow for the measured K₀ value seen in the Miocene clay SBPM tests (assuming a drained friction angle of 40 degrees). Unfortunately, drained triaxial tests from Mayu (1987) did not support the existence of 40 KN/M² drained cohesion.

When all arguments are considered, there is no clear answers to the issue of the high K₀ values. It seems most likely to be due to some combination of how the interpretation is made and a set of unusual conditions in the soil.

3.8 Summary

This chapter addresses the issue of interpretation of measurements of in situ stresses in soils where the lateral stresses are anisotropic. The SBPM is one device with enough redundancy in measurement to allow anisotropic stress conditions to be defined. However, there are questions concerning how the measurements should be interpreted. First, there is matter of determining the principal stresses from the measured values given that the stresses measured are not necessarily the principal stresses. Second, questions exist about possible stress concentration that created during the insertion of a rigid probe into the soil. In this chapter, these issues are examined and solutions are proposed to be used in future SBPM testing. In addition, cases where anisotropic stresses have been measured are re-examined in light of the findings of the alternative interpretation methods. Some changes were found in the values of in situ stresses after re-interpretation, but there were no radically different results.

Consideration is also given to the matter of high values of lateral stress reported from SBPM testing by Mayu (1987) in the Miocene Clays found in the Richmond, Virginia area. After re-consideration of the results, the in situ lateral

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stress values were reduced, but they remained above those normally expected for stiff clays. In spite of the high lateral stress values, there is circumstantial evidence for high lateral stresses in the Miocene Clays. Considering all of the evidence, it is felt that the SBPM data are not unreasonable, but no conclusive support for this can be developed based on present data.
CHAPTER IV PRESSUREMETER TESTING PROGRAM IN MIocene CLay

4.1 Description of the Site

The test site for this investigation was located in a graveled area which serves as a parking lot for Schnabel Engineering Associates. The lot is bounded by Foushee and Canal Streets in Richmond, Virginia. The Miocene deposit, as described in Chapters 2 and 3, exists near the ground surface at this site, which makes it readily accessible for testing. Figure 4.1 shows the site map and test hole locations of this study and previous investigations. Note that the test holes for both Mullen (1991) and this study are at the west side of the site. The Mayu (1987) test holes were at the east side. Figure 4.2 shows the typical site soil profile, natural water contents, and standard penetration test results. The plasticity chart shown in Figure 4.3 is based on studies of Mullen (1991) and Mayu (1987). All Atterberg limit data tend to follow the "A" line. The data of Mullen (1991) cluster on the CH clay side of the "A" line, while the data of Mayu (1987) cluster on the MH, OH, silt side of the "A" line. It is believed that the Atterberg limit data from Mullen (1991) are representative for this testing program because his testing holes were closest to those of this work.

4.2 Testing Equipment

Figure 4.4 shows the layout of equipment used in this investigation. Most of the testing equipment was the same as that described in Mayu (1987), although a new pressuremeter probe and an upgraded data acquisition system were used. The following sections give brief descriptions of the equipment.
## FIGURE 4.1 Testing site and testing hole locations

<table>
<thead>
<tr>
<th>HOLE No.</th>
<th>TEST No.</th>
<th>TEST TYPE</th>
<th>HOLE No.</th>
<th>TEST No.</th>
<th>TEST TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>* R1 - R5</td>
<td>SBPM</td>
<td>9</td>
<td>** SNBC1</td>
<td>CPT</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>SPT</td>
<td>10</td>
<td>** SNBC2</td>
<td>CPT</td>
</tr>
<tr>
<td>3</td>
<td>R7 - R9</td>
<td>PBPM</td>
<td>11</td>
<td>** SNBC3</td>
<td>CPT</td>
</tr>
<tr>
<td>4</td>
<td>R10, R11</td>
<td>SBPM</td>
<td>12</td>
<td>** SNBD1</td>
<td>DILATOMETER</td>
</tr>
<tr>
<td>5</td>
<td>R12 - R15</td>
<td>SBPM</td>
<td>13</td>
<td>** SNBD2</td>
<td>DILATOMETER</td>
</tr>
<tr>
<td>6</td>
<td>R16</td>
<td>SBPM</td>
<td>14</td>
<td>** SNBD3</td>
<td>DILATOMETER</td>
</tr>
<tr>
<td>7</td>
<td>R17 - R23</td>
<td>SBPM</td>
<td>15</td>
<td>** SNBP1 - SNBP3</td>
<td>PBPM</td>
</tr>
<tr>
<td>8</td>
<td>R24</td>
<td>PBPM</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* From Mayu (1987)

** From Mullen (1991)
[NOTE] Values of PL and LL are from Mullen (1991)

FIGURE 4.2 Soil profile at the Schnabel Center site
FIGURE 4.3 Plasticity chart of the Miocene clays
FIGURE 4.4 Site layout of the pressuremeter testing equipment
4.2.1 Pressuremeter

The pressuremeter used in this work was designed in 1986 by a group consisting of Dr. G. W. Clough and Mr. J. Pappas of Virginia Polytechnic Institute and State University, Dr. J. Hughes of Hughes Insitu Engineering in Canada, and Mr. C. Dalton of Cambridge Insitu. The probe was manufactured by Cambridge Insitu, a firm specializing in SBPM development and testing. The pressuremeter module, shown in Figure 4.5, is sixty inches long and 2.75 inches in diameter. High pressure tubing and a data cable connect the pressuremeter module to a pressure control panel and an automatic data acquisition system. The pressuremeter consists of two main sections: an electronic module which includes equipment for signal conditioning; and an expansion section, which contains a total pressure cell, nine strain arms and the expandable membrane. The expansion unit has a length to diameter ratio 5.43, and is surrounded by a thick adiprene membrane. The nine strain arms are divided into three sets with three strain arms each, one near the upper third, one in the middle and one in the lower third of the expansion unit. The strain arms are used to measure the expansion of the membrane, and at each level the strain arms are spaced 120 degrees apart. For future discussion, the strain arms are numbered as arm 1 to arm 9 (see Figure 4.6). Arms 1-3 are in the top portion of the probe, arms 4-6 in the mid-plane, and arms 7-9 in the lower portion of the probe.

The electronic device section next to the expansion unit consists of a 16-channel analog multiplexer and an 8-bit microprocessor. The data from the analog strain arms and total pressure cell data are digitized, and electronically transmitted through a shielded cable to a laptop computer. This type of down-hole electronic device provides clean signals and reduces outside interference.
FIGURE 4.5 Schematic of the new nine strain arms SBPM (From Pappas, 1993)
FIGURE 4.6 Numbering system of the nine strain arms of the SBPM
4.2.2 Casing

To reach the testing depth, the pressuremeter module was attached to standard EX drill casing in sections of five-foot in length. The EX casing has an outside diameter of 1.5 inches.

4.2.3 Cutter Unit

The cutter unit consisted of a hydraulic cutter drive unit, cutter rods, and a cutter head. The cutter head is located at the tip of the pressuremeter, just inside the cutting shoe. The cutter head position with respect to the edge of the probe cutting shoe is adjustable. Inside the EX casing, there are inner cutter rods that connect the cutter head and the cutter drive unit. The hydraulic cutter drive unit attached to the top of the EX casing is to rotate the cutter rods and cutter head during the self-boring operation.

4.2.4 Loading Frame

The loading frame consisted of the following components:

1. A ground frame equipped with two 10-ton hydraulic jacks that displace vertically. The frame also holds the EX casing, to which the pressuremeter module is attached, and provides necessary downward thrust for the self-boring operation.

2. A system consisting of steel I-section beams and anchors. With anchors driven into the ground, this system provides reaction for the ground frame hydraulic jacking load during the self-boring operation.
Mayu (1987) provided detailed descriptions of the loading frame. When setting up the equipment, the level of the integrated loading frame system was checked, especially at the location where the frame attached to the EX casing and the pressuremeter. The intent was to insure that the pressuremeter advanced vertically.

4.2.5 Data Acquisition

The automatic data acquisition system consisted of:

1. The electronic section in the pressuremeter module.
3. A software package programmed in Microsoft Quickbasic for data communication.

Dr. T. L. Brandon in Virginia Polytechnic Institute and State University developed the software package to handle ten channels of data transmission, including nine strain arms and one total pressure cell data. The software can also send the testing data to a disk and display graphical stress-strain curves of the nine strain arms on the laptop computer screen in real-time. The real-time on screen display not only allows the operator to monitor the test, but also gives the operator a better control over loading and unload-reloading operations.
4.2.6 Pressure Control Panel

Nitrogen under pressure was used to expand the membrane. A needle valve regulated the nitrogen pressure at a controlled rate. The pressure control panel allowed regulation and visual monitoring of the pressures applied to the expansion unit of the pressuremeter. The data acquisition system recorded the pressures in the pressuremeter as well as the movements of the strain arms. A ball valve provided a means for rapid pressure release if needed.

4.2.7 Auxiliary Equipment

Auxiliary equipment used in this testing program included:

1. A hydraulic power unit. The unit mobilizes the two jacks of the ground loading frame and the cutter drive unit.
2. A three horse power water pump driven by a gas engine. The pump can pressurize the water up to 690 KN/M² (100 psi). The water is pumped through the cutter rods to the cutting shoe, mixed with soil cuttings, and then returns to the ground surface.
3. A command table consisting of several control valves. The valves control different types of operation during the self-boring process, including control of the hydraulic and water pressures in the system.
4. A 55-gallon tank as a water reservoir. A fine wire screen attached on the top of the tank screens soil cuttings from the returning wash water.
5. A high pressure nitrogen tank.
4.3 Pressuremeter Calibration

The pressuremeter was calibrated regularly to test the consistency of the instrumentation readings and provide calibration constants for the recording devices. Calibration of the voltage readings of the strain arms against displacement was achieved by mounting a micrometer over one of the arms and relating the indicated movement to voltage reading. When calibrating the pressure cell, the membrane was put into place on the probe. Next, a rigid protective cover with a diameter slightly larger than that of the pressuremeter was slipped over the probe. This apparatus served to restrain the membrane expansion after liftoff. With this arrangement, the pressuremeter was expanded under atmospheric pressure, and the responses of strain arm movements versus the applied pressures were monitored to define the effects of membrane stiffness. Since the adiprene membranes were not perfectly uniform, the nine strain arms yielded slightly different calibration curves that account for each case.

Special calibrations of the SBPM were performed after repairing or dismantling the probe, or if a new membrane was installed.

