

THE FEASIBILITY OF USING THE MENARD-TYPE
PRESSUREMETER IN OKLAHOMA PERMIAN
RED SHALE-CLAYS FOR THE DESIGN
OF Laterally Loaded
DRILLED SHAFTS

By

MICHAEL L. HUGHES

Bachelor of Science
Oklahoma State University
Stillwater, Oklahoma
1969

Master of Science
Oklahoma State University
Stillwater, Oklahoma
1970

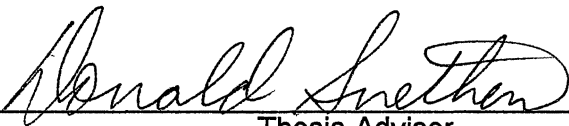
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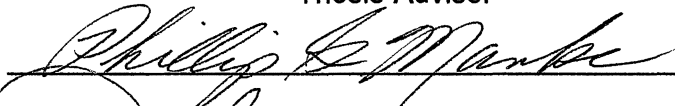
To my wife, Linda,
for her encouragement
and patience during
this study.

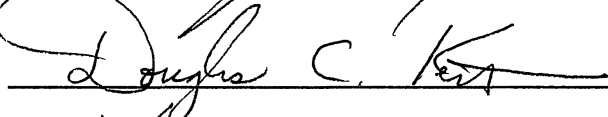
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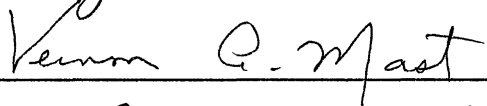
Thesis Approved:




Thesis Adviser









Dean of the Graduate College

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CHAPTER I

INTRODUCTION

General

Tapered octagonal steel poles are used to support 138 kilo-volt electric transmission lines in urban areas where right-of-way corridors are restricted. These steel poles are supported by drilled shafts ranging from 3 to 5 feet in diameter. The drilled shafts are cast-in-place without casing when the soil profile consists of hard dry clays and clay shales as commonly found in western Oklahoma. Large moments are produced by the weight of the conductors and live loads produced by wind and ice. These moments, which are normally larger than the vertical and horizontal forces, range from 500 to 1000 kip-feet. Foundations to support these loads are difficult to design because of the complex soil-structure interaction. It is also difficult to obtain adequate soil samples for laboratory testing. Unconfined compression tests have been used when adequate core lengths can be obtained. However, due to the layering of the soils, recovery of samples having sufficient length to test is not always possible. The unconfined compression test also produces a failure transverse to the direction of loading in laterally-loaded drilled shafts. Direct shear tests have also been used to determine soil strength properties. Sample preparation is difficult, even though the direction of failure is the same as the loading experienced by the drilled shaft. No testing device was available to conduct a laterally-loaded in-situ test until the pressuremeter was developed by Louis Menard. The pressuremeter is lowered into a borehole and soil parameters are determined using empirical relationships from the test results. The

pressuremeter causes the soil to fail transverse to the borehole alignment, in the same direction experienced by a laterally-loaded drilled shaft.

The Electric Power Research Institute (EPRI) has developed a computer program to design transmission line tower foundations. This semi-empirical program, called Moment Foundation Analysis and Design (MFAD) developed initially by Davidson (1982), uses pressuremeter and conventional laboratory test results as input information. This program was developed after conducting 14 field tests, one of which was located in Oklahoma City, Oklahoma. Commonwealth Electric, located in Eastern Massachusetts, has reported the savings of approximately \$100,000 on a million dollar project, which used the MFAD program for the design of 151 transmission towers (EPRI 1989).

Briaud, Smith and Tucker (1985) also prepared a method for designing laterally loaded drilled piers using pressuremeter test data from 17 test sites. Their work included the modification of the BMCOL76 program developed at the University of Texas during the 1960s which used load-deflection curves as input criteria. Pressuremeter results are used to develop load-deflection curves for the BMCOL76 program.

Purpose and Scope of Study

This research program had two primary objectives. The first objective was to determine whether the Menard Pressuremeter (MPM) yields realistic values when used in hard Permian clays and clay shale. The second objective was to determine if the MFAD and the BMCOL76 programs would give reasonable foundation design results.

CHAPTER II

USE OF THE PRESSUREMETER

Historical Uses

The use of the pressuremeter has been slow to gain acceptance by geotechnical engineers in the United States. Although the equipment has been used with apparent success in Europe, Canada, Australia, and the United States coastal areas where softer soil deposits can be found, conservative practices can be attributed to the lack of acceptance in the central plains of the United States. Much time has been devoted to correlating pressuremeter test results with more conventional laboratory methods of testing. Baguelin (1978) suggested this approach could lead to incorrect design values and prevent development of the full potential of the in-situ testing device. Nevertheless, few practicing geotechnical engineers have been willing to accept the use of the equipment and new design methods without some correlation with commonly accepted testing and design methods. Martin and Drahos (1986) investigated Calvert clay, a preconsolidated clay underlying the city of Richmond, Virginia. They successfully correlated pressuremeter and laboratory data. Based on measured settlement and Menard's (1975) equations, empirical values were developed that accurately predicted settlement using pressuremeter data.

Davidson and Bodine (1986) also reported the results of comparing pressuremeter data supplemented by a drilling, sampling, and laboratory testing program for a pile foundation to support a coal-fired power plant. They summarized their findings by indicating the pressuremeter is an effective soil characterization tool in soils which are difficult to sample and maintain in their in-situ structure if the

proper borehole preparation technique is used. In the stiff, slickensided clays at the power plant site, they reported pressuremeter derived strengths and moduli as being higher than those indicated from a conventional geotechnical investigation. However, they concluded the pressuremeter approach does not replace the need for laboratory testing; rather it complements and enables the laboratory testing to be optimized to a specific set of conditions identified by field measurements.

Fahey and Jewell (1984) also conducted comparisons between modulus and shear strength values derived from pressuremeter tests with results of other in-situ tests. They performed pressuremeter tests in a range of sands, silts, clays, and claystones in the vicinity of Perth, Australia. Pressuremeter tests were conducted using the English version of the Self-Boring Pressuremeter (SBPM) in soils and a high-pressure Menard pressuremeter in weak rocks. Their findings concluded that the standard penetration test, the dynamic cone and electric cone test did not provide an accurate method of settlement prediction if the shear modulus determined from the pressuremeter test is accepted as a "fundamental" soil parameter. They also suggested the shear strength derived from the pressuremeter test may need a reduction factor for use in settlement prediction, but additional research is necessary to support these findings.

Pressuremeter Tests in Hard Soils and Weak Rocks

Weak rocks and very hard, dry clays and clay shales often present inherent difficulties in sampling and testing. The use of the pressuremeter in weak rocks has been primarily limited to the MPM. In certain categories of weak rocks, such as some chalks, marls or mudstones, SBPM testing has been undertaken successfully; but presently the experience is relatively limited (Mair and Wood, 1987). The primary requirement for an MPM test is the formation of a good quality borehole with minimum disturbance. This requires extremely careful control. Results from pressuremeter tests can be assessed in conjunction with laboratory tests on carefully recovered rock cores, although it may not be possible to obtain cores

suitable for triaxial or direct shear testing. Mair and Wood (1987) also reported the undrained shear strength in marl, as determined using the pressuremeter, was normally much higher than that determined by either triaxial or unconfined compression tests. Undrained shear strength values determined by triaxial tests, ranging from 125 kN/m², were correlated with values determined from pressuremeter test values of 300 to 500 kN/m², respectively.

Jewell and Fahey (1984) also performed pressuremeter tests using a new high-pressure pressuremeter in siltstones and claystones. The high pressure pressuremeter developed at the University of Western Australia in conjunction with Golder Associates Pty. Ltd. has a nominal 20-mpa (2,900 psi) pressure capacity.

CHAPTER III

SITE LOCATIONS AND GEOLOGY

Geological History

The soils of interest investigated in the study were formed during the Permian period. The climate was warm and dry, and thick layers of gypsum and salt were deposited from evaporating sea water. Shallow seas covered the study sites intermittently from the Cambrian time to the middle of the Permian period. As these ancient seas evaporated, the Permian shales were over-consolidated through desiccation. These seas covered much of western Oklahoma and caused the soils to be formed in layers of interbedded sandstones, siltstones, and shales by alternating river delta and tidal-flats. The land generally sloped from the east to the west with many of the Permian soils originating in the Ozark uplift found along the eastern edge of the state and into what is now Missouri and Arkansas. As these elevated areas eroded, the material was carried by water to the vast marine lake (Dover, 1968). The red color of these Permian sandstones and shales comes from red iron oxide compounds in the form of oxidized minerals, such as hematite, deposited with the sands and muds. Soils found in the shallow depths of the study areas resulted from the weathering and disintegration of outcropping rock units. They are typically identified as the Renfrow Series, which are normally deep, well drained and very low permeability soils formed from weathered, clayey Permian shale. Reference to the process of how these soils were formed is a very important feature when considering the strength of soils. The layered deposition has caused a condition of anisotropy in the materials. This means the strength characteristics will be different when the soils are loaded perpendicular to the layers as compared to

when loaded parallel to the layers. This condition illustrates why an in-situ method of testing, where the load is applied parallel to the layer, is desirable when designing a laterally loaded pier.

Site Locations and Soils

Four of the five study sites are located in Oklahoma County with three of the sites located within five miles of one another. The first site is located on private property southeast of the intersection of Interstate Highways 35 and 240 and is designated as the "GAI Site" on Figure 1. This site was the location for one of the Electric Power Research Institute's full-scale tests conducted on drilled shafts throughout the United States in 1981. Two sites are located approximately five miles west of the GAI site. One site is located at the intersection of Interstate Highway 240 and May Avenue and is designated as the "May Site." Another site, the "I-44 Site," is located west of the May Site at the intersection of Interstate Highways 240 and 44. The remaining Oklahoma County site is located in the north-central part of the county at the intersection of Broadway Extension (U.S. Highway 77) and Interstate Highway 235. This site was referred to as the "Bdwy Site." The fifth site, the "Lawton Site," is located in southwestern Oklahoma within the City of Lawton as shown in Figure 1. Boring logs describing the soil profile at each site in shown in Figure 2. All five sites have the same characteristic red clay claystones and clay shales located at different depths beneath the surface of the ground and can be considered as "typical" occurrences of materials one might find when constructing an electric transmission line in western Oklahoma.

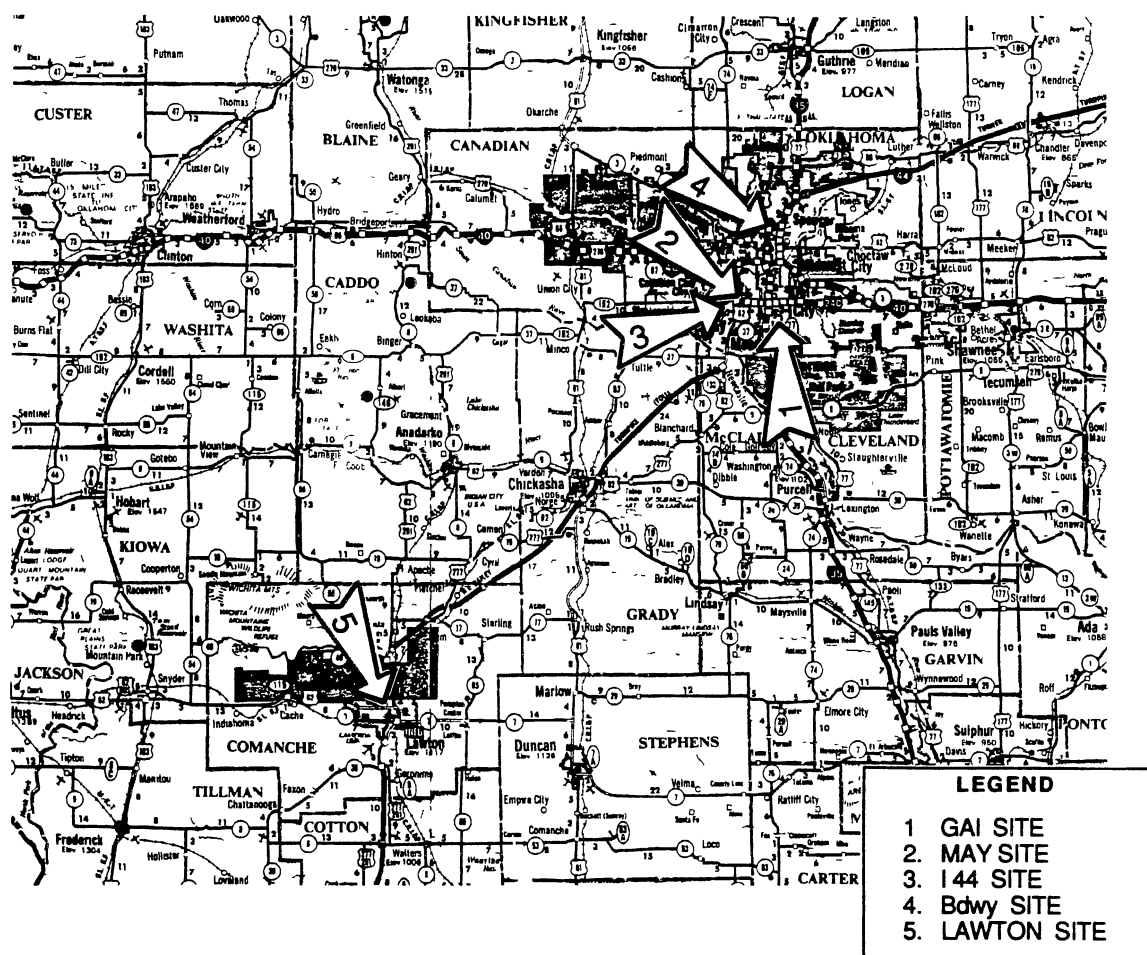
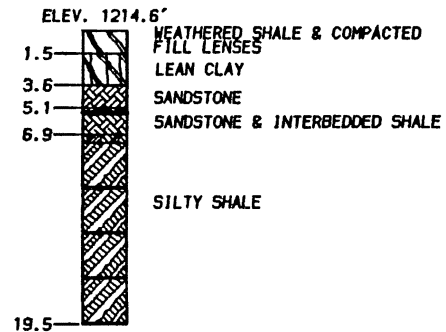
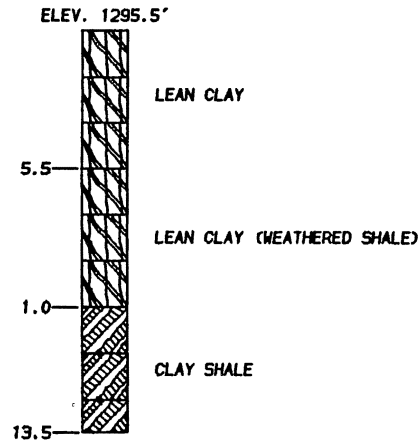