4.4 Testing Procedure

This investigation used two basic types of test approaches: (1) The pressuremeter was installed using a conventional self-boring process, and (2) The pressuremeter was installed into an open hole created by pushing thin-wall Shelby tubes. The second procedure was used to determine if this relatively simple method of installing a pressuremeter could be used to provide soil parameters that were comparable to those obtained by the self-boring technique.
4.4.1 Self-Boring Pressuremeter (SBPM) Tests

Proper operation of an SBPM test requires practice and care. One crucial task is to advance the probe to its testing depth with a smooth self-boring process, so the surrounding soils experience minimal disturbance. During the self-boring process, the following components should be controlled properly:

1. Rotation speed of the cutter drive unit.
2. Position of the cutter head relative to the cutting shoe of the pressuremeter.
3. Rate of the probe advance.
4. Pressure and quantity of the wash water.

Prior to conducting the tests at the Schnabel site, a series of practice tests were performed at a site near the Virginia Tech campus.

The self-boring operation in the Miocene clay used a constant advance rate of 2 inches per minute, and a cutter rotation speed of 2 cycles per second. These parameters were the same as those in the investigations by Mayu (1987). To assure that the probe advanced smoothly, the returning wash water and soil cuttings were frequently checked to see that no erratic patterns were developed.

The cutter head has to be positioned relative to the open end of the cutting shoe so that inward movement of the soil is prevented and the return of soil cuttings upwards through the cutting rods is smooth and continuous. In soft clays, Wroth and Hughes (1973) suggested the cutter head should be inside the cutting shoe a distance of at least one-third to one-half the diameter of the probe. In Mayu's tests with a 3.2-inch diameter pressuremeter in the Miocene clay, the cutter head was set 0.5 to 0.7 inch inside the cutting shoe. In this testing program, a 2.75-
inch diameter pressuremeter was used. Prior to using the probe at the Schnabel site, several trials were made at the site to decide an optimal cutter head position. It was found that if the cutter head was positioned inside the cutting shoe, the cutting shoe clogged frequently, and cuttings could not properly circulate with the flushing water. This could not be improved even by varying the cutter rotation speed or probe advance rate. To obtain a smooth self-boring process, it was found the cutter head should be set at 0.5 to 1 inch outside the cutting shoe. This seemingly unusual setting of the cutter head was necessary because of the smaller diameter of the new probe relative to those previously used.

Upon reaching a test depth, it is common practice in soft clays to hold the probe in place for a period before expanding the membrane. This allows dissipation of any possible excess pore pressure induced during the probe insertion. However, Mayu (1987) had experienced difficulties using this procedure when testing in the Miocene clay. He found if the probe stayed too long in the hole, the Miocene clay tended to close and grip the probe, making probe retrieval more difficult. The relaxation time used in this testing program was about 30 minutes, which yielded no difficulties in retrieving the probe.

In this investigation, the probe was expanded using stress control procedures. By regulating the nitrogen pressure through the pressure control panel, the probe expanded at a constant rate of 0.7 kPa per second during primary loading stages. However, the rate of unloading and reloading was more tightly controlled to ensure enough data points to define a smooth unload-reload loop.
4.4.2 Pre-Bored Pressuremeter (PBPM) Tests

If used in practice, the self-boring operation is not generally cost-effective because it is relatively slow and labor intensive. Thus, it is useful to find if a simpler technique can lead to the quality of results obtained using the self-boring operation. In this investigation, an alternative technique developed by Pappas (1993) was used. The testing holes were opened by pushing thin-wall Shelby tubes, which were only slightly larger than the pressuremeter. Using the same nine-arm Camkmeter as that used in the SBPM, the testing procedures for the Shelby tube holes were:

1. Drill a starter hole with a drill rig to a depth about sixty inches higher than the final testing depth.
2. Push a Shelby tube into the soil, and then remove it. This creates a clean, smooth hole about thirty inches long, and obtains soil samples for laboratory testing. The shove is carried out using the loading frame, which provides steady and smooth tube advance.
3. Repeat the last step. This extends the smooth hole to 60-inch long, which becomes enough to place the whole pressuremeter within the smooth hole.
4. Insert the pressuremeter into the borehole. Be sure that the 56-inch long module is inside the hole formed by the Shelby tubes.
5. Perform probe expansion following the same procedures of the SBPM test.
6. Retrieve the probe by pulling from the borehole after completing the test.

This testing procedure is similar to the Menard type pressuremeter test, but offers the following advantages:
1. The Shelby tube sampling equipment is readily available, and the diameter of hole formed by the Shelby tube is only slightly larger than that of the nine-arm Camkometer.

2. The hole formed by the thin-walled Shelby tube has smooth sidewalls. This reduces initial soil disturbance.

3. Valuable undisturbed soil samples for laboratory testing are available at the pressuremeter testing depth.

Tests performed using the Shelby tube procedure are referred to as pre-bored pressuremeter or PBPM testing.

4.5 Testing Program

The Miocene clays at the Schnabel site were previously investigated by Mayu (1987) and Mullen (1991). Mayu (1987) conducted five SBPM tests. Mullen (1991) conducted three cone penetration tests (CPT), three dilatometer tests (DMT), and three PBPM tests in holes opened by driving a 3-inch diameter sampler. This testing program, including four PBPM and 14 SBPM tests, was conducted from October through December in 1988. Basic information about the tests performed in this investigation is shown in Table 4.1. All test hole locations, including those of Mayu (1987) and Mullen (1991) are shown in Figure 4.1.
TABLE 4.1 Pressuremeter tests in the Miocene clays; Schnabel center site

<table>
<thead>
<tr>
<th>TEST No.</th>
<th>HOLE No.</th>
<th>DEPTH (m.)</th>
<th>TYPE OF TEST</th>
<th>DATE OF TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>R7</td>
<td>3</td>
<td>4.0</td>
<td>PBPM</td>
<td>Oct. 24, 1988</td>
</tr>
<tr>
<td>R8</td>
<td>3</td>
<td>4.6</td>
<td>PBPM</td>
<td>Oct. 24, 1988</td>
</tr>
<tr>
<td>R9</td>
<td>3</td>
<td>5.8</td>
<td>PBPM</td>
<td>Oct. 31, 1988</td>
</tr>
<tr>
<td>R10</td>
<td>4</td>
<td>4.3</td>
<td>SBPM</td>
<td>Nov. 7, 1988</td>
</tr>
<tr>
<td>R11</td>
<td>4</td>
<td>6.7</td>
<td>SBPM</td>
<td>Nov. 8, 1988</td>
</tr>
<tr>
<td>R12</td>
<td>5</td>
<td>4.0</td>
<td>SBPM</td>
<td>Nov. 19, 1988</td>
</tr>
<tr>
<td>R13</td>
<td>5</td>
<td>4.6</td>
<td>SBPM</td>
<td>Nov. 20, 1988</td>
</tr>
<tr>
<td>R14</td>
<td>5</td>
<td>5.5</td>
<td>SBPM</td>
<td>Nov. 20, 1988</td>
</tr>
<tr>
<td>R15</td>
<td>5</td>
<td>9.2</td>
<td>SBPM</td>
<td>Nov. 22, 1988</td>
</tr>
<tr>
<td>R16</td>
<td>6</td>
<td>3.7</td>
<td>SBPM</td>
<td>Nov. 25, 1988</td>
</tr>
<tr>
<td>R17</td>
<td>7</td>
<td>3.7</td>
<td>SBPM</td>
<td>Nov. 26, 1988</td>
</tr>
<tr>
<td>R18</td>
<td>7</td>
<td>4.6</td>
<td>SBPM</td>
<td>Nov. 25, 1988</td>
</tr>
<tr>
<td>R19</td>
<td>7</td>
<td>5.5</td>
<td>SBPM</td>
<td>Nov. 28, 1988</td>
</tr>
<tr>
<td>R20</td>
<td>7</td>
<td>7.3</td>
<td>SBPM</td>
<td>Dec. 3, 1988</td>
</tr>
<tr>
<td>R21</td>
<td>7</td>
<td>8.2</td>
<td>SBPM</td>
<td>Dec. 3, 1988</td>
</tr>
<tr>
<td>R22</td>
<td>7</td>
<td>9.2</td>
<td>SBPM</td>
<td>Dec. 3, 1988</td>
</tr>
<tr>
<td>R23</td>
<td>7</td>
<td>10.1</td>
<td>SBPM</td>
<td>Dec. 4, 1988</td>
</tr>
<tr>
<td>R24</td>
<td>8</td>
<td>4.6</td>
<td>PBPM</td>
<td>Dec. 4, 1988</td>
</tr>
</tbody>
</table>

NOTE: PBPM - Pre-bored pressuremeter test
      SBPM - Self-boring pressuremeter test
4.6 Experience From the SBPM and PBPM Testing

4.6.1 Self-Boring Pressuremeter Tests

During the SBPM test, the self-boring operation is critical. Special care is required to obtain good quality testing results. The trials performed in this investigation, both in tests at the Virginia Tech site and at the Richmond site, allowed the crew to develop a smooth process.

In all but one of the tests of this investigation, physical indications suggested that the self-boring process was appropriate with consistent patterns of return of wash water and soil cuttings. However, at the final self-boring stage of the test R16, the returning wash water showed no signs of soil cuttings. In spite of this, the probe still advanced at a rate of 2 inches per minutes. Subsequently, it was found that the inner cutter rod had become disconnected from the cutter head, probably due to reversing the cutter rotation direction during the probe advance. In such a condition, with soils clogged inside the cutting shoe, the probe must have advanced as a full displacement type of pressuremeter, leading to soil disturbance. The expansion phase of the test was still performed with the thought that it would provide useful insight to the influence of soil disturbance effects on the test results.

After reaching the testing depth, performance of the probe expansion was straightforward. The downhole electronic data acquisition system provided clean signal transmittal, and the computer real-time monitoring allowed the operator to have good control of loading and unloading operations.
Lessons learned from the SBPM testing program that are useful to note are:

1. The cutter rods must be properly tightened during installation.
2. Rotation of the direction of the cutter rod during insertion can unscrew the cutter head from the cutter rod.
3. Careful observation of the wash water and soil cuttings return can provide valuable signs of problems during insertion.

4.6.2 Pre-Bored Pressuremeter Tests

Lessons learned while performing the PBPM tests include:

1. The procedures used in preparing the PBPM test hole were relatively easy to perform and more efficient than self-boring.
2. It was difficult to assure that the pressuremeter was seated in the center of the test hole, and as a result, all strain arms do not simultaneously touch the sidewalls of the testing hole in the beginning of the test.
3. The nine-arm Camkometer was only partially adaptable for PBPM testing. The major problem occurred while pressurizing the probe, the topmost portion of the membrane could push upwards against the upper clamp of the probe, and ultimately deflated.

4.7 Summary

Recently, the Miocene clays at the Schnabel site in Richmond, Virginia were investigated by Mayu (1987) and Mullen (1991) using different in situ testing
techniques. This study focused on pressuremeter tests at the same site. While many of the procedures used in this work were similar to those of Mayu, differences existed as well. One of these related to the use of a new self-boring pressuremeter which included features such as: nine strain arms to measure membrane movement, an advance data acquisition system, and an on-screen pressuremeter curve monitoring system.