Figure 1. Test Site Locations

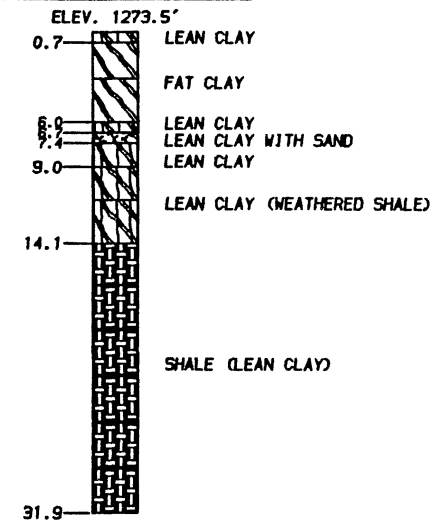
I-235 & BROADWAY EXTENSION



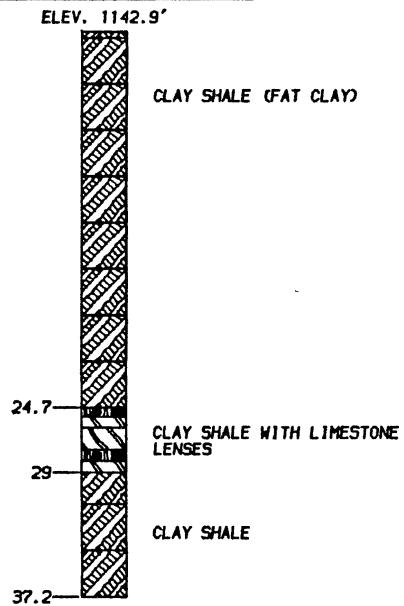
GAI SITE



I-240 & MAY AVENUE



LANTON FORT SILL BLVD



I-44 & US HIGHWAY 62

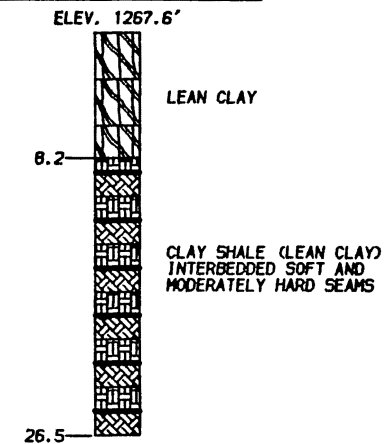


Figure 2. Boring Logs

CHAPTER IV

TEST EQUIPMENT, PRINCIPLES, AND PROCEDURES

Test Equipment

The MPM used in the research was manufactured by RocTest, Inc. of Plattsburgh, New York. The equipment is owned by the Oklahoma Department of Transportation (ODOT) who furnished the raw pressuremeter dated for the May, Bdwy, I-44, and Lawton Sites. The manufacturer produces several models and the model chosen for this work was the G-Am model. The G-Am model comes with a control unit, one probe, one bottle of compressed nitrogen gas, and the associated tubing used to connect the probe and the control unit. The unit comes standard with gauges capable of measuring a range of pressures from 0 to 25 bars (52.2 ksf). A high pressure conversion kit can also be purchased. This accessory will allow the equipment to be used with working pressures up to 100 bars (208.9 ksf). According to the manufacturer, this feature allows the equipment to be used in stiff soils and soft rocks. The equipment used in the research had this modification.

The control unit, shown in Figure 3, comes in a fiberglass case and includes pressure gauges and a manometer for reading the volume of water used to inflate the central measuring cell. The control unit is normally positioned adjacent to the borehole but yet in a location that allows a drilling truck to construct the borehole and the crew to conduct other forms of testing without requiring the control unit to be moved between tests.

An NX (70 mm diameter) probe was used in the research. The probe is 70 cm (27.6 in.) in length and weighs approximately 6.4 kg (14.1 lbs). The probe, as shown in Figure 4, is constructed with a cylindrical metal body with an inner rubber

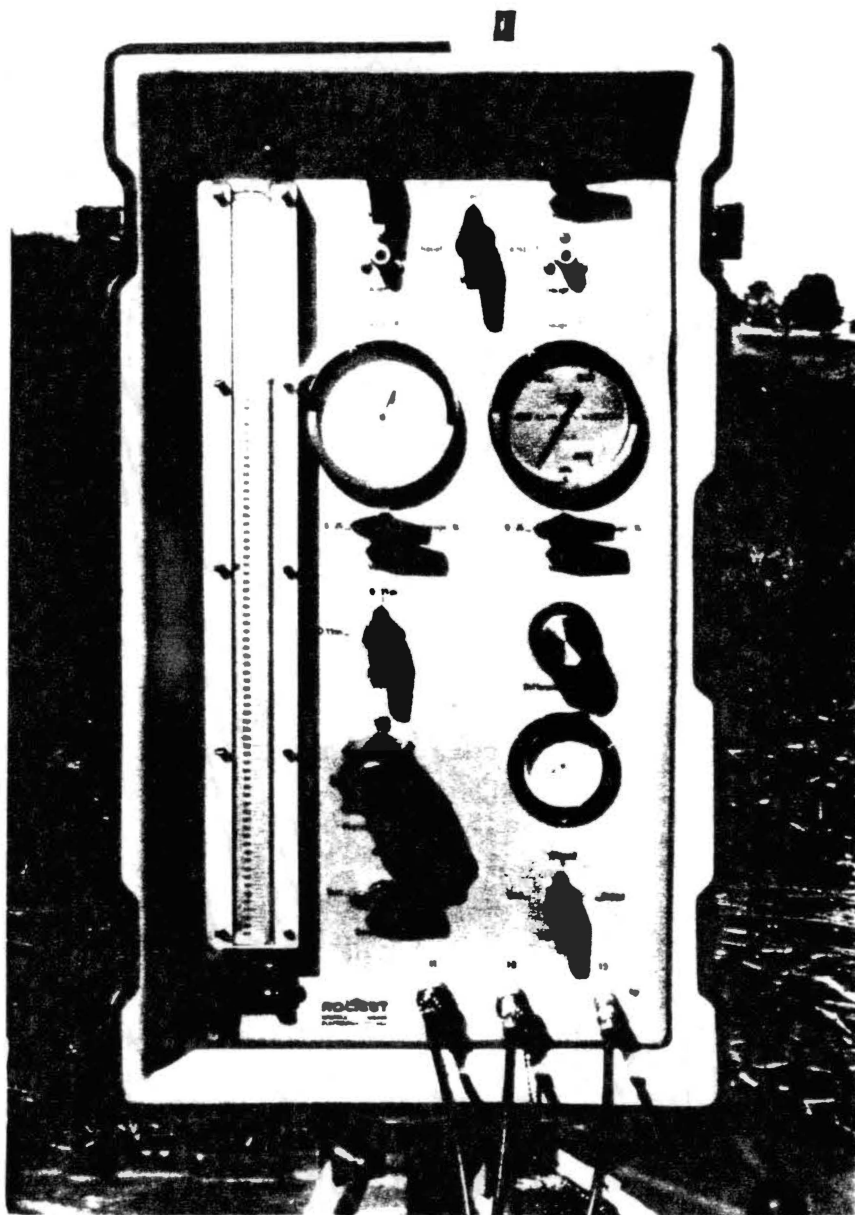


Figure 3. Control Unit

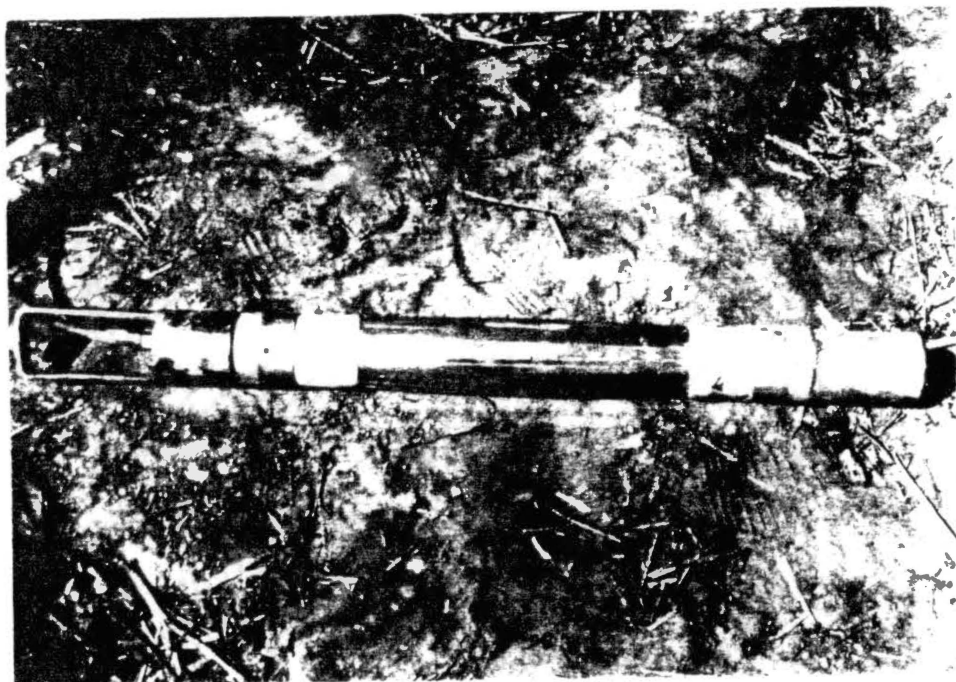


Figure 4. Pressuremeter Probe

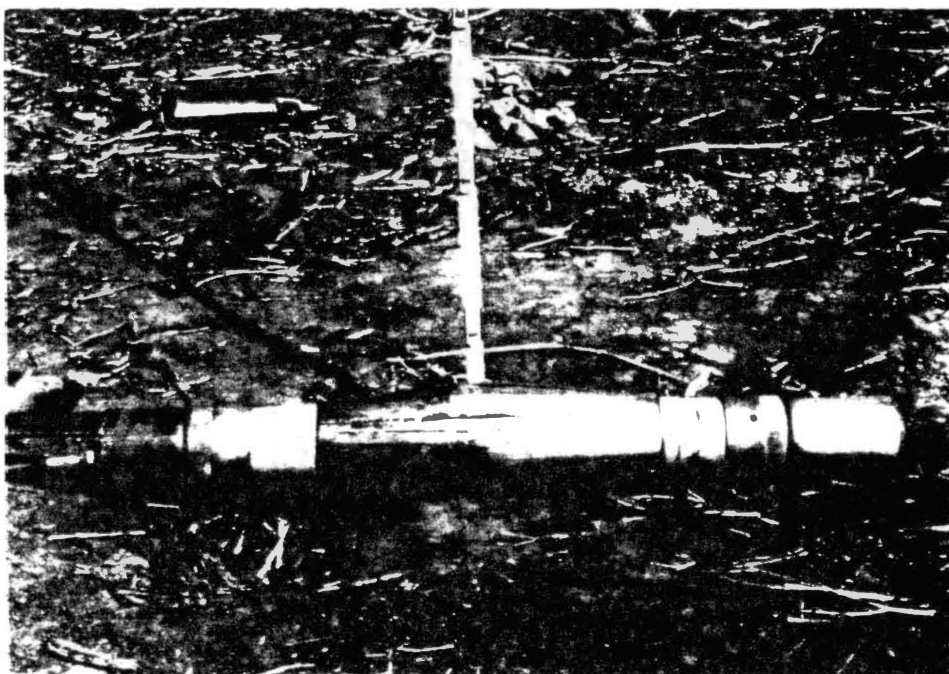


Figure 5. Inflated Pressuremeter Probe

membrane and outer protective sheath mounted to form an independent cell when properly inflated. Figure 5 shows the inflated probe. The cell is inflated with water. Figure 6 shows the nitrogen gas bottle located on the left side of the control unit and the container of water located on the right side of the unit. The probe is attached to the drill stem and lowered into the borehole until the test elevation is reached. The test location must be at least 38 cm (15 in.) above the bottom of the borehole to accommodate the portion of the probe below the central measuring cell.

Test Procedures

A drilling rig is used to advance a borehole at each test site. The method of advancement can be accomplished by different methods, but it is very important to reduce the disturbance of the borehole walls to a minimum. According to Finn (1984), the three major elements of disturbance of a borehole are:

1. Collapse or partial collapse (bulging) of the borehole wall.
2. Erosion of the borehole wall.
3. Softening of the borehole wall.

The Standard Test Method for Pressuremeter Testing in Soils is covered by ASTM Designation D 4719, hereafter referred to as the "Standard" (see Appendix E). This test was first standardized for use in the United States in 1987, but use of pressuremeter equipment preceded the Standard by more than 20 years. Although it is not the intent to restate the Standard in this work, it is necessary to emphasize several important points. Two conditions must be satisfied to obtain a satisfactory borehole. The diameter of the hole must meet the specified tolerances. For an NX size probe, the borehole cannot exceed 89 mm (3.5 in.). The other condition, according to the Standard, addresses the equipment and method used to prepare the hole to cause the least possible disturbance to the soil. It is imperative the pressuremeter test be performed immediately after the hole is formed. Figure 7 illustrates the shape of the pressure versus volume curve for conditions where the

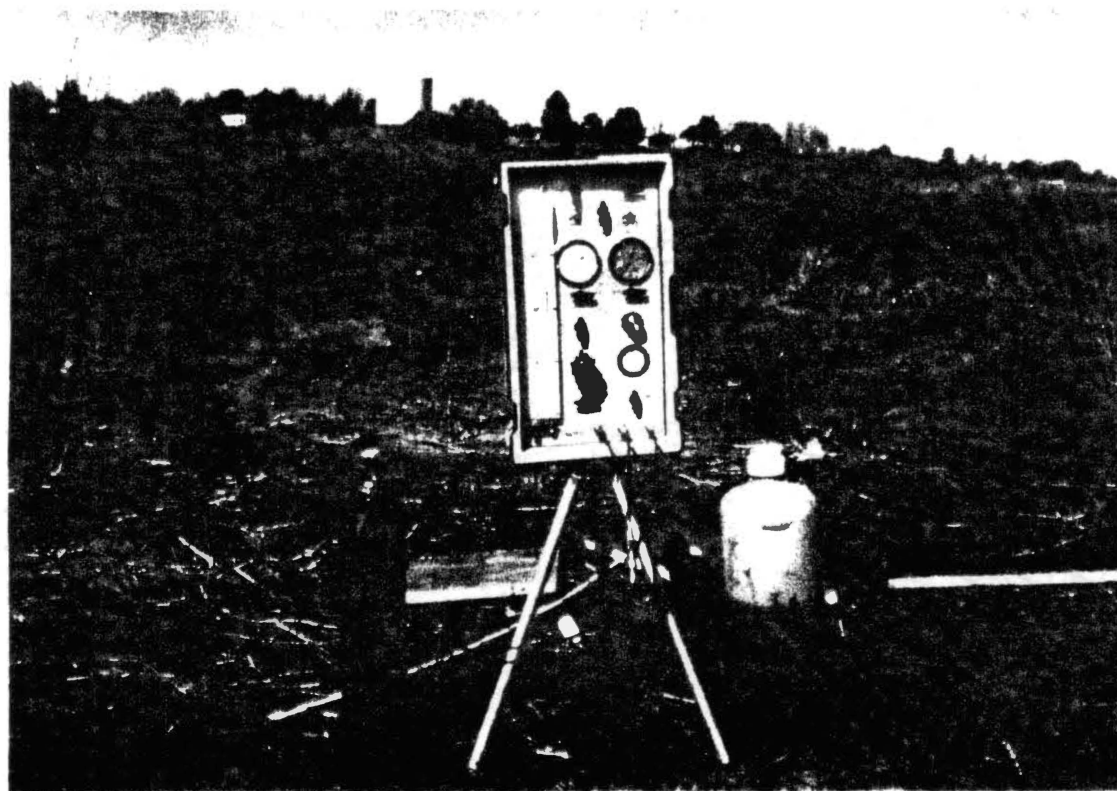


Figure 6. Control Unit at Site

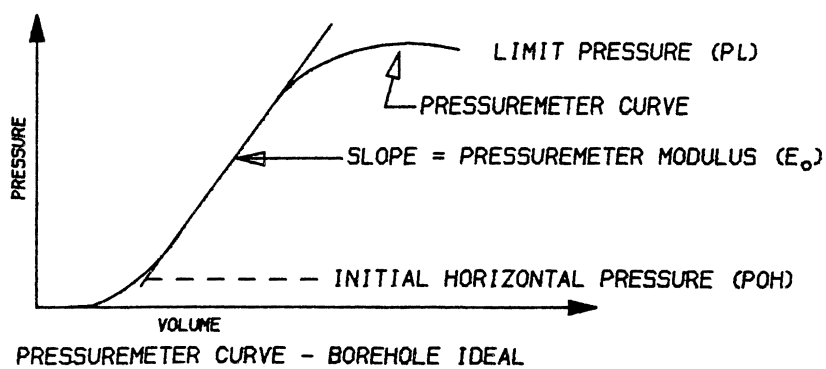
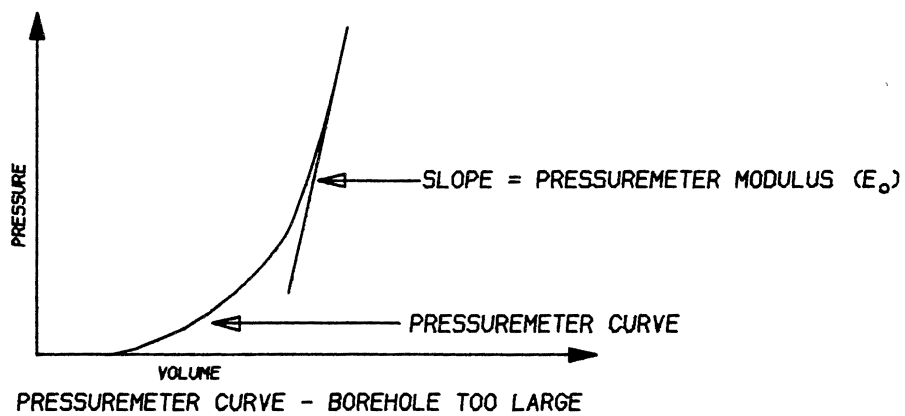
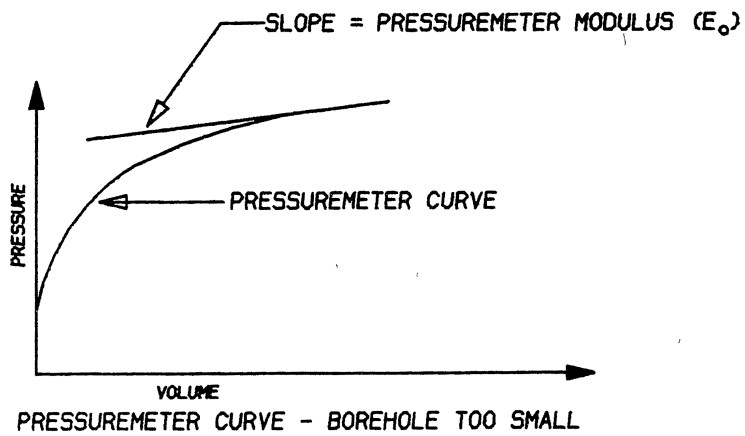


Figure 7. Pressuremeter Curve Shapes

borehole is too small, too large, and the ideal shape of the pressuremeter curve, respectively.

The pressuremeter must be calibrated at sufficient intervals. This is due to the change in the flexibility or fatigue of the membrane after repeated expansion and contraction which occurs during each test. The Standard indicates "the instruments should be calibrated before each use to compensate for pressure and volume losses." The RocTest instructions give four conditions for the need to calibrate the equipment and they simply state that calibration should be done regularly. ODOT calibrates the equipment each time it is assembled for use or when a membrane or tubing failure occurs. The Standard allows for two different types of test procedures: the "Equal Pressure Increment Method" and the "Equal Volume Increment Method." In the Equal Pressure Method, the pressure is increased in equal increments and the corresponding volume is read at 30- and 60-second intervals. Likewise, in the Equal Volume Method, the volume is increased in equal increments and the pressure is read at 30- and 60-second intervals. The Equal Pressure Method was used in this study.

Test Principles

Although the purpose of this investigation is not to elaborate on the principles of the pressuremeter test, a brief synopsis is provided. In principle, pressuremeter tests are equally applicable to soils and rocks according to Mair and Wood (1987). The MPM is inserted into a pre-drilled borehole where the user has some knowledge of the different strata. The membrane is expanded against the surrounding soil under pressure. Outward radial deformation of the soil occurs as the membrane expands. The object of the test is to obtain the relationship between the applied pressure and deformation of the soil. Deformation of the soil is measured by the volume of fluid injected into the center cell. The result is an in-situ stress-strain response for each tested soil layer. The pressuremeter modulus or the soil modulus of deformation can be determined from the test results. The soil limit

pressure is also obtained from the test results. The limit pressure can be considered as the ultimate loading pressure.

CHAPTER V

PRESENTATION AND DISCUSSION OF RESULTS

Pressuremeter Test Results

A total of 29 pressuremeter tests were used in the research. Table 1 summarizes the results of all pressuremeter tests. The initial pressure (P_0) and the net limit pressure (P_l^*) were not reported by Davidson (1982). One should note the large limit pressures and pressuremeter moduli at the deeper depths where the clay-shales were encountered. A larger limit pressure indicates an increase in shear strength; the larger pressuremeter modulus (E_0), which is similar to a modulus of deformation, implies the material is more rigid. This would be expected when shales similar to those found at the five sites are encountered.

Some tests appeared to be flawed; one questionable test was conducted at the 3-foot level of the May Site. Figure 8 is the pressuremeter curve data for this particular test. When compared to Figure 7, one can conclude the hole may have been too small. A significant time interval occurred between the time the hole was drilled and when the pressuremeter test was performed. This could have allowed the borehole to decrease its diameter as the internal stresses in the soil relaxed after the drilling operation. Water also had to be bailed from the hole before the pressuremeter test was performed which would have facilitated the relaxation. This explanation emphasizes the need for a properly constructed borehole which is discussed in the Standard. Corrected pressuremeter data and curves for tests conducted at the four sites can be found in Appendix A.

The limit pressure (P_l) is never obtained during field testing. Should the test continue to the limit pressure, failure of the membrane can be expected. To

TABLE 1
PRESSUREMETER TEST RESULTS

Site	Depth (Feet)	Initial Pressure Poh (ksf)	Limit Pressure PI (ksf)	Net Limit Pressure PI* (ksf)	MPM Modulus Eo (ksf)
GAI (Davidson)	2.5	N/A	9.4	N/A	86
	7.5	N/A	15.7	N/A	212
	11.4	N/A	70.3	N/A	1228
GAI (Hughes)	2.5	1.8	17.8	16.0	212
	7.5	2.8	22.7	19.9	410
	11.4	4.5	87.0	82.5	2587
May	3.0	1.5	22.7	21.2	272
	6.0	1.4	22.0	20.6	243
	9.0	2.0	17.6	15.6	269
	12.5	2.0	32.6	30.6	621
	16.0	2.5	61.6	59.1	1127
	19.0	4.0	51.6	47.6	833
	22.0	6.5	62.4	55.9	1502
	25.0	4.0	45.2	41.2	249
	28.0	10.0	145.1	135.1	7495
I44	2.5	0.8	18.8	18.0	112
	7.5	2.0	42.0	40.0	356
	10.0	3.0	37.9	34.9	87
	13.0	3.5	40.6	37.1	226
	16.5	7.5	177.0	169.5	4677
	24.8	10.0	118.0	108.0	5165
Bdwy	3.0	1.0	13.0	11.8	148
	7.0	2.0	186.0	184.0	217
	10.0	2.0	60.0	58.0	627
	12.0	5.0	48.0	43.0	1046
	14.0	2.2	85.0	82.8	838
	17.0	10.0	310.0	300.0	1001
Law	4.0	3.0	28.8	25.8	468
	8.0	3.5	48.5	45.0	1829
	12.0	3.8	34.8	31.0	530
	16.0	4.0	56.4	52.4	1225
	20.0	5.0	47.5	42.5	1352

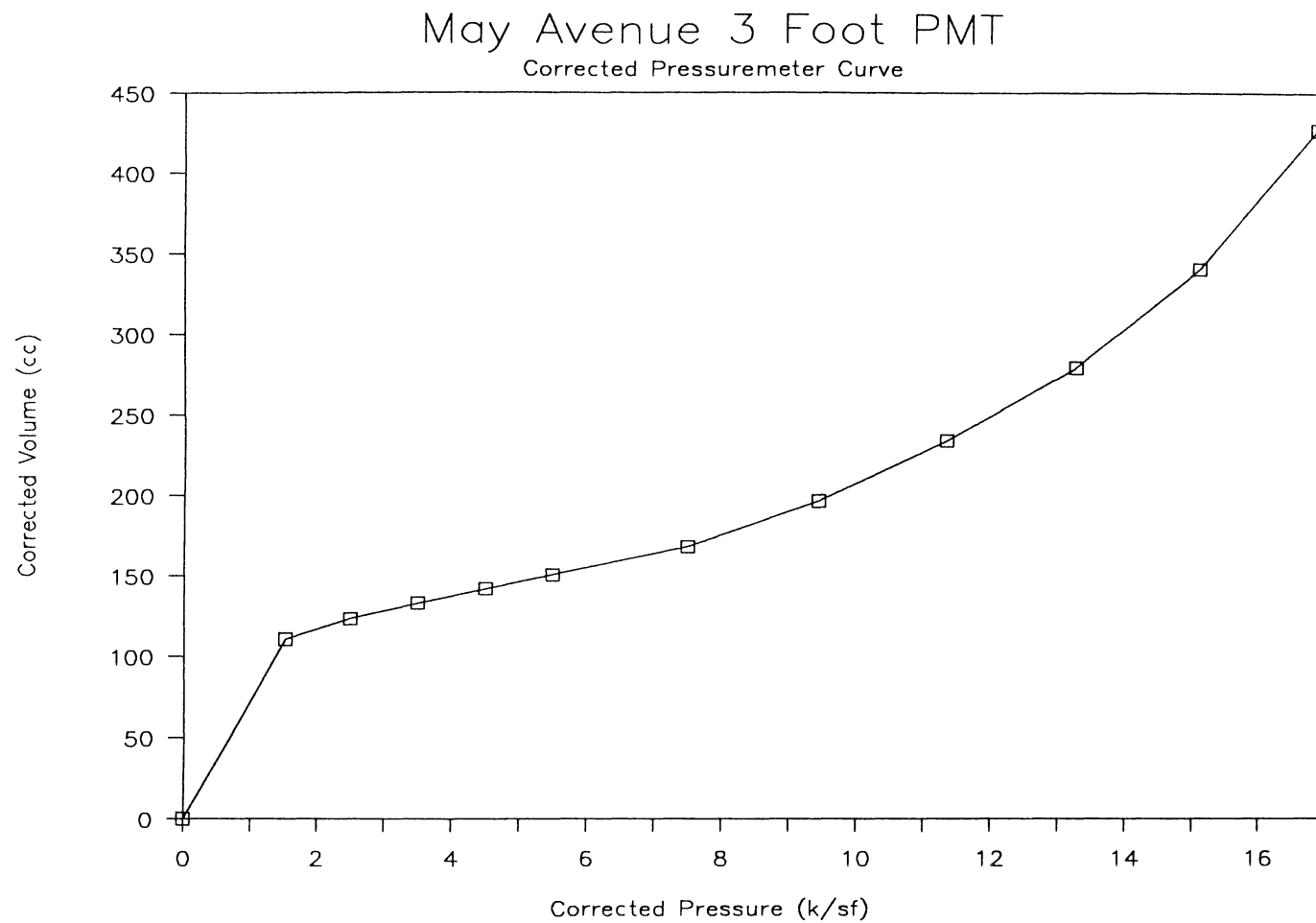


Figure 8. Pressuremeter Curve, May Site 3-Foot Depth

compensate for this dilemma, the Standard allows the user to use a plot of the inverse of the corrected volume readings versus the corrected pressure to determine the limit pressure. Figure 9 is taken from the Standard and illustrates the method of determining the limit pressure. Theoretically, the limit pressure is the pressure where infinite expansion of the borehole cavity occurs. For practical purposes, the limit pressure is defined as the pressure at which the inflated probe doubles the volume of the original soil cavity.

The limit pressure determined from Figure 9 is approximately 18 tsf and the last available pressure reading is approximately 12 tsf. This is an interpolation of only 6 tsf or approximately 50 percent of the last available reading. It is not known whether the ASTM committee anticipated any problems that could occur with this approach. Figure 10 is the curve developed for the I-44 Site at the 2.5 foot level. In this test, the limit pressure was estimated to be 18.8 ksf and the last pressure reading was at approximately 12.5 ksf. This condition reasonably relates to the example given in the Standard. Figure 11 is the plot used for the same site at the 7.5 foot level. The last available pressure reading occurred at approximately 21 ksf and the limit pressure occurs at nearly 42 ksf. This value is twice the value of the last available reading or a 200 percent increase. This phenomenon was also reported by Baguelin (1978) when he compared three methods of extrapolation and concluded the method described in the Standard was the only method of extrapolation that errs on the safe side by underestimating the value of the limit pressure. However, Baguelin warned the method used should not exceed 25 to 30 percent of the data on the test curve, and extrapolation beyond these limits should be avoided. These occurrences can be expected when testing hard clays or weak rocks and when the limit pressure of the soil is considerably larger than the capacity of the test equipment.