Two types of testing procedures were adopted in this study: (1) Self-boring pressuremeter (SBPM) tests. In this case the pressuremeter probe self-advanced to testing depth, and induced minimal soil disturbance. (2) Pre-bored pressuremeter (PBPM) test, where the pressuremeter was inserted into a test hole created by pushing thin-wall Shelby tubes.
CHAPTER V  MIOCENE CLAY: PRESSUREMETER TESTING RESULTS

5.1 Introduction

This chapter presents the results of the pressuremeter testing program. The nine-arm pressuremeter described in Chapter 4 was used throughout the test program, and it generated nine sets of pressuremeter curves (a plot of applied pressure versus cavity strain) for each test. Figures 5.1 and 5.2 show typical pressuremeter curves derived from a PBPM and an SBPM tests respectively. The pressuremeter curves are plotted in three sets, with each set representing data from three strain arms lying in a common vertical plane (hence, all measure radial movements in the same direction). The arm numbering scheme is such that the smallest numbered arm is always the one positioned near the top of the probe. The intermediate numbered arm lies in the center plane of the probe, while the highest numbered arm is located near the bottom of the probe. For example, in the set 1, 4, and 7, arm 4 is in the mid-plane while 1 is near the top and 7 near the bottom of the probe. This type of presentation allows one to observe if the arms on different sides of the probe behave in a similar, or different fashion. An alternative form to present the data is to combine the arms at the same elevation in one plot. Subsequently, this form is used, and is needed to reduce the data from the test to obtain soil parameters.

For purposes of documentation, the appendix of this report gives the pressuremeter curves from all the tests.
FIGURE 5.1 Typical results of PBPM testing in the Miocene clay
FIGURE 5.2 Typical results of SBPM testing in the Miocene clay
5.2 Comments on Typical PBPM Results

When conducting the PBPM test, a smooth test hole was formed by pushing cylindrical thin walled Shelby tubes into the clay and removing the soil. The smooth test hole was long enough to allow the pressuremeter fully embedded in the soil and minimize end effects during pressuremeter testing. Because the pressuremeter was about 6 mm. smaller in diameter than the hole created by the Shelby tube, the probe did not contact sides of the hole when starting the test. Results of the PBPM test R7 (Figure 5.1) show that arms 1-4-7 and 2-5-8 follow a common expansion pattern. The membrane expanded radially outward in the beginning of the test, and met little resistance. The lack of resistance in this early phase of the test was due to the presence of a gap between the membrane and the side walls of the hole. Once the membrane contacted the sides of the hole, the increase of the cavity strains rapidly slowed as pressure increased. The differences in the response before the membrane made contact with the hole, and after it did, led to a reverse curvature in the early stage of the pressuremeter curves of the arms 1-4-7 and 2-5-8 in the test R7. In contrast, in the same test, the pressuremeter response for arms 3-6-9 did not clearly show the reverse curvature as the other arms did. Apparently, in test R7, the side of arms 3-6-9 was initially in closer proximity to the probe borehole wall than the other sides, or it was pushed closer by early expansion of the membrane on the other side of the probe. This response suggests that the probe was not perfectly centered in the test hole in the beginning of the test, or that the membrane did not have a uniform stiffness. The former is more likely the primary contributor since the probe could not be accurately centered because of the nature of the hole opening and probe positioning procedures.

Although the probe could not be perfectly centered in the hole in the PBPM tests, the actual differences in movement from one side of the probe to the other were not large. Referring to the PBPM testing results shown in the Appendix,
cavity strains required to reach the sides of the borehole ranged from 0% to almost 4%. However, 4% cavity strain corresponds to only a 1.4 mm. membrane radial movement. Interestingly, the ability to assess the difference in the early stage of movements was only possible by virtue of having the nine strain arm measurements.

After the membrane fully contacted the borehole wall in the PBPM tests, the pressuremeter curves from the different arms had similar appearances, and reached about the same peak pressure value. Slopes of the initial loading curves and the unload-reload curves were similar in all cases. Thus, in spite of the difference in the initial portions of the curves, the soil response to full contact loading was similar. The measured unload-reload response in the PBPM tests showed a unique behavior in that the "loops" formed in the tests were "open" as opposed to "closed" as is more common. The open loop occurred since the hysteretic response was very strong. This is thought to be related to the unique sensitivity of Miocene clays.

5.3 Comments on Typical SBPM Results

Comparing Figures 5.1 and 5.2, differences in pressuremeter curves for the SBPM and the PBPM tests are: (1) There is little sign of a reverse curvature of the early phases of the SBPM pressuremeter curves; (2) The slope of the initial loading portion of the pressuremeter curve is higher in the SBPM test than that of the PBPM test; and (3) In general, the unload-reload loops for the SBPM tests are closed as opposed to the open form for the PBPM tests. The latter two trends show that the Miocene clay responds differently under different testing procedures, especially the test hole preparation technique. The relative lack of reverse curvature in the early phases of the SBPM test is expected since the self-boring process should leave the probe in tight contact with the hole. However, what was unexpected was the fact that some SBPM tests did show pressuremeter curves with
a limited degree of reverse curvature. This suggested that some form of gap existed between the pressuremeter and the soil before the expansion of the membrane began.

In Figure 5.2, the reverse curvature was most obvious in the pressuremeter curves of arms 2-5-8. Results of the SBPM tests show that the cavity strain reached a maximum of 2% (0.7 mm. of membrane radial movement) before the membrane contacted the borehole walls. A review of the pressuremeter curves from all of the SBPM tests shows that the reverse curvature consistently occurred at the location of arms 2-5-8. In some instances, the reverse curvature also occurred at other sets of arms, but it was not nearly as consistent, nor as pronounced.

After giving thought to the possibilities for the reverse curvature, it was postulated that some form of probe misalignment may have occurred. To investigate this possibility, a complete inspection of the probe was made. Visual inspection of the probe did not reveal any problems, which explains why no warnings existed during the actual field testing program. However, a careful measurement program showed that at the coupling connecting the probe to the casing, a slight bend from the vertical existed. This bend would cause the probe to tilt away from the vertical as it entered the ground. It is likely that the tilt would become more severe in the field due to the effects of the vertical reaction force applied to the probe during the insertion. The tilt would lead to a gap between the probe and the soil as shown in Figure 5.3. While the nature of the gap can be defined theoretically, aberrations could easily occur in its shape due to closure, collapse or erosion. Erosion would occur if the drilling return water was able to escape locally under the cutting shoe. Such gap irregularity was observed from the pressuremeter curves. In most cases, the maximum gap did not occur at the top of the pressuremeter.
Section A-A
Top view of the borehole

EX drill casing

Borehole sidewall

Pressuremeter probe

NOTE: Graph not to scale. Exaggerated for clarification.

FIGURE 5.3 Misalignment effect of the pressuremeter probe
Theoretically, the opening created by the tilting probe advancement would be elliptical in plane. Referring to the patterns of the pressuremeter curves, the maximum gap likely occurred near the side of arms 2-5-8. On the opposite side of the probe from arms 2-5-8, the probe should have been forced into hard contact with the soil. Positioned at 60 degrees from the side of arms 2-5-8, the sides of arms 1-4-7 and arms 3-6-9 were between "maximum gap" and "hard contact" conditions. This probably explains why some of the arms 1-4-7 and 3-6-9, but not all, tended to show a small reverse curvature.

The tilt of the probe made it impossible to interpret in situ stress from the SBPM test results because the membrane lift-off was not well defined. However, the tilt ultimately gave an insight to how a small degree of disturbance affected the interpretation of other soil parameters. This will be further addressed in subsequent sections. It is important that the presence of the tilt could be anticipated only because the probe had nine strain arms which could be independently monitored. This illustrates the utility of having multiple points of membrane movement measurement.

5.4 Shape of Pressuremeter Membrane in the Expansion Process

Having three strain arms at each elevation, the nine-arm pressuremeter gives an insight to the shape of the membrane during expansion. To quantify the membrane shape variability, a parameter, termed deviation percentage, "DP" is used. The deviation percentage presents deviation between individual arm displacement and the average of all the arm displacements; i.e.:
\[ DP. = \frac{(\text{Individual Arm} - \text{Average Arm Displacement})}{\text{Average Arm Displacement}} \times 100\% \]

If the deviation percentages of all strain arms are close to 0\%, it suggests that the membrane expands uniformly and has a shape close to a perfect circular cylinder. Figures 5.4 and 5.5 show deviation percentages of all the SBPM tests (Tests R10 to R23). The figures present the data at four pressure levels: 20, 50, 75 and 100\% of the maximum applied pressure, \( P_{\text{max}} \). The results lead to the following comments:

1. At the 25\% \( P_{\text{max}} \) level, deviation percentages are the largest, and reach as high as 250\%.
2. Deviation percentages decrease with increasing level of loading.
3. At 100\% \( P_{\text{max}} \) level, deviation percentages are typically less than 50\%.

Because the deviation percentages are non-zero at all pressure levels, it can be concluded the membrane does not expand in a perfectly cylindrical shape. However, it is important to note that the difference in absolute magnitude of membrane deformations is never large, particularly in the early phases of movement when the DP values are the largest. As an example, at 25\% of final loading, movements are less than 0.5 mm; even in the latter phases of the test, movements are no more than 6 mm. Thus, a higher percentage deviation at lower pressure level does not equate to a greater difference in the absolute movement.

In part, some of the large DP values in the early stages of the test can be attributed to the gap that was created as the probe entered the ground. This effect gradually became less important as the expansion process progress as is evidenced by the reduction in DP values at higher pressures.
FIGURE 5.4 Deviation percentages represent uniformity of pressuremeter membrane expansion in Tests R10 to R16.

LEGEND
A - 25% of Pmax
B - 50% of Pmax
C - 75% of Pmax
D - 100% Pmax
FIGURE 5.5 Deviation percentages represent uniformity of pressuremeter membrane expansion in Tests R17 to R23
5.4.1 Shape of the Vertical Profile of the Membrane

In the past, it has been suggested that the restrained ends of the membrane may affect the vertical shape of the pressuremeter expansion. Studies of this issue for the MPM were carried out in the laboratory using radiographic methods (Ohya et al., 1983), and for the SBPM using finite element methods (Yeung and Carter, 1990). The deformation measurements from the nine-arm pressuremeter at different loading stages can help visualize the vertical membrane profile. Figures 5.6 and 5.7 show membrane movement data in a format which allows the membrane shape along different vertical axes for SBPM tests R13, R15, R18, and R22 to be studied. Conclusions from the data are:

1. Deformations measured from the top, center, and bottom strain arms along a certain vertical plane are similar. There is no evidence suggesting that the restraints of top and bottom membrane collars significantly influence the behavior of top and bottom strain arms.

2. Different sides of the membrane do not always expand in the same pattern or by the same magnitude.

3. The membrane shape at lower pressure levels seems more consistent than that at the higher pressure levels. (Note this is true even though a higher deviation percentage occurs at the lower pressure levels.)