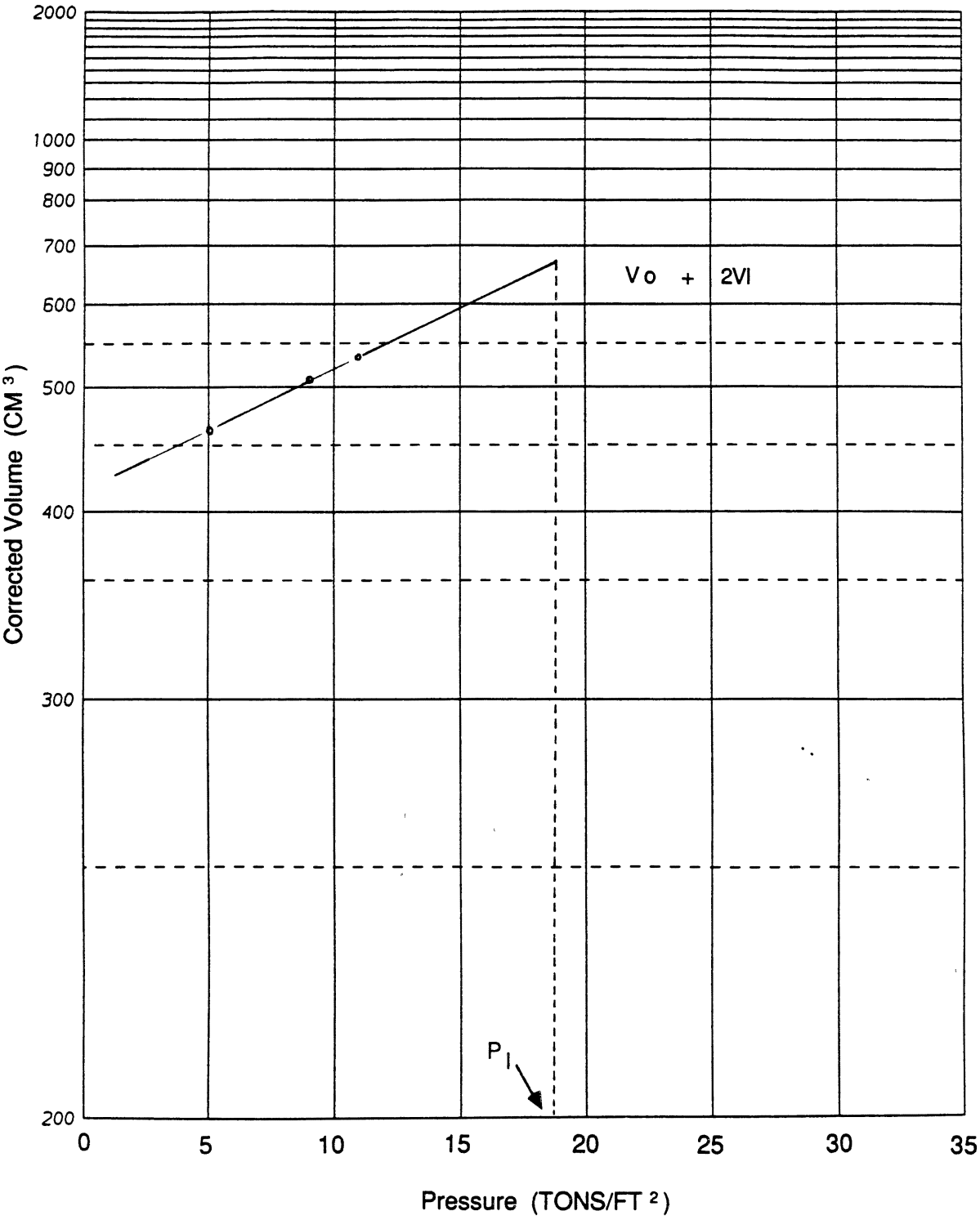


Figure 9. Inverse of Volume Versus Pressure, ASTM

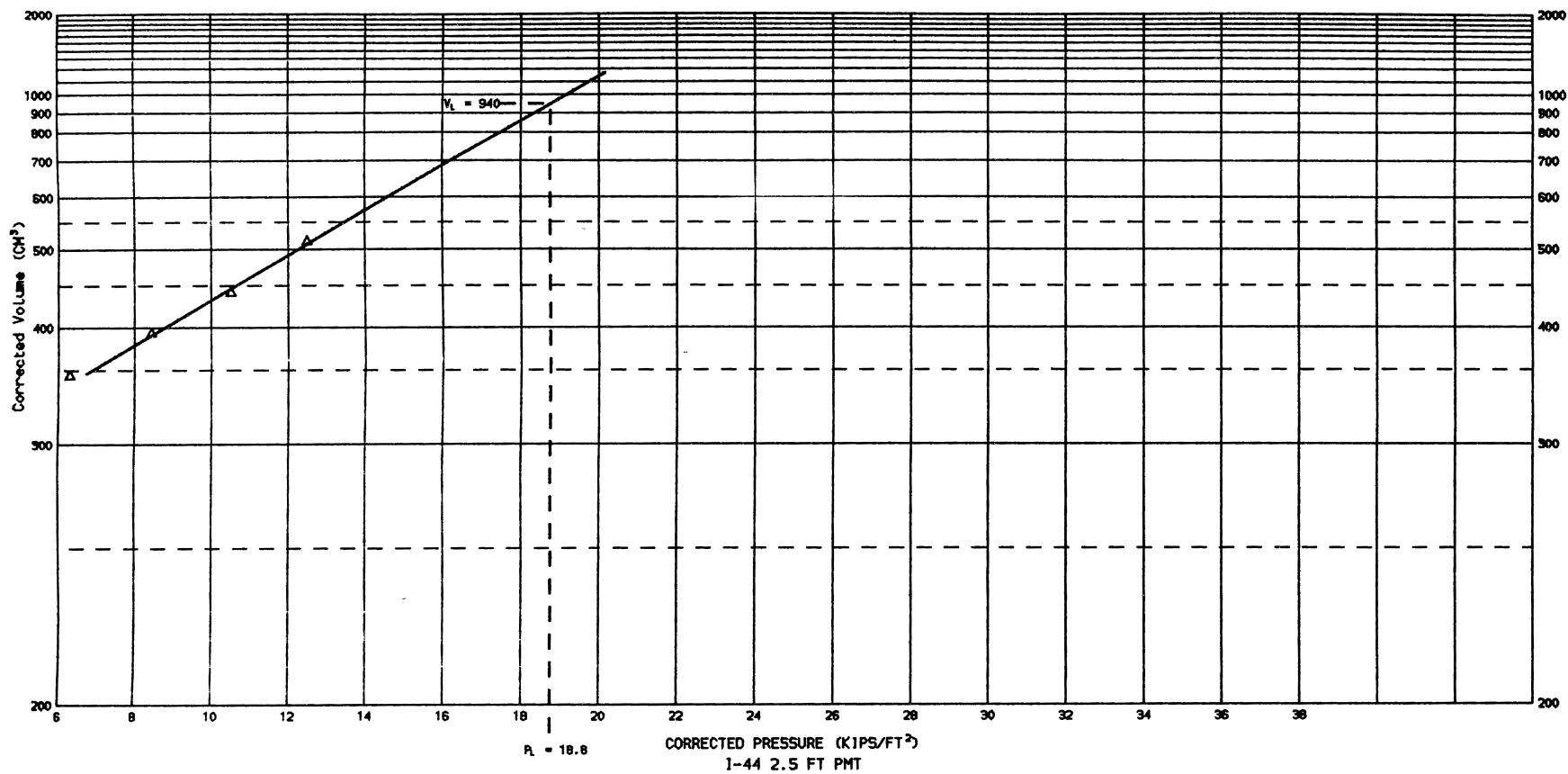


Figure 10. Inverse of Volume Versus Pressure, I-44 Site, 2.5-Foot Depth

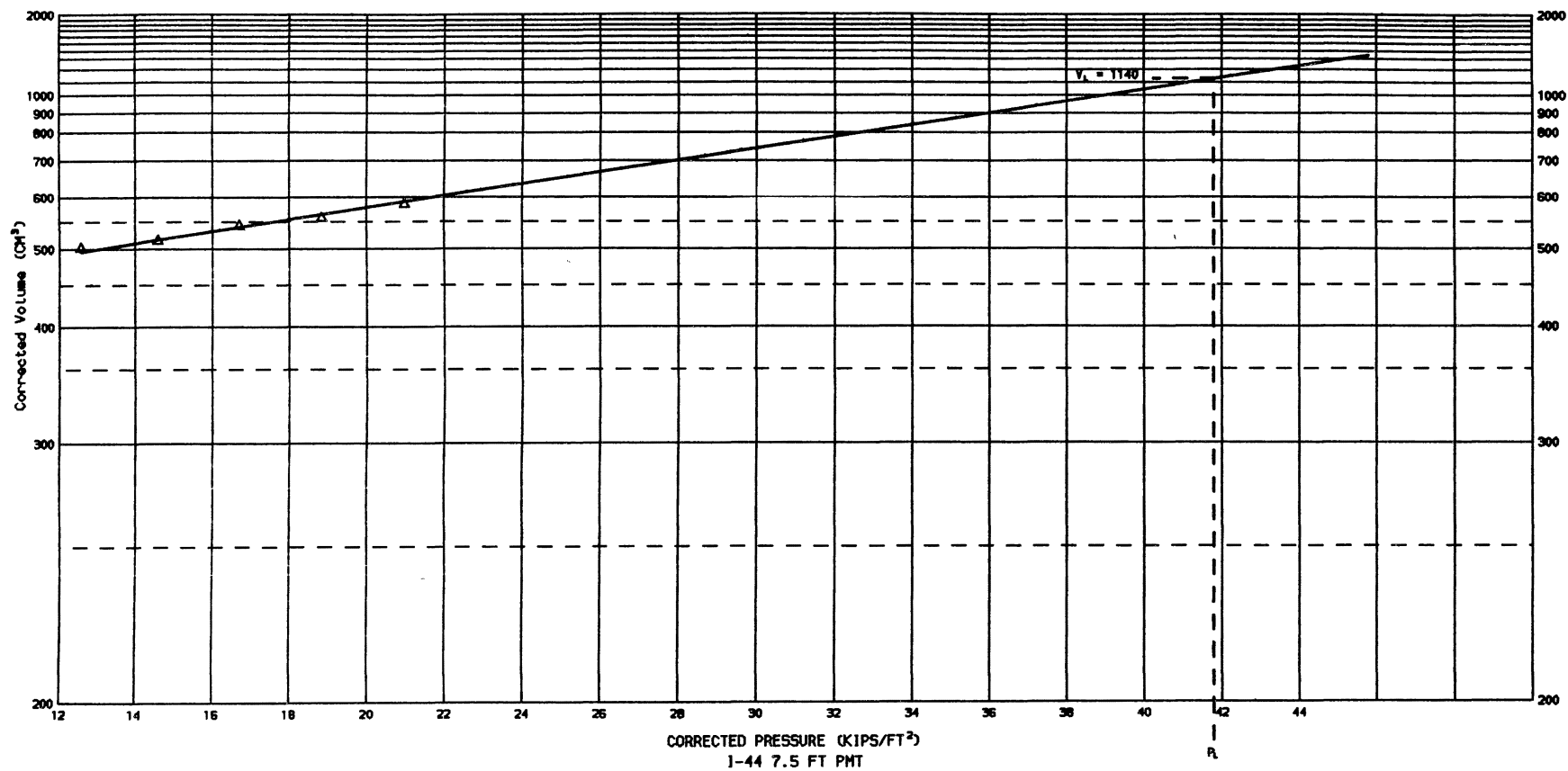


Figure 11. Inverse of Volume Versus Pressure, 1-44 Site, 7.5-Foot Depth

Estimation of the Undrained Shear Strength

To evaluate the MPM in estimating reasonable strength characteristics for the Permian clays and clay shales, the undrained shear strength was chosen as a measure of the instrument's performance. A number of researchers have proposed methods to estimate the undrained shear strength from pressuremeter data. Baguelin (1978) used a log-log regression curve of the undrained shear strength (S_u) versus the net limit pressure ($P\ell^*$) to develop Equation (1):

$$S_u = 0.21(P\ell^*)^{0.75} \quad (1)$$

The net limit pressure given in Equation (1) must be in terms of tons per square foot.

Briaud (1989) offered the simple relationship between the undrained shear strength and the limit pressure ($P\ell$) with the following equation:

$$S_u = \frac{P\ell}{7.5} \quad (2)$$

Orchant (1986) also developed correlations between the undrained shear strength and the limit pressure. His work, shown as Equation (3), must have the limit pressure given in kilo-pascals:

$$S_u = \frac{P\ell}{10} + 25 \quad (3)$$

Briaud (1989) developed a relationship between the pressuremeter modulus (E_o) and the undrained shear strength in Equation (4):

$$S_u = \frac{E_o}{100} \quad (4)$$

Briaud's data led him to conclude the relationship may be unreliable. However, this relationship was included in this study as one additional method of determining the undrained shear strength from the pressuremeter test.

Jewel and Fawley (1984) suggested the undrained shear strength was a function of the change in pressuremeter pressure (dP) and the natural log of the volumetric strain (dV/V) as shown in Equation (5):

$$S_u = \frac{dP}{\ln \frac{dV}{V}} \quad (5)$$

The undrained shear strength can be graphically determined from the plot of pressuremeter pressure versus the natural log of the volumetric strain after the initial elastic phase as shown in Figure 12. An approximation for the volumetric strain was used in this research. The volumetric strain is equal to approximately two times the radial strain when very small increases occur in the radial distance. This would be a reasonable assumption for hard clays where the deformation is small; but it may be less correct in the more plastic soils typically found near the ground surface.

Kulhawy (1990) suggested there is a relationship between the net limit pressure ($P\ell^*$), which is the difference between the limit pressure and the initial horizontal pressure, and the pressuremeter modulus. His equation, based on the cavity expansion theory, is as follows:

$$S_u = \frac{P\ell^*}{1 + \ln \frac{E_o}{3S_u}} \quad (6)$$

This equation can be solved by trial and error by assuming a value for the undrained shear strength (S_u) in the right-hand portion of the equation and comparing the calculated result with the original assumption.