4. In some tests, e.g., test R18, the membrane near the bottom of the probe bulged at final loading stage. Reasons for this are not clear. Ohya et al. (1983) found such a bulge occurred consistently for a Menard type of pressuremeter, but this was a result of having different water pressures inside the probe, a mechanism not applicable to the gas-expanded SBPM.

5. The order of deformation magnitudes along different vertical axes at the final loading stage does not necessarily follow that at the early
FIGURE 5.6 Vertical membrane expansion profiles of Tests R15 and R22
FIGURE 5.7 Vertical membrane expansion profiles of Tests R13 and R18
loading stage. Thus, the arm that moves first, may not be the one moves the most at the end of the test. This implies that factors affecting the initial soil response become less significant in the final stages of testing.

The latter observation is significant in that the arms that moved the first usually were in the 2-5-8 plane where the gap existed between the probe and the soil. Apparently after some expansion had taken place, the movements of the other arms "caught up" with those in the 2-5-8 plane. Subsequently this behavior will be further discussed in the assessment of soil parameters from the results of the different strain arms.

5.4.2 Shape of the Membrane in Horizontal Section

During the self-boring operation, a vacuum was applied inside the pressuremeter probe to insure that the membrane was not pulled away from contact with the probe. Therefore, it is safe to presume the shape of the membrane was cylindrical before expansion. Assuming the soil to be isotropic and homogeneous, membrane expansion upon pressurization should proceed in circularly in a horizontal cross section. However, reviewing the expansion test data of this investigation, it was found that this pattern was not followed in the field. It is useful to ask, "What was the membrane shape in the horizontal plane, and why was it not circular?" These questions can be explored using the data from the nine-arm pressuremeter. At each level of the nine-arm pressuremeter, three lateral movement measurements are obtained that are in radial directions 120 degrees apart. Given that the three lateral movement measurements are not sufficient to define a singular shape for the membrane, no exact answer may be provided for the
actual shape. However, some insights can be developed reviewing the measurements.

It is useful to first consider the possibility that, in spite of differences in movements from one side of the probe to the other, the shape of the membrane somehow remained circular at all levels during expansion. Because the strain arm movements are not uniform at any level, a circle can only be fitted to the data if the center of the membrane is assumed to shift during testing. Applying this assumption for strain arms at all levels, it is found that the centers of the fitted circles shift in different directions and amounts. It seems unreasonable that different parts of the membrane could move in such patterns. Therefore, it appears more likely that the membrane did not expand following a perfect circular shape.

It is difficult to define rigorously the nature of a noncircular horizontal membrane cross section. The simplest option is to assume that the shape was elliptical. Based on fitting observed movement data, Jerzy and Ginter (1977) suggested a method for fitting ellipses to define coal mine opening shapes. They verified that the method was reasonable based on case history data. Their fitting method is applied to the SBPM testing data in this study. The procedures are:

1. In a horizontal plane, define a distance component as the distance between the membrane and the center of the probe; i.e., a strain arm deformation plus the probe initial radius.
2. Assume the center axis of the pressuremeter does not move during the test.
3. Using the equations shown in Figure 5.8 and the data in step 2, derive an ellipse to fit the data.
(x,y) coordinate system with x chosen arbitrarily parallel to Arm 1. 
(X,Y) coordinate system with X and Y respectively parallel to $\epsilon_1$ and $\epsilon_2$, the major and the minor principal total lateral soil pressures in the horizontal plane.

Equation of the ellipse in the (X,Y) coordinate system

$$\frac{x^2}{(\epsilon_1)^2} + \frac{y^2}{(\epsilon_2)^2} = 1$$

Equation of the ellipse in the (x,y) coordinate system

$$x^2 \left[ \frac{\cos^2 \alpha}{(\epsilon_1)^2} + \frac{\sin^2 \alpha}{(\epsilon_2)^2} \right] + y^2 \left[ \frac{\sin^2 \alpha}{(\epsilon_1)^2} + \frac{\cos^2 \alpha}{(\epsilon_2)^2} \right] + 2xy(\cos \alpha)(\sin \alpha) \left[ \frac{1}{(\epsilon_1)^2} - \frac{1}{(\epsilon_2)^2} \right] = 1$$

or

$$0Ax^2 + By^2 + Cxy = 1$$

$$\begin{bmatrix} x_1^2 & y_1^2 & x_1 y_1 \\ x_2^2 & y_2^2 & x_2 y_2 \\ x_3^2 & y_3^2 & x_3 y_3 \end{bmatrix} \begin{bmatrix} A \\ B \\ C \end{bmatrix} = \begin{bmatrix} 1 \\ 1 \end{bmatrix}$$

where ($x_1$, $y_1$), ($x_2$, $y_2$) and ($x_3$, $y_3$) are the coordinates of the total lateral pressures measured by Arm 1, Arm 2 and Arm 3 respectively. Solve for A, B and C. Then, solve for $\alpha$, $\epsilon_1$, and $\epsilon_2$ with

$$\tan 2\alpha = \frac{C}{A-B}$$

$$\frac{1}{(\epsilon_1)^2} = A\cos^2 \alpha + C(\sin \alpha)(\cos \alpha) + B\sin^2 \alpha$$

$$\frac{1}{(\epsilon_2)^2} = A\sin^2 \alpha - C(\sin \alpha)(\cos \alpha) + B\cos^2 \alpha$$

**FIGURE 5.8** Ellipse approach for determining shape of the SBPM membrane expansion
The magnitude of the deformation measured by a strain arm is much smaller than the initial probe radius. This makes it difficult to visualize the projected shape of the pressuremeter membrane in horizontal cross section if depicted using a true scale. To help visualize the predicted shape of the deformed pressuremeter membrane, a magnification factor of 1000 was used for plotting purposes. Figure 5.9 shows examples of the interpreted horizontal cross sections at different probe elevations for two SBPM tests at the final loading stage. In test R12, the interpreted ellipses at different elevations are similar in shape, and the long and short axes are in a consistent alignment. On the other hand, in test R17, there is little agreement for the ellipse alignment or shape at different levels. Of these two tests, R12 is more typical of the other SBPM tests in the Miocene clay.

There are several reasons for a non-circular membrane expansion in SBPM tests: (1) Hole shape defects caused by the probe misalignment; (2) An anisotropic soil stiffness or strength; and (3) An anisotropic stress field in the soil. The first reason would influence the initial membrane expansion shape, but as shown, it does not affect the response at final loading stages. Since the elliptical shapes are maintained to the end of the test, it seems that hole shape defects do not serve to explain the observed membrane behavior. On the other hand, the latter two factors may act alone, or in concert to affect both the initial and final membrane shape. In most alluvial or marine soils, these factors would not be unexpected. If these conditions prevailed, it suggests that the elliptical shape would reasonably represent the deformed membrane.
TEST R17; DEPTH = 3.7 m.; P = 1067 kPa = 100% P max

NOTES: (1) Deformations were magnified 1000 times
(2) Use elliptical fitting

FIGURE 5.9 Interpretation of membrane horizontal cross sections

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5.5 Interpreting the Results in Terms of Anisotropic In Situ Lateral Stresses

As described in Chapter 2, Mayu (1987) performed five SBPM tests in the Miocene clay at the Schnabel site using a three-arm SBPM. He had no apparent problems with probe alignment or the self-boring process, and his pressuremeter curves showed clear lift-off pressures. The test data led him to suggest that there was anisotropy in the in situ lateral stresses at the Schnabel site. His data suggested that the direction of the minor principal lateral stress at the site pointed toward the James River (Figure 5.10), and followed the slope at the site. This seemed consistent with the notion that there was a stress relief effect caused by the James River cutting through the soils at the site. The major principal stress was perpendicular to the minor principal stress, or in an alignment roughly parallel to the crest of the slope.

The present test data cannot be used directly to check Mayu's hypothesis because the lift-off pressures were not clear. However, it seems reasonable that the shape of horizontal membrane cross section may give information related to the directions of in situ principal lateral stresses. It should follow that the major principal horizontal stress direction agrees with the direction showing the smallest membrane deformation in a pressuremeter test. Thus, if we have an elliptical membrane deformation, the short axis of the ellipse would track the direction of the highest lateral stress. To examine this possibility, all data points representing the directions of the ellipse axis having the smallest deformations of all the SBPM tests we compiled. Note that the orientation of the pressuremeter during the tests was pre-set, so that absolute direction of the strain arms was known. This allows one to obtain the absolute orientation of the axes of the ellipse fitting to the strain arm data.
FIGURE 5.10 Directions of the principal insitu lateral stresses at the Schnabel Center site
In this testing program the pressuremeter was oriented such that the plane containing arms 3-6-9 was very close to the major principal lateral stress direction proposed by Mayu. If there is perfect match between the short deformation axis and the hypothesis of Mayu, the short axis should follow the arms 3-6-9 orientation. Figure 5.9 shows the results for test R12, where the short deformation axis does consistently follow the orientation of arms 3-6-9. Figure 5.11 presents a plot of the directions of the short axis data points for all the tests. The figure shows that the majority of the short axis orientation data tend to accumulate about the arms 3-6-9 axis. While this supports the Mayu (1987) thesis, there is also a smaller set of data clustered around the orientation of arms 1-4-7. To be exact, fifteen observations lie in the direction close to arms 3-6-9, while nine observations are closer to the arms 1-4-7 direction. Only one observation is close to the direction of arms 2-5-8.

The direction of arms 1-4-7 is close to the direction of the slope at the Schnabel site. This is essentially opposite to the expected direction of the major principal stress if the Mayu hypothesis is correct. In the end, the data are too contradictory to draw a firm conclusion that whether the lateral stress field is clearly anisotropic, although the majority of the data supports the hypothesis proposed by Mayu (1987).

5.6 Undrained Shear Strength

Chapter 2 reviews several methods for interpreting undrained shear strength from pressuremeter test results. These include the Menard, Gibson and Anderson (GA) and the Baguelin, Ladanyi and Palmer (BLP) interpretations. The Menard method cannot be used in this study, because the SBPM tests did not reach volumetric strains large enough to define limit pressures. The BLP method requires the pressuremeter curve have a clearly defined strain datum, and the problems
FIGURE 5.11 Observations of the membrane short deformation axis of the pressuremeter tests in the Miocene clays
induced by probe alignment ruled out its use. Fortunately, the GA interpretation method can be readily applied in this study. The GA method has been reported as the most reliable approach for stiff clays (Wroth, 1984; Mair and Wood, 1987).