Table 2 gives a summary of undrained shear strength values calculated using each of the equations presented above. Equation (4) yields a significantly different answer at deeper depths where the pressuremeter modulus is higher for clay-shales. This may imply the equation is not suited for clay-shales found at the test sites. Equation (5) yields inconsistent results with Equations (1), (2), (3), and (6). The undrained shear strength values calculated from tests performed near the surface are typically two to five times the values calculated from Equations (1), (2), (3), and (6). However, values calculated for deeper shale materials having higher

May Avenue, 19 Foot PMT Pressure vs Ln Volumetric Strain

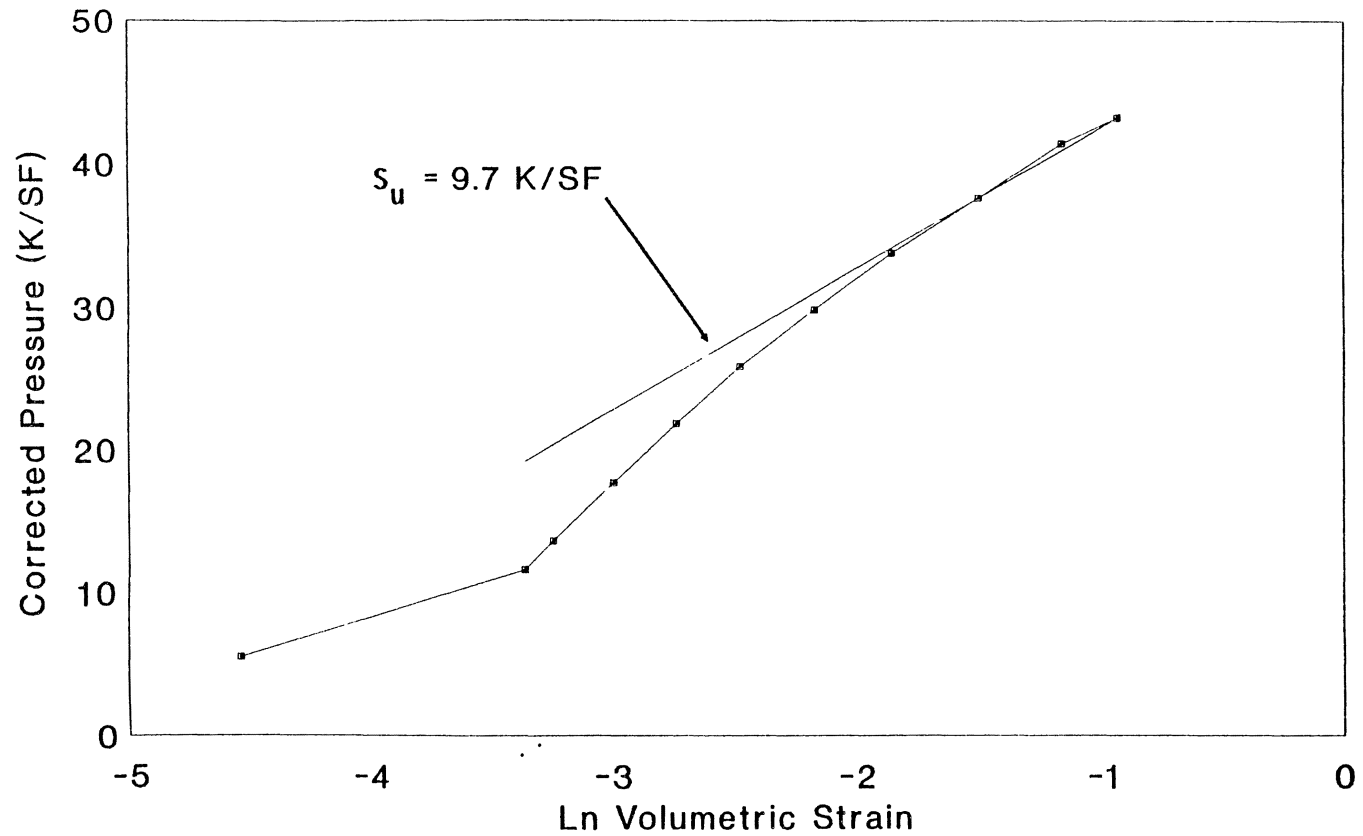


Figure 12. Pressure Versus Ln Volumetric Strain

TABLE 2
UNDRAINED SHEAR STRENGTH VALUES

Site	Depth (Feet)	Equation 1*	Equation 2*	Equation 3*	Equation 4*	Equation 5*	Equation 6*	Equation 7*
GAI (Hughes)	2.5	2.8	2.1	2.1	2.1	5.8	2.5	3.9
	7.5	3.3	2.7	2.7	2.7	5.5	2.8	2.8
	11.4	9.7	11.0	11.0	11.0	20.0	8.3	4.8
May	3.0	2.5	2.8	2.6	2.7	9.6	3.3	2.8
	6.0	2.4	2.7	2.6	2.4	N/A	3.2	2.7
	9.0	2.0	2.1	2.1	2.7	5.6	2.4	2.8
	12.5	3.2	4.1	3.6	6.2	11.3	4.0	3.7
	16.0	5.3	7.9	6.4	11.3	20.5	6.7	6.6
	19.0	4.5	6.3	5.3	8.3	9.7	5.7	5.5
	22.0	5.1	7.5	6.1	15.0	10.0	6.2	6.2
	25.0	4.1	5.5	4.6	2.5	15.0	5.8	5.0
	28.0	9.9	18.0	14.0	75.0	23.0	12.0	13.5
I44	2.5	2.2	2.3	2.3	1.12	10.3	3.1	2.5
	7.5	4.0	4.5	4.5	3.6	25.5	5.3	4.6
	10.0	3.6	4.0	4.0	0.9	13.5	5.6	4.3
	13.0	3.8	4.2	4.2	2.3	21.9	5.3	4.4
	16.5	11.7	17.5	17.5	46.8	92.0	15.3	15.5
	24.8	8.4	14.4	11.3	51.7	16.0	10.0	11.0
Bdwy	3.0	2.4	1.6	1.7	1.5	4.3	2.1	2.0
	7.0	12.5	24.5	18.9	21.7	N/A	17.6	18.4
	10.0	5.2	7.7	6.3	6.3	22.0	7.0	6.6
	12.0	4.2	5.7	4.8	10.4	18.2	5.1	5.0
	14.0	6.9	11.0	8.8	8.4	20.5	9.3	9.0
	17.0	18.0	40.0	30.5	100.0	400.0	24.4	28.2
Law	4.0	4.0	3.4	3.1	4.7	7.5	3.5	3.5
	8.0	6.1	6.0	5.0	18.3	12.0	5.0	5.5
	12.0	4.6	4.1	3.6	5.3	6.8	5.7	4.5
	16.0	6.9	7.0	5.7	12.2	8.0	6.0	6.4
	20.0	5.9	5.7	4.8	13.5	7.0	4.9	5.3

*Kips per square foot.

pressuremeter moduli seem to be more consistent with the other equations with the exception of Equation (4). This observation may imply that the assumption made for the volumetric strain may be incorrect for the more plastic residual soils but more reasonable for the deeper unweathered shales. Equation (7) is discussed below.

Unconfined compression tests conducted by Davidson (1982) at the GAI Site ranged from 4.0 to 9.2 ksf. These values are representative of values experienced by the author during his work in the Oklahoma City area over a 20-year period prior to this research. These values imply the undrained shear strength to be in the range of 2 to 5 ksf. Unfortunately, conventional laboratory testing was not performed on material from any of the research test sites. Poor recovery of sufficient sample lengths prevented the conduction of unconfined compression or triaxial shear tests. However, Standard Penetration Tests (SPT) were performed in residual soils at each site. Grain size analyses were also accomplished as a means to assist in logging the soils.

Kulhawy (1990) examined the relationship between SPT N-values and undrained shear strength, given in ksf, and established the relationship given by Equation (7) to be reasonably accurate when the same drilling equipment, SPT procedure, and consistent reference undrained shear strengths were employed:

$$S_u = 0.58 N^{0.72} \quad (7)$$

Although the SPT test is considered to be unreliable for design purposes by some geotechnical engineers, the test was used in this research as a means to correlate geotechnical similarities between the different sites. This could only be accomplished for the residual soils, since it is impossible to drive the test device into hard unweathered shales. Two different pieces of drilling equipment were used during the field work. The same equipment was used for the May and Lawton sites and another piece of equipment was used at the I-44 and Bdwy sites. The number of blows (N) counted during the STP was corrected, as recommended by Bowles (1988), for overburden pressure and the type of drilling rig to yield a

corrected value, N_{cor} . These values were used in Equation (7) to estimate the undrained shear strength as shown in Table 2. These same values and average values for undrained shear strength, as calculated from the pressuremeter test using Equations (1), (2), (3), and (6), are summarized in Table 3 along with the grain size analysis for soil passing a number 200 sieve, the Plasticity Index and the Liquid Limit.

Undrained shear strengths calculated from SPT results are not numerically equal to those calculated from pressuremeter tests; but they are in the same order of magnitude and should only be used as a means to correlate similarities between soils found at different sites rather than depending on absolute values. Relative values of undrained shear strength generally increase with depth with higher values being located at the interface of the weathered and unweathered shale and the shale. With the exception of the Bdwy site, where the STP was suspended once the sandstone was encountered, the upper eight feet of residual soils at the other sites had the approximate same shear strength values, with the Lawton site having the highest value. The May site had the deepest horizon of residual soils whereas the Lawton site had shale reported at a depth of 24.7 feet. Based on the STP results, one would expect the Lawton site to be a weathered shale if not a hard clay or claystone. Using the Unified Soil Classification System, residual soils for the May, I-44, Bdwy, and GAI sites have a classification of "CL" and can be considered to be quite similar. The higher Plasticity Index at the Lawton site produced a classification of "CH" which would also explain the higher shear strengths.

The Bdwy site had over three feet of interbedded sandstone and shale below the three-foot level. The results of the pressuremeter test at the seven-foot level indicate a much higher net limit pressure than observed in the other residual soils. The sandstone material will have a direct influence on a laterally loaded drilled

TABLE 3
RESIDUAL SOIL CHARACTERISTICS

Site	Depth	N	N _{Cor}	S _u SPT	Avg S _u MPM Test	% Passing #200 Sieve	PI	Liquid Limit
May	3.0	18.0	15.0	2.0	2.8	89	37.0	54.0
	6.0	21.0	12.0	1.7	2.7	94	31.0	46.0
	9.0	11.0	5.0	0.9	2.8	93	19.0	36.0
	12.5	28.0	11.0	1.6	3.7	99	21.0	45.0
	16.0	60.0	22.0	2.7	6.6	95	13.0	33.0
I-44	2.5	19.0	15.0	2.0	2.5	87	20.0	36.0
	7.5	56.0	26.0	3.0	4.6	96	14.0	33.0
Bdwy	3.0	20.0	15.0	2.0	2.0	90	N/A	38.0
Lawton	4.0	13.0	9.0	1.4	3.5	90	41.0	73.0
	8.0	25.0	13.0	1.8	5.5	82	29.0	58.0
	12.0	25.0	10.0	1.5	4.5	83	37.0	65.0
	16.0	39.0	14.0	1.9	6.4	87	44.0	66.0
	20.0	46.0	15.0	2.0	5.3	79	37.0	66.0
GAI	2.5	17.0	14.0	1.9	2.4	87	18.0	40.0
	7.5	19.0	9.0	1.4	2.9	81	27.8	47.0
	11.4	50.0	19.0	2.4	10.0	N/A	N/A	N/A

shaft, enabling it to support a much higher load as a result of the higher limit pressure.

The grain size analysis for the percent passing a number 200 sieve indicates that much of the material is clay size particles as supported by the Plasticity Index. This supports the conclusion that soils have similar classifications, although the Lawton site appears to have more clay size particles and a higher undrained shear strength. The information presented in Table 3 can be considered as a means to compare the five sites which are sufficiently similar geotechnically, and should exhibit similar strength trends but not necessarily the same absolute strength values.

Full Scale Drilled Shaft Load Test

Full-scale load tests were conducted as part of Davidson's (1982) work. Fourteen sites located throughout the United States were used in the investigation. Ten sites had granular soils and four sites had silts or combinations of silts and clays or shale. Only the GAI site had a clay overlying a shale. The other three sites, EPRI 1, 13, and 14, had cohesive-type soils. The EPRI 1 site, located in southwestern Pennsylvania, had alternating layers of stiff and medium stiff clayey silt overlying a sandy silt. The EPRI 13 site, located west of Portland, Oregon, had a stiff clayey silt for nearly the entire depth of the drilled shaft with a small layer of stiff silt at the bottom of the shaft. The EPRI 14 site, located in southwestern Iowa, had a stiff clayey silt overlying a medium stiff to stiff silty clay. Full-scale load test results at these four sites are shown in Figure 13. The four test shafts exhibit the same general shape for moment-deflection curves. All four test shafts have approximately the same deflection for groundline moments less than 650 k-ft. Davidson (1982) concluded the test shaft failed when a groundline moment caused a 2 degree rotation in the shaft. This caused a deflection of four to four and one-half inches. The groundline moment that caused the 2 degree rotation was different as

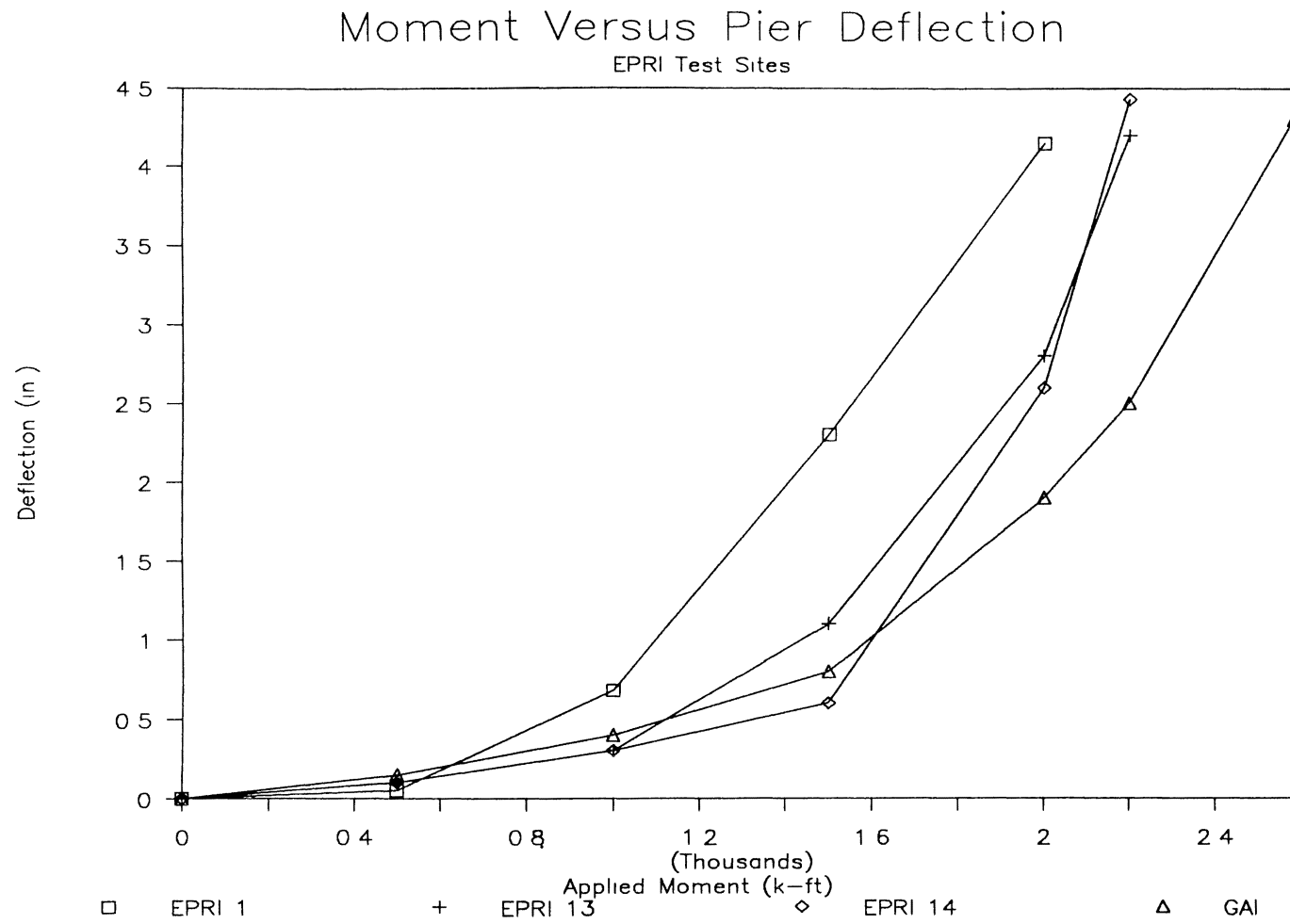


Figure 13. Moment Versus Pier Deflection, EPRI Test

one might expect, since the strength of the soils to resist the moment varies from one site to another. Nevertheless, the trend of small deflections when small groundline moments were obtained and much larger deflections when larger moments are applied was consistent at all four sites. Therefore, data developed from the GAI drilled shaft should be representative for drilled shafts constructed to support similar loads at locations with comparable soils.

Drilled Shaft Analysis Using Pressuremeter Data

Two finite difference models were used to determine the feasibility of using the pressuremeter data as input data for drilled pier design and analysis. The two models included the MFAD and BMCOL76 programs previously mentioned. Both programs require basic input parameters such as shape, diameter, length, moment of inertia, and modulus of elasticity for the drilled shaft.

The MFAD model uses input soil parameters of unit weight, pressuremeter modulus, friction angle, and cohesion. The program also uses a strength reduction factor developed to reduce the undrained shear strength of the soil. This reduction factor is a nonlinear function of the undrained shear strength and ranges from 0.40 to 1.0. The MFAD program is very "user friendly" but it does require an IBM PC compatible computer with a math co-processor; a 20 megabyte hard disk drive is also recommended. However, the author found a 10 megabyte hard disk drive to be sufficient. A sample of the input and output report is presented in Appendix B.

A pressuremeter reduction program (PRESRED) and a companion program for load-deflection (P-Y) curves (PYPMT) were developed by Briaud, Smith, and Tucker (1985) to complement the use of the BMCOL76 program. The PRESRED and PYPMT programs were written in BASIC computer language; the BMCOL76 program was written in FORTRAN and compiled for an 8088 IBM compatible computer. These programs were not user friendly and the documentation was found to be very marginal. The soil input data included field pressuremeter

readings--information regarding the pressuremeter equipment such as the size of the probe and the depth of the test. Information developed in the PRESRED program was stored on disk and used in the PYPMT program. Likewise, information developed in the PYPMT program was saved and used as input for the BMCOL76 program. Reports from the PRESRED program are contained with pressuremeter curves in Appendix A. PYPMT results for the May, Broadway, I-44 and Lawton sites are contained in Appendix C. Example input data and output reports which form the BMCOL76 program are presented in Appendix D.

The GAI site was used as the primary means to test the programs and the pressuremeter data. The full-scale test was conducted after a 60-inch reinforced concrete drilled shaft was constructed to a length of 13.5 feet with one foot of length extending above ground. The test shaft was loaded using a tapered octagonal steel pole with a measured horizontal load applied at the top of the pole to create an applied moment at the top of the shaft. Davidson's (1982) pressuremeter results were used in MFAD to compare deflection at the top of the drilled shaft with actual full-test shaft deflections. The PRESRED and PYPMT programs were used to create pressuremeter data from Davidson's uncorrected pressuremeter test curves. This information was then used in the BMCOL program to determine the calculated deflection at the top of a shaft having the same physical characteristic as the test shaft. Results of the MFAD and BMCOL analyses and the observed full-scale test are shown in Figure 14. The curve marked GAI uses the pressuremeter information reported by Davidson (1982). Values calculated from the pressuremeter test for the undrained shear strength as shown in Table 2 were used in the MFAD program as the "cohesion" values. The friction angles were assumed to be zero. The full-scale test shaft and the MFAD program output using data reported by Davidson (shown as GAI) is in good agreement as shown in Figure 14. However, the deflections calculated from the BMCOL program do not agree with the observed full-scale test shaft. The BMCOL curve is almost a linear plot whereas the

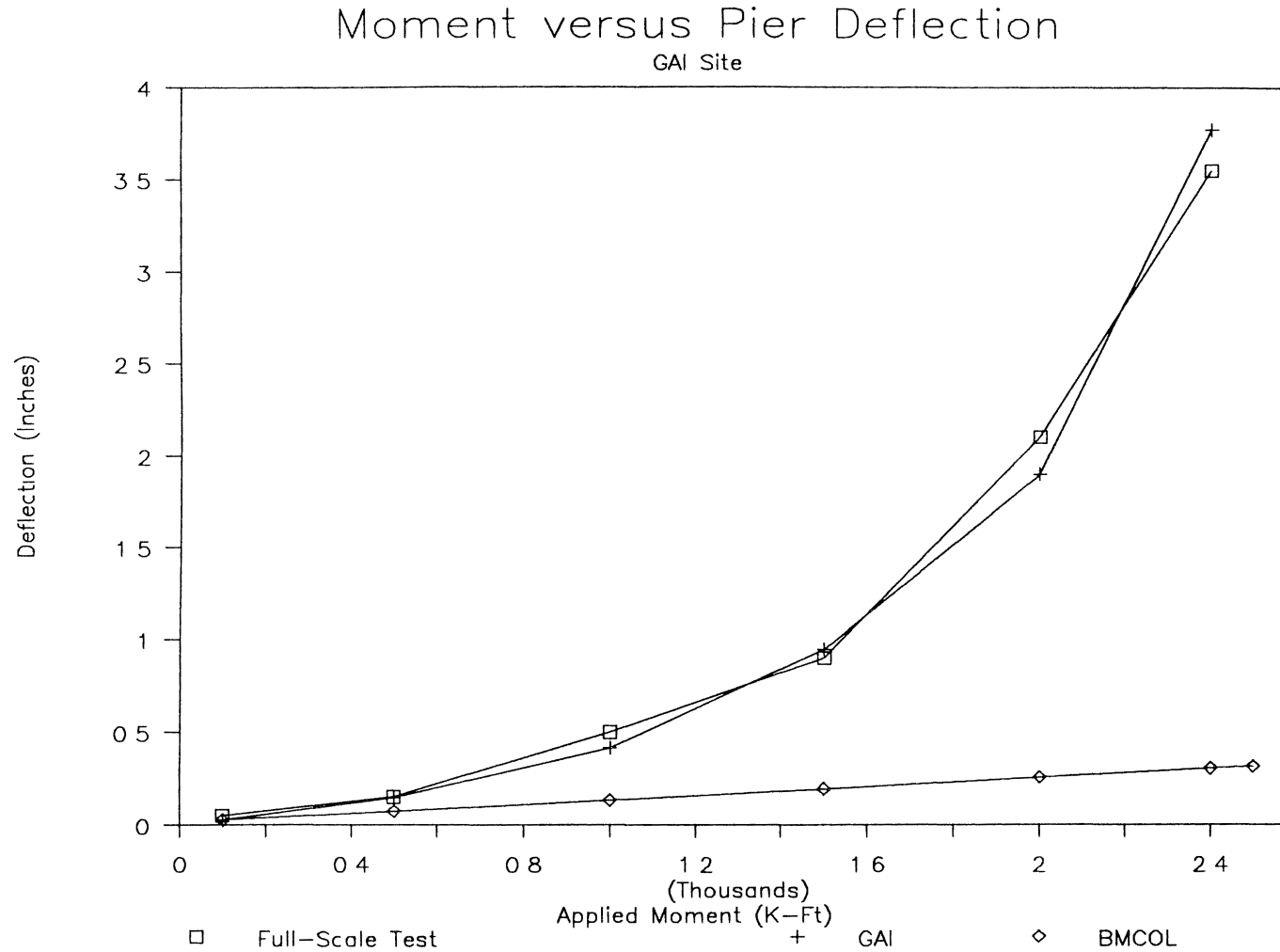


Figure 14. Moment Versus Deflection Curves, GAI Site

MFAD GAI plot appears to show much more deflection as the moment increases as did the full scale test shaft at the GAI site. Three possible explanations can be offered for the differences:

1. The pressuremeter curves used as input data for the BMCOL analysis were not corrected and therefore influenced the results.
2. The PRESRED and PYPMT programs may yield load deflection (P-Y) curves that are not representative of the behavior as a shaft placed and loaded in Permian clays and clay-shales.
3. The BMCOL program may not be applicable for Permian clays and clay-shales.

To explore these considerations, additional drilled shaft analyses were undertaken for a hypothetical 48-inch reinforced concrete drilled shaft, 18 feet long to be modeled using data from the other four sites. The shaft length was chosen so the shaft would extend well into the shale as opposed to the full-scale test shaft that only extended 2.5 feet into the shale. Values calculated as the undrained shear strength from the pressuremeter tests, shown in Table 2, were used for the "cohesion" values required as an input parameter in the MFAD program for each soil layer. Results of these analyses are shown in Figures 15,16,17, and 18 for the May, I-44, Bdwy, and Lawton sites, respectively. Again, the BMCOL curve is almost linear. The MFAD curve shows a marked increase in deflection as the moment exceeds 3,000 k-ft. The Bdwy site exhibited the ability to resist the greatest load. This is the result of the interbedded sandstone and shale layer located approximately three and one-half feet below the ground surface. The May site only had approximately 4 feet of shale supporting the hypothetical drilled shaft whereas the I-44 and Broadway sites had approximately 10 to 11 feet of shale surrounding the shaft. When this information is compared to the results obtained for the GAI site, the following observations can be made.