In the GA interpretation, the results of a pressuremeter test are plotted as the natural logarithm of the volumetric strain plotted versus the applied pressure. The undrained shear strength is determined using the GA formula, and the gradient of the straight line portion of the GA diagram. Figure 5.12 shows GA diagrams for four of the SBPM test results using the data each of the three strain arms at the mid plane of the probe (thus, there are three curves for each test in Figure 5.12). The results are complex because the curves, at least in their initial phases, are apparently not consistent in shape. Some strain arms show rapidly increasing strains under very small pressures, but others require higher pressures to initialize soil strain response. These differences in behaviors can be explained in terms of the previously discussed probe alignment problems. The strain arms at the location with an initial gap between the probe and the soil, e.g., arm 5, show rapid strain increase in the beginning of the test. Those on the side with the probe more in contact with the soil show the opposite trend. Importantly, in all cases, when the pressuremeter curves are well past the initial portions, they tend to have the same slope and merge at the highest pressure levels. This shows that straining the soil to a certain point eliminates the effects of the initial conditions around the probe.

The principal issue in using the data in a GA interpretation revolves around where to choose the gradient for the plot. Notably, the smaller the gradient, the higher the prediction of undrained shear strength. For discussion, it is of interest to focus first on the behavior of arm 4 in tests R19 and R22. Note that arm 4 was located on the side with good probe and soil contact after self-boring, and its pressuremeter curve showed no initial reverse curvature. For tests R19 and R22, the pressuremeter curves of this arm exhibit a classical form of behavior in the GA
FIGURE 5.12 GA diagrams from middle set arms readings of SBPM tests in Miocene clay
concept. The data follow an upward path until the soil begins to shear, and then a concave curved section is observed. The initial curved portion is then followed by a straight line with essentially a single gradient. This gradient provides the information needed for the GA interpretation. The straight line for arm 4 also provides a base for judging data from the other arms. In each case, the data for arms 5 and 6 gradually merge with that from arm 4. However, the merging of the data of arm 5 occurs in a later stage than the data for arm 6. This response is consistent with the knowledge that arm 5 is located on the side adjacent to the gap between the pressuremeter and the soil. In this case, the pressuremeter curve and the GA curve show strong reverse curvature effect. Arm 6 was in a condition between those of arms 4 and 5, and the data reflect this state.

It is notable that once the mobilization of the shear strength is complete, the GA diagram for arm 4 shows little, if any, change in slope. This implies that any strain softening that might have occurred in these sensitive clays was accommodated early in the test. The merging of the pressuremeter and GA curves from arms 5 and 6 with 4 implies that the strength is uniform in all directions.

Focusing now on data from tests R17 and R18 data, it is found that the arm 4 pressuremeter curve begins in the same way as it did for tests R19 and 22. However, in the latter portions of the curves for tests R17 and R18, the straight line shows a final upward tilt. The reason for this is not clear, but may be related to some collapse of the sensitive clay that did not occur early in the loading. It is in this stage that the curves for the arms 5 and 6 merge with that from 4. Considering the test results from all of the tests, about half of them show the final upward tilt of the GA plot, and half do not.

Figure 5.13 compares GA diagrams obtained from the readings of the three strain arms in the upper sets (arms 1, 2 and 3) for tests R14 and R15, and the lower
FIGURE 5.13 GA diagrams from upper and lower set arms readings of SBPM tests in Miocene clay
sets (arms 7, 8 and 9) for tests R12 and 13. These results illustrate the point that
the responses of the upper and lower sets of strain arms are similar in nature to
those noted for the middle set of arms. Although differences may exist in the early
portions of the curves, they merge to a common line in the end.

Figure 5.14 shows the predicted undrained shear strengths plotted versus
depth from all of the SBPM data of this investigation as interpreted using the GA
procedure. The gradients used to obtain the GA interpretations were those
obtained in the final loading stages of each test. The undrained strengths obtained
using the data from the center arms are shown by points, and those using the data
from the other two sets of arms are shown as a range. Also shown in Figure 5.14
are the strengths obtained by Mayu using the GA interpretation from his five SBPM
tests at the Schnabel site (his tests were limited to 6 meters in depth). The results
lead to the following comments:

1. The undrained shear strength increases with depth down to 6 m., then
decreases with depth. (This trend is consistent with that observed from
other in situ tests at the Schnabel site as will be shown in the next
section)

2. Undrained shear strengths obtained from all sets of strain arms are
similar.

3. The undrained shear strengths from this investigation and those of
Mayu are in reasonable agreement, although those of Mayu are
relatively constant with depth, while those from this study increase with
depth (from 0 to 6 meters).

The consistency of the data from all of the strain arms is considered
encouraging and reflects the fact that the GA curves for all strain arms merged in
the later phases of the tests.
FIGURE 5.14 Undrained shear strength profile from SBPM tests by using GA interpretation
5.6.1 Comparison of SBPM and PBPM Undrained Shear Strengths

Figure 5.15 shows typical GA diagrams for the PBPM tests. Note that all
the initial portions of the curves for the PBPM tests show the expected response of
a membrane not initially in contact with the soil. Due to the restriction on total
expansion of the membrane to no more than 8 mm, the PBPM tests could not
generate soil strain levels as high as the SBPM tests did. Partly due to the lack of
ability to strain the soil, the merging of the GA plots for the different arms was not
as complete as was achieved in the SBPM tests. This also had the effect that the
straight line portions of the GA curves were flatter in the PBPM tests than those
in the SBPM tests. This inherently led to higher undrained shear strength
interpretations for the PBPM tests than those for the SBPM tests.

Figure 5.16 compares the undrained shear strengths obtained from SBPM
and PBPM tests of this study and those found by Mayu. The PBPM test data are
consistently higher than those from the SBPM tests by a factor of two or more.
Could the PBPM tests give the same values of undrained shear strength as the
SBPM tests if the pressuremeter had a greater expansion capability? This cannot
be answered from the data of this investigation, although it is notable that the
slopes of the GA diagrams from most of the PBPM tests had not fully stabilized in
the final stages of the test, and were tending toward those of the SBPM tests.

5.6.2 Comparison With Cone Penetration and Other Testing Results

Table 5.1 summarizes the undrained shear strengths obtained from the
SBPM testing results of this study, the CPT and DMT results from Mullen (1991),
and laboratory test results obtained by Mayu (1987). The information is sorted in
FIGURE 5.15 Typical GA diagrams for the PBPM tests in Miocene clays
FIGURE 5.16 Comparisons of the undrained shear strength of the Miocene clay
## TABLE 5.1 $S_u$ values of the Miocene clays from different testing methods

<table>
<thead>
<tr>
<th>Investigators</th>
<th>Sample Type</th>
<th>Testing Method</th>
<th>$S_u$ (KN/M$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Casagrande (1965)</td>
<td>Shelby Tube</td>
<td>UU Triaxial (Remolded)</td>
<td>Upper Miocene: 220-245</td>
</tr>
<tr>
<td></td>
<td></td>
<td>UU Triaxial</td>
<td>Lower Miocene: 319-441</td>
</tr>
<tr>
<td></td>
<td></td>
<td>UU Triaxial</td>
<td>Upper Miocene: 40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>UU Triaxial</td>
<td>Lower Miocene: 10-30</td>
</tr>
<tr>
<td>Martin and De Stephen  (1983)</td>
<td>Shelby Tube</td>
<td>UU Triaxial</td>
<td>Lower Miocene: 240-442</td>
</tr>
<tr>
<td></td>
<td></td>
<td>UU Triaxial</td>
<td>Lower Miocene: 56-77</td>
</tr>
<tr>
<td></td>
<td></td>
<td>UU Triaxial</td>
<td>Lower Miocene: 79-122</td>
</tr>
<tr>
<td></td>
<td>Block</td>
<td>UU Triaxial</td>
<td>Upper Miocene: 274-500</td>
</tr>
<tr>
<td></td>
<td>SBPM (GA Method)</td>
<td>Upper Miocene</td>
<td>265</td>
</tr>
<tr>
<td></td>
<td>SBPM (BLP Method)</td>
<td>Lower Miocene</td>
<td>215-320</td>
</tr>
<tr>
<td></td>
<td>CPT</td>
<td>Upper Miocene</td>
<td>170-660</td>
</tr>
<tr>
<td></td>
<td>DMT</td>
<td>Lower Miocene</td>
<td>311-816</td>
</tr>
<tr>
<td></td>
<td>SBPM (GA Method)</td>
<td>Upper Miocene</td>
<td>56-535</td>
</tr>
<tr>
<td>This Study</td>
<td></td>
<td>Lower Miocene</td>
<td>224-786</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Upper Miocene</td>
<td>180-445</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lower Miocene</td>
<td>325-585</td>
</tr>
</tbody>
</table>
terms of upper and lower Miocene, using the local argument that this soil has two different units. For the Schnabel site, all evidence appears to support a break point between the two units at about 6 meters. This separation can be seen in the undrained shear strengths of the SBPM by the different trends in strength profile above and below 6 meters. It also should be noted that no undrained shear strengths are reported in Table 5.1 for the PBPM tests which were preformed by Mullen. These tests were conducted in holes opened by driving a split spoon sampler, and the results were not suitable for interpreting shear strength using conventional interpretation procedures.

The laboratory test results reported in Table 5.1 are scattered over a relatively large range. Some differences could be expected since the samples for the tests were obtained using different means. Disturbance is important to testing of the Miocene clay because of its sensitivity, and the degree of disturbance will be dependent on the type of sampling and even the methods used to remove the sample from the sampler device. Prior to Mayu, only pushed thin-wall Shelby tubes were used; Mayu used both Shelby tubes and carved block samples. Although there is some scatter, the block samples tended to provide the highest strengths, presumably because this technique induces the least disturbance in the sample. The SBPM $S_u$ values tended to agree best with those from the block samples. This can be seen in Figure 5.16 where the $S_u$ values from the SBPM and those from the laboratory tests by Mayu on Shelby and block samples are compared. The laboratory tests on Shelby tube samples yielded strengths much lower than from the block samples or derived from the SBPM tests. The positive comparison between the undrained strengths from the SBPM tests and the laboratory tests on the block samples is felt to provide strong support for the results obtained in the SBPM tests. These results also help confirm that the PBPM procedures lead to $S_u$ values that are too high.
The CPT provides cone tip resistance values that can be interpreted in terms of undrained shear strength. Mullen (1991) used the following commonly accepted equation for this purpose:

\[ S_u = \frac{(q_c - \sigma_o)}{N_k} \]

where
- \( q_c \): cone tip resistance
- \( \sigma_o \): in situ total vertical pressure
- \( N_k \): cone factor

In sensitive clays there is a question about the value of \( N_k \). Robertson and Campanella (1982) suggest \( N_k = 15 \) for an "average" clay, but should be 10 or less for sensitive clays. The Miocene clays are sensitive, but not radically so. Therefore, to cover both bases, \( S_u \) values were calculated using \( N_k = 10 \) and \( N_k = 15 \); results are plotted versus depth in Figure 5.17. The figure also shows the \( S_u \) values of the SBPM tests of this study. Note that the SBPM tests of this investigation and CPT tests yield similar undrained shear strength profiles with depth. The magnitude of the \( S_u \) values from the CPT tests using for either of the \( N_k \) values did not match those from the SBPM. However, if an average value of \( N_k \) had been used between 10 and 15 to reduce the test results, the magnitudes of the resulting undrained shear strengths would be in reasonable agreement with the SBPM data. There is some rationale for such an approach since the Miocene clay has characteristics in between a nonsensitive clay and those for which the \( N_k \) of 10 is normally used.