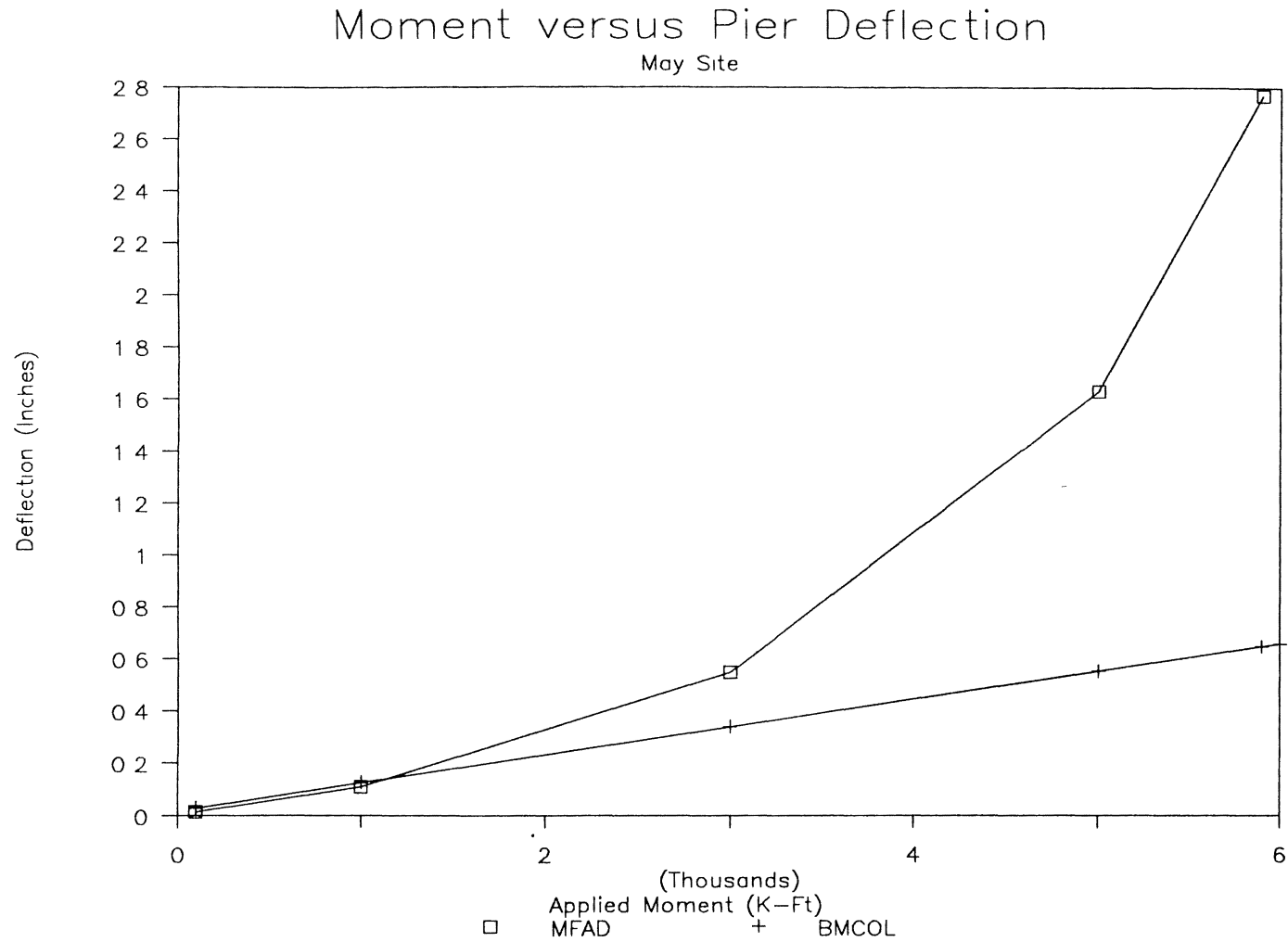


Figure 15. Moment Versus Deflection Curves, May Site

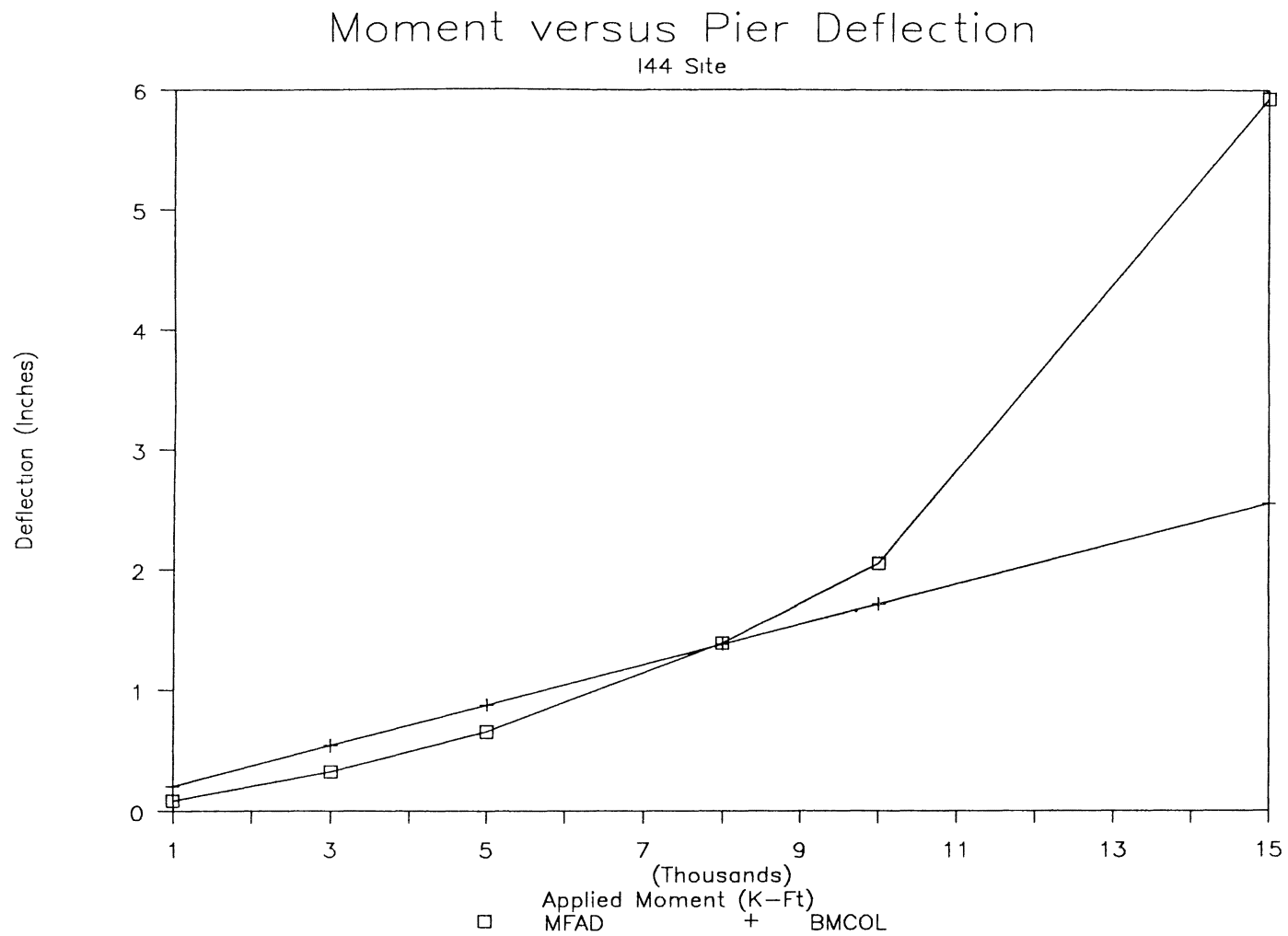


Figure 16. Moment Versus Deflection Curves, I-44 Site

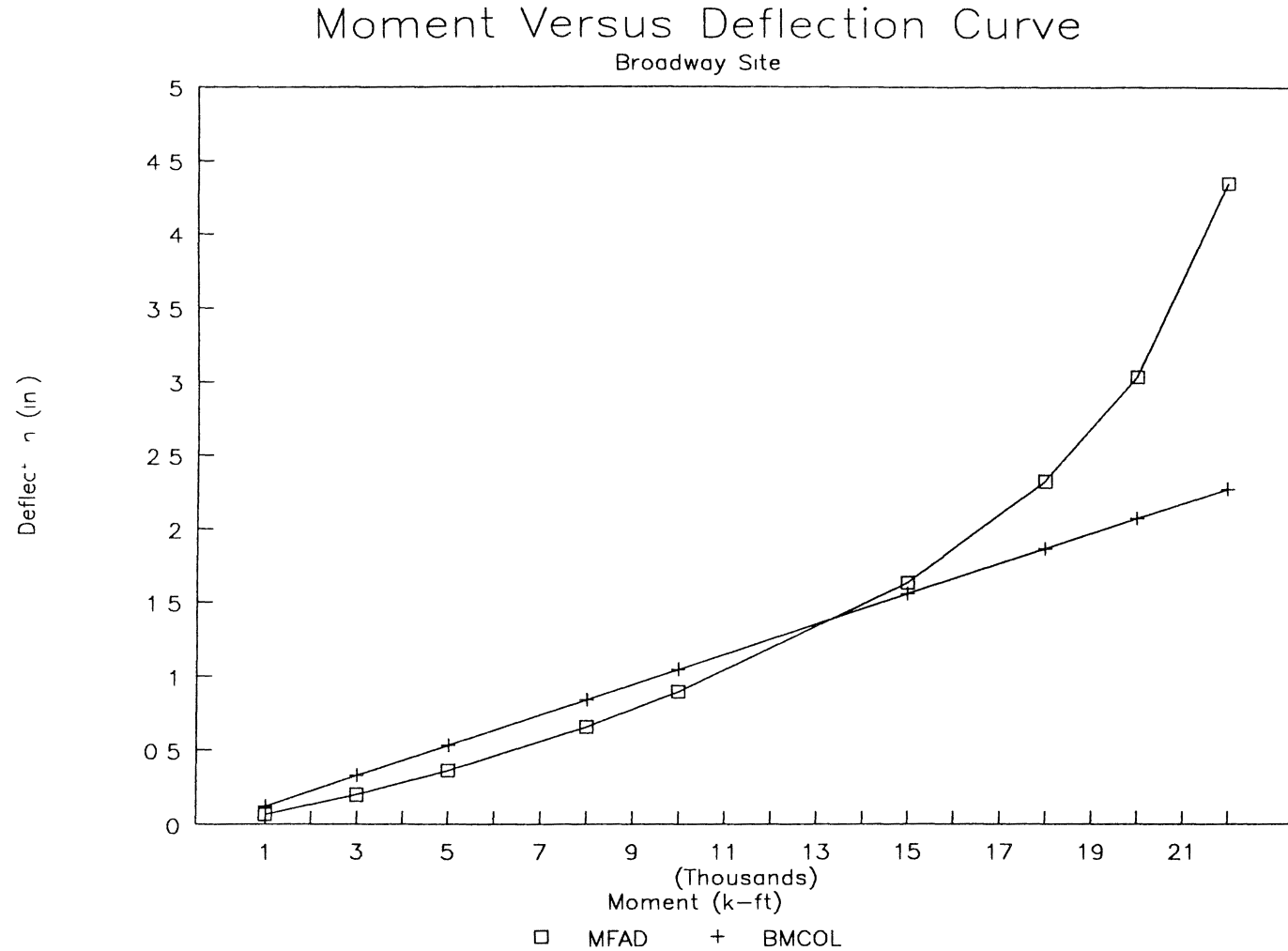


Figure 17. Moment Versus Deflection Curves, Broadway Site

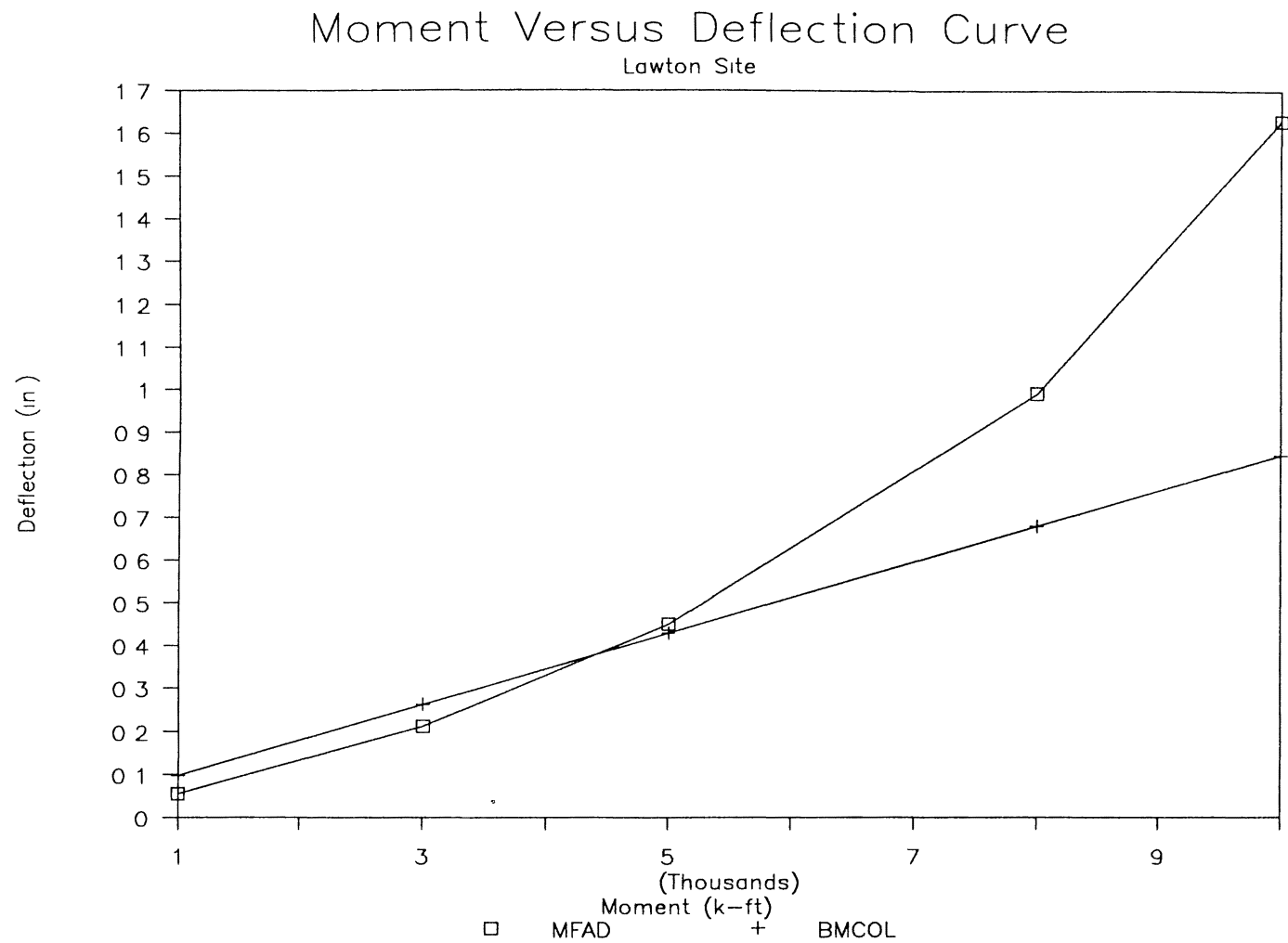


Figure 18. Moment Versus Deflection Curves, Lawton Site

1. The MFAD program produces results that agree with the full-scale load test trends as supported by the shape of the curves found in Figures 13 through 18.
2. The MFAD results for the GAI site agree with the full-scale drilled shaft test conducted at that site as shown in Figure 14.
3. The PRESRED, PYPMT and BMCOL76 programs do not appear to be suitable for use in the Permian clay-shales as evidenced by the minimal deflection calculated at large moments as compared to the deflections reported by the MFAD program and the full-scale test at the GAI site.
4. The pressuremeter appears to be suitable for use in residual soils based on the shape of the curves produced from MFAD data.
5. Results of the analyses appear to be influenced by the amount of shale or sandstone and the depth of the formations. This is evident at the I-44 and Bdwy sites where larger moments can be supported without excessive deflections.
6. The pressuremeter test results taken in the shales, do not appear to adversely affect the drilled shaft analyses as indicated by the shape of the curves.

CHAPTER VI

CONCLUSION AND RECOMMENDATIONS

Conclusions

The following conclusions can be drawn from the work presented herein:

1. A pressuremeter capable of reading 2,900 psi (200 bars) is needed if a true limit pressure is to be determined for the shales.
2. The type of pressuremeter should also be reconsidered for use in clay-shales. Although a pressuremeter used for prebored holes is still the best selection, the volume measurements should be measured using a probe with electro-mechanical feeler gauges. This will allow very small volume increases to be measured which will improve the accuracy of the test results.
3. The pressuremeter test can be used in Permian residual soils. A determination of how the pressuremeter test results might compare with a full-scale test shaft that extends well into the shales could not be determined by the work contained herein, but the trend shows promise based on the modeling results conducted with the MFAD program at the four test sites.
4. The PRESRED, PYPMT, and BMCOL76 programs do not appear suitable for analysis of laterally-loaded drilled shafts at sites where Permian clays and clays-shales are found without further studies.
5. The MFAD program appears to correlate well with actual field test results at the GAI site, but use of the pressuremeter and MFAD programs for the clay-shales should be applied cautiously since the full-scale test did not penetrate the clay-shale any significant distance.

Recommendations

This work should be considered as a preliminary investigation. Only one full-scale test has been conducted. The test shaft did not extend into the clay-shales a sufficient depth to determine exactly how the shaft might behave. Extensive use of the pressuremeter in the Permian clays and clay-shales has been limited. Additional work should be performed to reinforce and expand the knowledge gained from this research. Additional research should include the following:

1. Full-scale testing of drilled shafts constructed well into the Permian clay-shale layers.
2. Use of a higher capacity pressuremeter, preferably having an electro-mechanical sensing capability.
3. Perform unconfined compression and direct shear tests on soils at various depths to better correlate in-situ and laboratory data needed for the MFAD program.
4. Choose different sites that have the same soil characteristics.