Figure 5.17 also shows the undrained shear strengths profile based on dilatometer testing (DMT) results of Mullen (1991). Interpretation of the DMT follows the procedures outlined by Marchetti (1980). These are largely empirically based and the information used in their development apparently did not include a soil like the Miocene clay. Thus, it was not expected that the DMT would
FIGURE 5.17 Undrained shear strength of Miocene clays determined from pressuremeter testing, CPT and DMT
necessarily agree with those from the other tests, but these results proved to be in reasonable agreement with those from the SBPM and the CPT. The DMT values are slightly higher in the upper Miocene and slightly lower in the lower Miocene, but the trends are consistent with those from the CPT and the SBPM.

5.7 Shear Modulus Values

Usually two types of modulus values are obtained from a pressuremeter test: (1) An initial shear modulus, \( G_i \), derived from the initial linear portion of a pressuremeter curve; and (2) Unload-reload shear moduli, \( G_{ur} \), calculated from the unload-reload cycles. As noted in Chapter 2, Windle and Wroth (1977) suggest that \( G_{ur} \) is more reliable, because \( G_i \) is more sensitive to initial borehole disturbance caused by probe insertion. Figure 5.18 shows schematically how the different modulus values were determined from the pressuremeter tests in this study.

5.7.1 Initial Shear Modulus Value

The \( G_i \) values obtained from all SBPM and PBPM tests of this study are plotted versus depth in Figure 5.19. The \( G_i \) values presented are the average from a set of three strain arms, each oriented along one vertical plane. The moduli are identified by which set of arms were used. The results are scattered, but the overall trend mimics that of the undrained shear strength in that the \( G_i \) values increase to about 6 meters deep and decrease thereafter. There are indications of a particularly stiff layer at a depth between 4.5 and 6 meters. This layer also exhibited higher than normal undrained strengths.
FIGURE 5.18 Determination of unload-reload shear modulus from pressuremeter testing
FIGURE 5.19 Initial shear modulus values from pressuremeter testing in Miocene clays
An interesting detail is the observation that the modulus values obtained from arms 2-5-8 tend to be lower than those from the other arms. This effect presumably reflects the influence of the gap between the soil and the membrane next to the location of arms 2-5-8.

Comparing the initial shear modulus values from the PBPM to those of the SBPM two trends are evident: (1) Those from the PBPM are less scattered; (2) The values from the SBPM are higher, on average by about 50%. Of these trends, the finding of a lower modulus in the PBPM was expected due to the effects of soil disturbance caused by the pre-boring using the Shelby tube hole opening technique. The relative lack of scatter in the PBPM test results versus that seen for the data from the SBPM test is likely also related to disturbance. In opening the hole for the PBPM test, there is general disturbance of the soil surrounding the hole, whereas in the SBPM there were variations in this phenomenon depending on the effects of the tilt of the probe. Thus, in the SBPM test, the values from the strain arms on the side of the gap induced by the tilt of the probe were distinctly lower than those on the other side of the probe.

5.7.2 Unload-Reload Shear Modulus

For the early phases of the loading in the SBPM testing, the unload-reload loops were well defined, and it was easy to obtain $G_u$ values. However, this was not universally true for unload-reloading in the final loading stages, since the unload-reload loop deteriorated due to the sensitive nature of the clay. Wherever the loop was not well defined, no unload-reload shear modulus was determined. This was a particular problem for the PBPM tests, where the unload-reload loops were very poorly defined.
5.7.2.1 Relationships with Cavity Strain Amplitude

In a pressuremeter test with an unload-reload cycle, it is useful to define a term called "cavity strain amplitude." The cavity strain amplitude is the amount of cavity strain that occurs during the unload-reload process (see Figure 5.18). For pressuremeter testing in both sands and clays, studies show that the $G_{ur}$ value may be affected by the cavity strain amplitude as well as by the strain level where the unload-reload cycle was initiated (Clarke and Allan, 1990). Tests in this study have unload-reload cycles performed at various cavity strain amplitudes and cavity strain levels. This section presents data on the effects of cavity strain amplitude, and the next section deals with the cavity strain effects.

Figure 5.20 shows the relationship between $G_{ur}$ values defined from the mid-plane arms and cavity strain amplitudes from all the SBPM tests. The $G_{ur}$ values decrease as the cavity strain amplitude increases up to a value of about 0.2%. For cavity strain amplitudes greater than 0.2%, the $G_{ur}$ value levels off, apparently reaching a steady state value. The steady state value is about 50% of the maximum $G_{ur}$ value found at the lowest values of cavity strain amplitude. Clearly in commenting on shear modulus values, it is necessary to identify which cavity strain amplitude is involved.

Mayu (1987) performed unload-reload cycles in his SBPM tests at the Schnabel site, but he did not focus on the effects of cavity strain amplitude. All the unload-reload cavity strain amplitudes in his tests were about 0.2%. Figure 5.21 compares the $G_{ur}$ values from Mayu's investigation to the results of this study, with reference provided to cavity strain amplitude. For values in the vicinity of 0.2% cavity strain, there is good agreement between the two investigations.
FIGURE 5.20 Relationship between unload-reload shear modulus and cavity strain amplitude
FIGURE 5.21 Comparisons of unload-reload shear moduli from different investigations in Miocene clays
Figures 5.22 and 5.23 present the relationship between the $G_{ur}$ values and cavity strain amplitudes for the sets of strain arms which lie in different vertical planes for SBPM tests R15 and 22, and tests R13 and 18, respectively. Although at the smallest cavity strain amplitudes the $G_{ur}$ values from arms 2-5-8 are slightly lower than those from the arms 1-4-7 and 3-6-9, there is generally very good agreement of absolute values and trends with cavity strain amplitude. Thus, this evidence suggests that the gap on the side of arms 2-5-8 did not have a large effect on the $G_{ur}$ value, particularly at the larger cavity strain values. As indicated by previous investigators, the $G_{ur}$ value is insensitive to a moderate degree of disturbance.

Table 5.2 summarizes the $G_i$ and $G_{ur}$ values derived from the PBPM and SBPM tests in this investigation. For the SBPM tests, two sets of $G_{ur}$ values are given, one for a cavity strain level of 0.1% and the other for cavity strains greater than 0.2%. The shear modulus values are averages from all the available strain arm measurements in a given test. Figure 5.24 presents the values of $G_i$ and $G_{ur}$ from the SBPM tests plotted versus depth, where all $G_{ur}$ values are for cavity strains greater than 0.2%.

The basic trends for the $G_i$ and $G_{ur}$ values with depth are similar, but the $G_{ur}$ values are less scattered. Both sets of values show increases with depth to about 6 meters with decreases thereafter. In terms of absolute magnitude, both sets of values are about the same. Figure 5.25 compares the $G_{ur}$ values for cavity strains of 0.1% and greater than 0.2% in a plot versus depth; both the data for the SBPM and PBPM are shown (the PBPM data consists of only two points because of the problems mentioned in determining values from the poorly formed unload-reload loops). As previously indicated, the $G_{ur}$ values for 0.1% cavity strain amplitude are larger than those for cavity strains above 0.2%. The $G_{ur}$ values for 0.1% cavity strain also are less expressive of the trends of values with depth than that indicated
FIGURE 5.22 Unload-reload shear moduli from different strain arm readings of Tests R15 & R22
FIGURE 5.23 Unload-reload shear moduli from different strain arm readings of Tests R13 & R18
TABLE 5.2 Shear moduli determined by pressuremeter testing in the Miocene clays at the Schnabel Center site

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Depth (m.)</th>
<th>( G_i ) (MPa)</th>
<th>Cavity Strain Amplitude (%)</th>
<th>( G_{ur} ) (%)</th>
<th>Cavity Strain Amplitude (%)</th>
<th>( G_{ur} ) (MPa)</th>
<th>Test Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>R7</td>
<td>4.0</td>
<td>14</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>PBPM</td>
</tr>
<tr>
<td>R8</td>
<td>4.6</td>
<td>18</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>PBPM</td>
</tr>
<tr>
<td>R9</td>
<td>5.8</td>
<td>22</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>PBPM</td>
</tr>
<tr>
<td>R10</td>
<td>4.3</td>
<td>55</td>
<td>0.1</td>
<td>60</td>
<td>&gt;0.20</td>
<td>44</td>
<td>SBPM</td>
</tr>
<tr>
<td>R11</td>
<td>6.7</td>
<td>47</td>
<td>-</td>
<td>-</td>
<td>&gt;0.25</td>
<td>35</td>
<td>SBPM</td>
</tr>
<tr>
<td>R12</td>
<td>4.0</td>
<td>21</td>
<td>0.1</td>
<td>65</td>
<td>&gt;0.20</td>
<td>35</td>
<td>SBPM</td>
</tr>
<tr>
<td>R13</td>
<td>4.6</td>
<td>72</td>
<td>0.1</td>
<td>80</td>
<td>&gt;0.20</td>
<td>55</td>
<td>SBPM</td>
</tr>
<tr>
<td>R14</td>
<td>5.5</td>
<td>77</td>
<td>0.1</td>
<td>80</td>
<td>&gt;0.20</td>
<td>65</td>
<td>SBPM</td>
</tr>
<tr>
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<td>29</td>
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<td>70</td>
<td>&gt;0.30</td>
<td>32</td>
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<td>-</td>
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<td>0.1</td>
<td>65</td>
<td>&gt;0.25</td>
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<td>&gt;0.25</td>
<td>40</td>
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<td>SBPM</td>
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<td>32</td>
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<td>80</td>
<td>&gt;0.30</td>
<td>20</td>
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</tr>
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</table>

NOTE: The shear modulus values, \( G_i \) and \( G_{ur} \), are the average of all strain arm measurements.
SBPM test results

- □ Gi values from Arms 1, 4, and 7
- △ Gi values from Arms 2, 5, and 8
- ◇ Gi values from Arms 3, 6, and 9
- ■ Gur values (with cavity strain greater than 0.2% and cavity strain amplitude $>$ 0.2-0.3%)

**FIGURE 5.24** Compare SBPM test initial shear moduli with the unload-reload shear moduli having cavity strains greater than 0.2%
FIGURE 5.25 Unload-reload shear moduli from pressuremeter testing in different cavity strain amplitude ranges
previously for $S_u$, $G_i$, and $G_{ur}$ values for cavity strain amplitude above 0.2%. The reason for this is not clear.

Only two modulus values were available from the PBPM tests, with one for a cavity strain above 0.2% and one at 0.1%. The PBPM value at 0.2% was lower than those from the SBPM for comparable cavity strains, but the PBPM modulus at 0.1% cavity strain was in good agreement with the values from the SBPM at this cavity strain level. It is tempting to draw conclusions from these results, but there are not enough from the PBPM testing to allow statistically meaningful analyses to be made.

5.7.2.2 Relationship with Cavity Strain and Pressure Levels

The previous section showed the $G_{ur}$ value is dependent on the cavity strain amplitude during the unload-reload cycle. Some authors have suggested that there may be also a dependency between the $G_{ur}$ values and the absolute cavity strain or the applied pressure where the unload-reload cycle starts. However, under ideal, undrained conditions this would not be the case (Mayu, 1987). The data in this investigation can be used to investigate the effects of the absolute cavity strain parameter, although it is necessary to eliminate any influences of cavity strain amplitude. For purposes of this discussion, only $G_{ur}$ values defined over a consistent cavity strain amplitude are used. Figure 5.26 shows $G_{ur}$ values plotted versus absolute cavity strain amplitude, and the data suggest that the $G_{ur}$ values are almost constant regardless of absolute cavity strain. If any relationship exists, there is a slight decline of $G_{ur}$ value with increasing cavity strain. This effect may be induced because of the softening of the Miocene clay as the test shearing the soil beyond its peak strength at larger cavity strains. In a practical sense, absolute cavity strain apparently does not have an important effect on modulus.
FIGURE 5.26 Relationship between unload-reload shear modulus and cavity strain
5.7.2.3  Effect of Strain Rate

Strain rate is known from laboratory testing to have some influence on soil parameters, although Anderson et al. (1987) concluded that strain rates used in normal pressuremeter testing did not affect the shear modulus values. There are data available in this study to examine the strain rate effect for the Miocene clays. Guidelines used for obtaining the data were:

1. Use data from unload-reload cycles with comparable cavity strain amplitudes.
2. Use data from unload-reload cycles where the cycle started at a cavity strain higher than 2%.

Strain rates could be determined in the pressuremeter testing because the computer data acquisition system registered the applied pressure and the corresponding strain arm displacement every 10 seconds. Thus, the number of data points registered in an unload-reload loop can be used to define the rate of the loading. $G_{ur}$ values from the SBPM tests are plotted versus durations of the unload-reload cycles in Figure 5.27. No effect of the duration is seen for the range of loading rates used in this test program.

5.7.2.4  Effect of Multiple Unload-Reload Cycles

Some SBPM tests in this study had repeated unload-reload cycles at a given pressure level. Results of such tests allow study of the influence of multiple unload-reload cycles on the soil stiffness. Comparisons were based on similar levels of cavity strain amplitude and of absolute cavity strain. Figure 5.28 shows the $G_{ur}$
FIGURE 5.27 Relationship between unload-reload shear modulus and duration of unload-reload loop
FIGURE 5.28 Unload-reload shear moduli from subsequent unload-reload cycles in pressuremeter testing
values obtained for five subsequent unload-reload cycles in tests R19 and 21. From a practical point of view, the $G_{ur}$ values are constant as the loading is cycled. Previously, Mayu (1987) found small decreases in modulus in his SBPM tests in Miocene clays.

5.7.2.5 Comparisons of Results from Different Testing Methods

Although a number of different types of tests have been performed at the Schnabel site on the Miocene clay, few of them can be used to determine modulus values. In this case, access is obtained to in situ test unload-reload values from only the PBPM values of this investigation and that of Mullen where he conducted tests in a hole opened using a driven spoon sampler.

Figure 5.29 compiles the available unload-reload shear modulus values for cavity strain levels above 0.2%. The PBPM values are lower than those from the SBPM tests by about 50%. A similar level of difference between PBPM and SBPM initial tangent modulus values was observed earlier. The lower modulus values in the PBPM tests presumably are due to the effects of disturbance induced during opening of the hole. It is somewhat surprising that the modulus values from Mullen's tests in holes opened by driven samplers are as reasonable as they are given the crude nature of the hole opening process. Perhaps because of the unusual sensitivity of the Miocene clay, the effects of disturbance are essentially the same after some degree of this effect.

The level of disturbance induced in opening the holes for the PBPM tests was apparently far greater than that caused by the tilt of the pressuremeter in the SBPM tests. In the latter case, the unload-reload modulus values from each set of
NOTE 1: Gur is determined from the unload-reload cycle with cavity strain amplitude greater than 0.2%.

FIGURE 5.29 Compare unload-reload shear moduli from different pressuremeter testing techniques in Miocene clays
arms was similar, regardless of whether it was from the set of arms on the side of the probe with a gap or not.

Comparison of modulus values from in situ tests to laboratory tests is difficult because of the different strain and stress levels in the tests. Mayu (1987) provided a comparison based on the octahedral stresses that existed in his laboratory tests and those calculated for the pressuremeter tests. He concluded the modulus values from his SBPM tests compared well with those determined in his tests on undisturbed block samples. Since the SBPM modulus values from this investigation are in good comparison to the SBPM values determined by Mayu, it can also be assumed that those herein are in good agreement with those from the tests on block samples by Mayu.

5.8 Discussion

The SBPM tests of this investigation allowed a unique insight into the effects of details on the results of the tests. Having nine independent strain arms to monitor the membrane movement provided the data needed to know that the probe had entered the ground with a slight tilt (subsequently found to have been caused by a bent coupling). The evidence suggests that the tilt produced a gap between the probe and the ground on one side of the probe on the order of no more than 0.7 mm. This small gap was enough to make reliable interpretation of the lift off pressures, and hence the in situ lateral pressures, impossible. However, it did not interfere with interpretation of the undrained shear strength or the modulus values. Obviously, if a standard self-boring pressuremeter was being used with only three strain arms, this would not have been possible and the ensuing interpretation of parameters would have been subject to considerable guesswork. There is an issue related to the SBPM test R16 as discussed in Section 4.6.1. Due to the difficulty
met in the test R16 self-boring operation, the test might work as a full displacement pressuremeter test. Data showed that test R16 obtained lower undrained shear strength and lower shear modulus values. Due to the limited information, this comparison is not conclusive.

With an accounting for the tilt of the probe, the SBPM results, interpreted using the GA procedure, were found to provide undrained shear strengths that were comparable to the best of the available laboratory test results that had been performed on undisturbed block samples. Similarly, modulus values from the SBPM were inferred to have a similar degree of validity. These results suggest that the SBPM does have a place in testing of stiff clays, although it is understood that this test is expensive relative to others.

The SBPM undrained shear strengths were also found to compare reasonably well with values from the CPT and DMT. There is some question about the interpretation procedures for the CPT and DMT in a soil as unique as the Miocene clay, and thus the absolute magnitude of the shear strengths is to some degree questionable. However, the trends of undrained strength should be accurate, and these agreed well with those interpreted from the SBPM tests.

Combining the work by Mullen and this investigation, pressuremeter tests using three different methods of pressuremeter insertion can be compared. These include the SBPM test and two types of PBPM tests, with one set performed in holes opened using a driven sampler, and the other in holes opened by pushing a thin-walled Shelby tube. By all measures, it was expected that the latter PBPM procedure should induce less disturbance than the former. However, in both cases, the hole opening equipment must be removed to introduce the pressuremeter, and this leaves the hole unsupported for a period of time before the pressuremeter can be inflated against the sides of the hole. In the case of the SBPM tests using the
self-boring process, the probe drilled itself into the ground while keeping contact with the soil. Except for the effects of the tilt of the probe, the soil was supported fully during this approach.

The undrained shear strengths determined from the pressuremeter tests allow an insight into the effects of the hole opening process. Unfortunately, Mullen's tests could not be interpreted for shear strength because the pressuremeter was not able to reach levels of cavity strain needed to make a suitable GA interpretation. Considering the remaining types of tests, the $S_u$ values from the SBPM tests were lower by a factor of two or more than those from the PBPM tests conducted in the hole opened by pushing a Shelby tube. This reflects the anomaly in interpretation of pressuremeter tests whereby disturbance causes undrained strengths to be overestimated, the opposite effect that is normally seen in laboratory testing. In the GA interpretation method, this is physically seen in that the slope of the plot of the natural log of volumetric strain versus applied pressure is flatter than it should be. Since the inverse of the slope of this graph is used to determine the $S_u$ value, tests with disturbance yield undrained strengths that are too high. Some effects of this type were also seen when sorting out the influence of the initial gap caused by probe tilting during self-boring in the SBPM tests. Fortunately, with full expansion of the pressuremeter membrane, the slopes of the GA diagrams were consistent regardless of whether the strain arms were in the presence of the initial gap or not.

Considering the interpreted undrained shear strengths from the SBPM and PBPM tests, the degree of disturbance in the pressuremeter test can be seen to be important. The Miocene clay may serve to heighten this influence because of its sensitivity to disturbance. Looking to degree of effect that caused by the minor tilt during the installation of the probe in the SBPM tests was not severe enough to distort the findings, once this effect was understood in reducing the data. However,
the effects of opening the hole by a thin-walled Shelby tube were large enough to
distort the interpreted undrained shear strengths in an unconservative manner.

Comparisons of modulus values from the SBPM and PBPM tests showed
that in terms of either the initial tangent or unload-reload modulus values, there
was a difference in results. Either of the modulus values from the SBPM tests
were on the order of 50% higher than those from the PBPM tests. Effects of the
gap formed by probe tilting in the SBPM tests were apparent in lowered interpreted
initial modulus values on the side of the gap, but essentially nonexistent in the
unload-reload modulus values. Considering all of the data for modulus values, as
with undrained shear strengths, it is apparent disturbance is important to the results
obtained. However, modulus values from a disturbed pressuremeter test will be
conservative, as opposed to the trend toward unconservative undrained shear
strengths.

5.9 Summary

In aggregate, the results for the PBPM tests performed in the holes opened
by pushing a Shelby tube suggest that this method did not do a satisfactory job of
modeling the SBPM test in the Miocene clay. However, it is not appropriate to
extrapolate this finding to other stiff clays without further study. The Miocene clay
is sensitive and is more strongly affected by disturbance than most normal stiff
clays. It remains for future research to determine if the Shelby tube technique for
opening the hole for a pressuremeter test can or cannot provide useful results for
geotechnical practice.
CHAPTER VI SUMMARY AND CONCLUSIONS

Stiff clays pose problems for geotechnical engineers in that they typically contain numerous fissures and slickensides that cause wide scatter in conventional laboratory tests. The typical geotechnical engineering response, when confronted by such test results, is to use very conservative design parameters. While there may be some merit in choosing such parameters for problems where the soil is relatively unconfined (e.g., slopes and conventional retaining structures), this is overly conservative practice in cases where the soil remains confined during loading (i.e., pier foundations and pre-stressed tied-back walls).

This study involved an investigation of an in situ testing technique which has the potential to help reduce conservatism in design in stiff clays. The principal focus was on the use of the pressuremeter, and a field investigation conducted at a site in Richmond, Virginia. The soil at this site, known locally as Miocene clay, is stiff, sensitive, and contains few fissures or slickensides. The lack of defects in the clay makes it an ideal candidate to allow laboratory and field tests to be compared. The Miocene clay also has the advantage that several research investigations have been performed on it, providing a useful data base for comparison purposes.

The testing program was accomplished using a pressuremeter with unique capabilities. Important to this investigation was the ability to measure membrane movements at nine points around the membrane. This allows more information to be obtained on the membrane expansion process than in any previous study. The probe was also designed so that the signals from the electronic devices could be digitized and amplified before they were sent to the ground surface. This allowed the results to be directly monitored in real-time using a portable computer during
the test. Finally, the probe had the capability to be used in either a self-boring or pre-boring mode.

The test program consisted of 14 SBPM tests and 4 PBPM tests, the latter of which were conducted in holes opened by pushing in a thin-walled Shelby tube. The SBPM tests are known to have the ability, if performed correctly, to develop high quality design parameters. However, the SBPM process is expensive and time-consuming. The PBPM tests were relatively simple to perform, and they were conducted to determine if they could provide an inexpensive alternative to obtain SBPM quality parameters.

In addition to the field tests, an analytical study was undertaken to explore the issues associated with the effects of the presence of an anisotropic stress field on the interpretation of the pressuremeter test results. Recent tests with the SBPM have found evidence for anisotropic stresses in soil masses and such conditions have been suggested to exist in the Miocene clay. Thus, it was felt important to understand this issue to make appropriate interpretations of the data that would be obtained in the field testing.

Conclusions drawn from the results of this investigation are given in the following paragraphs.

6.1 Interpretation of In Situ Stresses in SBPM Testing in an Anisotropic In Situ Stress Field

This study used two-dimensional finite element (FEM) analyses to investigate the soil response in an SBPM test in an anisotropic stress field. In the FEM analyses, interface elements were used to simulate the effects of the membrane
which surrounds the probe and serves as the contact between the soil and the probe. Anisotropic in situ stresses were induced in the soil by applying stresses at boundaries to the soil mass surrounding the probe. Using the pressures determined in the interface elements, the lift-off pressures that would be observed in a pressuremeter test were defined. It was found in the presence of anisotropic stress fields, the lift-off pressure distribution showed greater degree of anisotropy than that of the actual in situ stresses. This was explained in terms of a stress concentration effect.

Two approaches for interpreting lift-off pressure distribution in the presence of anisotropic stress fields were reviewed in view of FEM results. Of these, it was concluded the strain rosette theory approach proposed by Dalton and Hawkins (1982) was preferred to that of the elliptical stress approach proposed by Mayu (1987). Using the strain rosette theory and considering the stress concentration effect, procedures for interpreting in situ horizontal stresses based on SBPM testing were developed and applied to two case histories. It was found the new procedure led to reductions in predictions of in situ stresses when anisotropic conditions were present. However, the in situ stresses measured by Mayu in the Miocene clay were still larger than could be explained from conventional theories.

6.2 Shape of Pressuremeter Membrane During Expansion

While it is assumed that the pressuremeter membrane expands cylindrically, there have been few studies performed to investigate this assumption. The present pressuremeter, with the ability to measure membrane movement at nine points, provided the first opportunity to examine this issue in the field. The results showed the membrane did not expand perfectly uniformly in any of the tests performed, including the SBPM tests. However, in terms of absolute movements, the
nonuniformities were relatively small, and thus the membrane appearance closely resembled that of a cylinder. The studies of the membrane shape did show that in the SBPM tests one set of the strain arms moved consistently earlier than the others. This led to an exaggerated nonuniform membrane shape in the first phases of the tests. The phenomenon of early movement was found to have been caused by the formation of a small gap (typically less than 0.7 mm) during probe insertion. This was created by a slightly bent coupling that led to the probe being tilted as it moved downward during insertion. Membrane shape measurements during the later expansion phases showed that the "gap" effect did not hold throughout the test, since with expansion, the arms that moved first were not always the ones that moved the most at the end of the test.

The shape of the membrane in cross-section in the early phases of the test was found to be reasonably fitted by an ellipse with increasing pressures, the shapes remained elliptical, but tended more toward circular. It was postulated that the elliptical shapes of the membranes could be caused in part by the existence of an anisotropic in situ horizontal stress field. However, while there was some evidence for this theory, it was not conclusive.

6.3 Undrained Shear Strength

After some deliberation, it was determined the best technique for obtaining the undrained strengths from the pressuremeter tests was that proposed by Gibson and Anderson. This was used throughout the investigation. The nine-arm probe provides data for up to nine GA curves which can be used for interpreting an $S_u$ value. In most cases, the results from the arms were used in various combinations so as to allow study of possible differences in soils strength from one side of the probe to the other, or to examine the influence of end effects on the upper and
lower sets of arms. After an accounting was made for the effects of the gap formed in the SBPM insertion process, little difference was found from any of the alternatives.

The undrained shear strengths from the SBPM tests were found to compare well with those obtained in laboratory tests on undisturbed block samples of the Miocene clay and the previous SBPM tests by Mayu (1987). Also, trends of the SBPM undrained strengths with depth were essentially the same as those from CPT and DMT tests. Magnitudes of the strengths from the SBPM tests were similar to those interpreted from the CPT and DMT, even though there is some question about the use of the standard methods for reducing the CPT and DMT results in a sensitive clay like the Miocene.

Undrained strengths from the PBPM tests with a hole opened using a pushed Shelby tube were from 2 to more times those from the SBPM tests. The high values obtained in the PBPM tests were due to effects of soil disturbance during hole opening. This effect is the opposite of that seen in laboratory tests where disturbance leads to lower strengths. It is an artifact of the assumptions used in the procedures for reducing strengths from the pressuremeter test.

6.4 Shear Modulus Values

Two types of shear modulus were evaluated in this study: (1) The initial shear modulus, \(G_i\), obtained from the initial linear portion of a pressuremeter curve; (2) The unload-reload shear modulus, \(G_{ur}\), obtained from unload-reload cycles during the test. The initial shear modulus is reported to be more sensitive to initial soil disturbance than the unload-reload shear modulus. This proved to be true for the tests of this investigation. For example, in the SBPM tests, the initial shear
modulus value was affected by the presence of the gap caused by the insertion of the probe, but the unload-reload modulus was not.

One of the details of the interpretation of the SBPM or PBPM test that is important to the $G_{ur}$ value is the amplitude of the cavity strain which occurs during the unload-reload cycles. The maximum $G_{ur}$ value occurs at small cavity strain amplitudes, and decreases as the cavity strain amplitude increases. In this investigation when the cavity strain amplitude was higher than 0.2%, the $G_{ur}$ value reached a steady state value, at about 50% of the maximum value. The $G_{ur}$ values that are usually reported in the literature are those at higher cavity strain amplitudes. In these results, the $G_{ur}$ values at cavity strain amplitudes greater than 0.2% better reflected trends with depth that were seen for other tests and other parameters.

$G_{ur}$ values were found independent of the following factors:

1. The pressure or cavity strain level where unload-reload cycles were originated.
2. The strain rate of the unload-reload cycle.
3. The number of unload-reload cycles.
4. The probe strain arm locations.

The $G_{ur}$ values obtained in this study were found to compare closely with those determined in previous laboratory testing on undisturbed block samples. Good agreement was also obtained with the modulus values from SBPM tests of Mayu (1987) if allowance is made for the level of cavity strains in the two investigations.
$G_i$ and $G_{ur}$ values from the PBPM tests of this investigation and those of Mullen (1990) were found to be up to 50% less than those from the SBPM tests. This apparently reflects the effects of disturbance caused when opening the hole for the PBPM tests.

6.5 PBPM vs. SBPM Tests

Disturbance in opening the hole had a significant influence on the results of the PBPM test results. This led to undrained shear strengths that were higher than those from the SBPM tests by factors of two and more, and unload-reload modulus values that were 50% of those of the SBPM tests. Although the PBPM test is far more efficient to perform than the SBPM test, the quality of the test results is seriously degraded by the testing process. This conclusion does not bode well for the PBPM test, but it should not be extrapolated to other stiff clays until further evidence is in hand since the Miocene clay is unusually sensitive.

The SBPM test proved itself capable of providing high quality parameters for the stiff clay of this investigation. However, it was also shown that the interpretation of the SBPM test can be a delicate task. Because of the effects a gap opened between one side of the probe and the soil due to a slight tilt induced in the pressuremeter during the self-boring process, no reasonable estimate of the in situ stresses could be obtained. Fortunately, having the measurements provided by nine strain arms allowed this test artifact to be understood and to allow for it in interpretations of undrained strength and modulus. Without access to the information needed about the orientation of the probe, the value of the test program would have been considerably reduced.
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APPENDIX

Pressuremeter Curves of SBPM and PBPM Tests in Miocene Clays
FIGURE A.1 Pressuremeter curves of Mayu (1987) SBPM test R1
FIGURE A.2 Pressuremeter curves of Mayu (1987) SBPM test R2
FIGURE A.3  Pressuremeter curves of Mayu (1987) SBPM test R3
FIGURE A.4  Pressuremeter curves of Mayu (1987) SBPM test R4
(SBPM TEST; MAYU, 1987)

TEST R5  DEPTH = 6.0 M

--- ARM1  ---- ARM2  --- ARM3

FIGURE A.5  Pressuremeter curves of Mayu (1987) SBPM test R5
FIGURE A.6 Pressuremeter curves of PBPM test R7
FIGURE A.7 Pressuremeter curves of PBPM test R8
FIGURE A.8 Pressuremeter curves of PBPM test R9
FIGURE A.9  Pressuremeter curves of SBPM test R10
FIGURE A.10 Pressuremeter curves of SBPM test R11
FIGURE A.11 Pressuremeter curves of SBPM test R12
FIGURE A.12  Pressuremeter curves of SBPM test R13
FIGURE A.13  Pressuremeter curves of SBPM test R14
FIGURE A.14 Pressuremeter curves of SBPM test R15
FIGURE A.15 Pressuremeter curves of SBPM test R16
FIGURE A.16 Pressuremeter curves of SBPM test R17
FIGURE A.17 Pressuremeter curves of SBPM test R18
FIGURE A.18 Pressuremeter curves of SBPM test R19
FIGURE A.19 Pressuremeter curves of SBPM test R20
FIGURE A.20  Pressuremeter curves of SBPM test R21
FIGURE A.21 Pressuremeter curves of SBPM test R22
FIGURE A.22 Pressuremeter curves of SBPM test R23
FIGURE A.23 Pressuremeter curves of PBPM test R24
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