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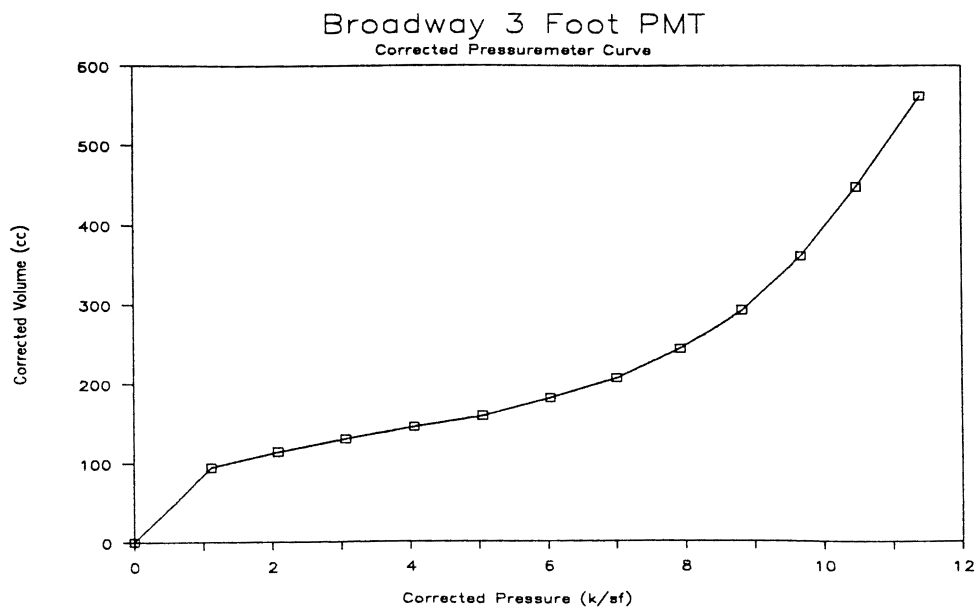
APPENDIX A

CORRECTED PRESSUREMETER DATA AND CURVES

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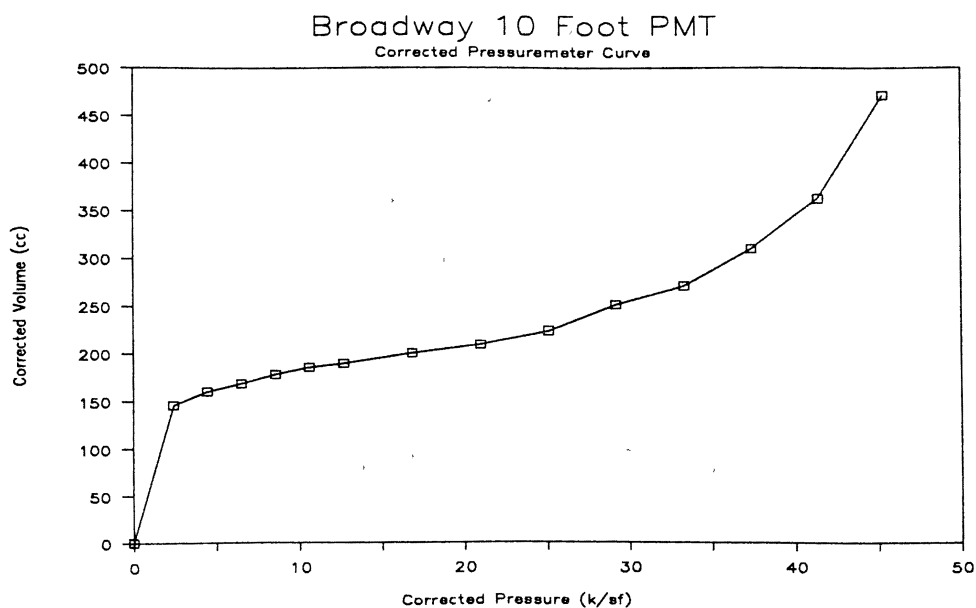
I 235 & Broadway Extension

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	17.000	0.0	0.00	0.00	0.44
2	112.000	0.5	94.70	5.83	1.12
3	132.000	1.0	114.40	7.00	2.09
4	148.000	1.5	130.10	7.93	3.07
5	164.000	2.0	145.80	8.85	4.06
6	178.000	2.5	159.50	9.64	5.06
7	200.000	3.0	181.20	10.89	6.04
8	226.000	3.5	206.90	12.35	7.01
9	264.000	4.0	244.60	14.46	7.93
10	313.000	4.5	293.30	17.12	8.83
11	381.000	5.0	361.00	20.73	9.68
12	468.000	5.5	447.70	25.20	10.48
13	582.000	6.0	561.40	30.83	11.40
Po = 1.0 ksf Pl = 12.8 ksf Pl* = 11.8 ksf Eo = 148 ksf Er = 0 ksf Eo/Pl* = 12.5					



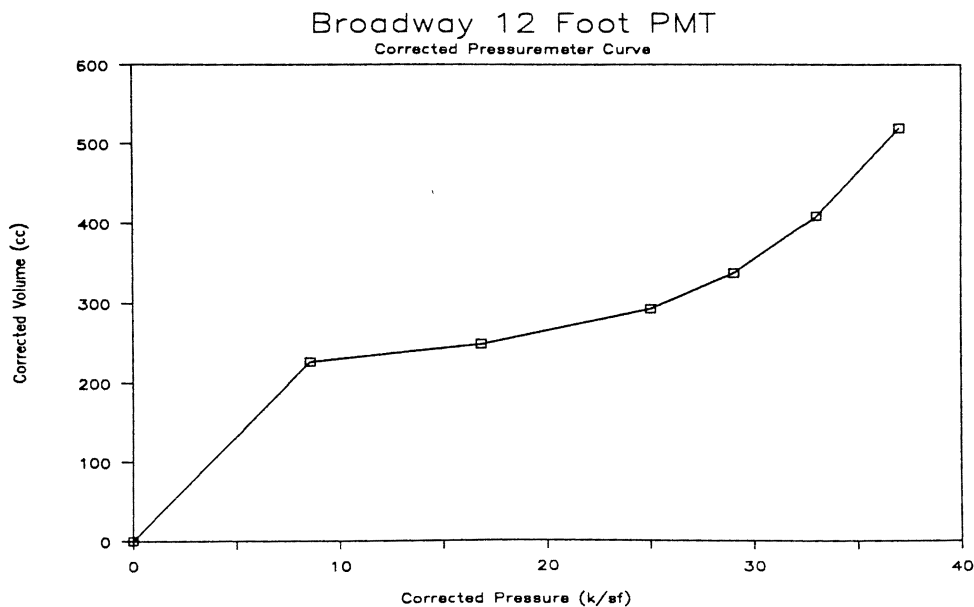
I 235 & Broadway Extension 10 Feet

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	57.000	0.0	0.00	0.00	0.87
2	203.000	1.0	145.40	8.83	2.42
3	218.000	2.0	159.80	9.66	4.46
4	227.000	3.0	168.20	10.15	6.52
5	237.000	4.0	177.60	10.68	8.57
6	245.000	5.0	185.00	11.11	10.64
7	250.000	6.0	189.40	11.36	12.71
8	262.000	8.0	200.20	11.97	16.85
9	272.000	10.0	209.00	12.47	20.99
10	288.000	12.0	223.80	13.30	25.12
11	317.000	14.0	251.60	14.84	29.21
12	338.000	16.0	271.60	15.94	33.32
13	377.000	18.0	309.80	18.01	37.38
14	430.000	20.0	362.00	20.78	41.41
15	539.000	22.0	471.00	26.37	45.28
Po =	2.0 ksf	Pl =	60.0 ksf	Pl* =	58.0 ksf
Eo =	627 ksf	Er =	0 ksf	Eo/Pl* =	10.8



I 235 & Broadway Extension 12 Feet

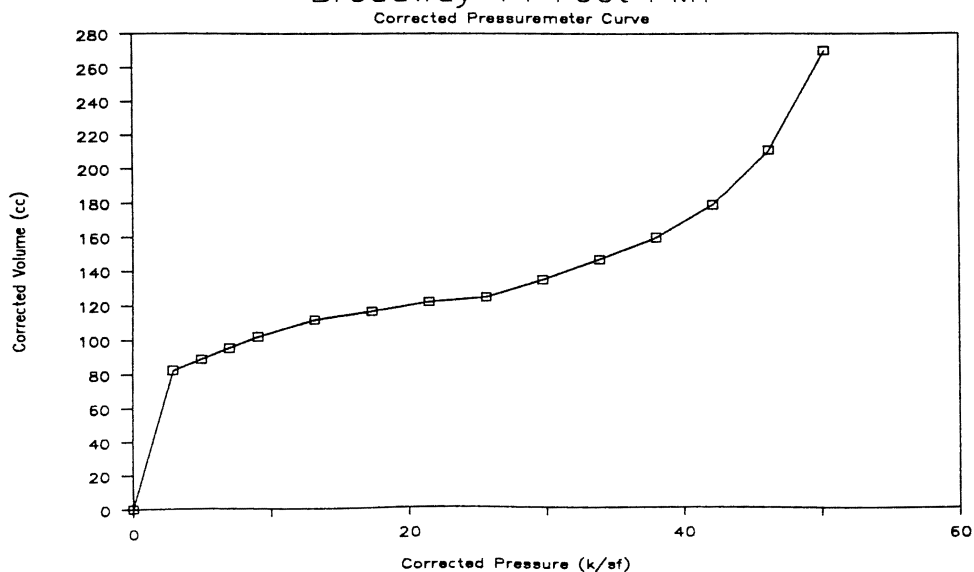
POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	70.000	0.0	0.00	0.00	1.00
2	299.000	4.0	226.60	13.46	8.55
3	323.000	8.0	248.20	14.66	16.83
4	370.000	12.0	292.80	17.10	25.04
5	416.000	14.0	337.60	19.50	29.08
6	489.000	16.0	409.60	23.26	33.05
7	600.000	18.0	519.80	28.80	37.01
Po = 5.0 ksf Pl = 48.0 ksf Pl* = 43.0 ksf Eo = 1046 ksf Er = 0 ksf Eo/Pl* = 24.3					



Interstate 235 & Broadway Extension 14 Foot

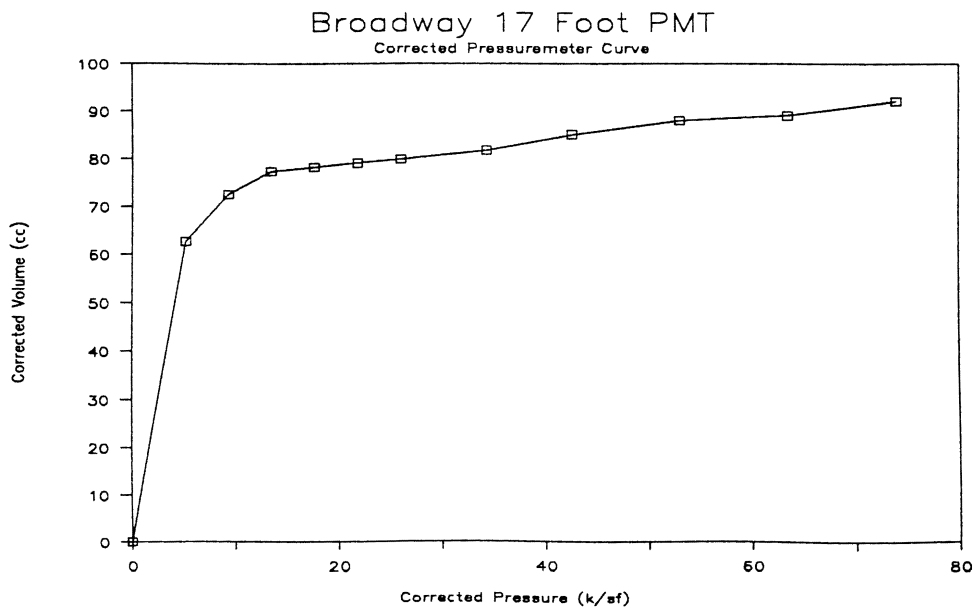
POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	155.000	0.0	0.00	0.00	1.12
2	238.000	1.0	82.40	5.09	2.90
3	245.000	2.0	88.80	5.48	4.96
4	252.000	3.0	95.20	5.86	7.02
5	259.000	4.0	101.60	6.24	9.08
6	270.000	6.0	111.40	6.83	13.21
7	276.000	8.0	116.20	7.11	17.37
8	283.000	10.0	122.00	7.45	21.52
9	287.000	12.0	124.80	7.62	25.68
10	298.000	14.0	134.60	8.19	29.81
11	311.000	16.0	146.60	8.90	33.95
12	325.000	18.0	159.80	9.66	38.08
13	345.000	20.0	179.00	10.77	42.20
14	377.000	22.0	211.00	12.58	46.28
15	436.000	24.0	270.00	15.85	50.27
Po =	2.2 ksf	Pl =	85.0 ksf	Pl* =	82.8 ksf
Eo =	838 ksf	Er =	0 ksf	Eo/Pl* =	10.1

Broadway 14 Foot PMT



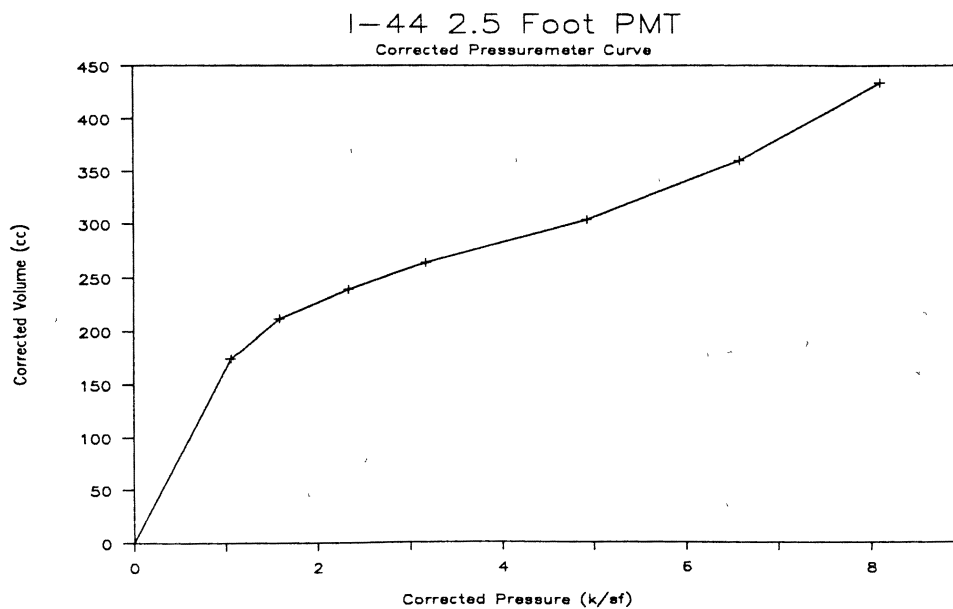
Interstate 235 & Broadway Extension 17 Feet

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	135.000	0.0	0.00	0.00	1.31
2	199.000	2.0	62.80	3.90	5.24
3	210.000	4.0	72.60	4.50	9.38
4	216.000	6.0	77.40	4.79	13.53
5	218.000	8.0	78.20	4.84	17.70
6	220.000	10.0	79.00	4.89	21.87
7	222.000	12.0	79.80	4.94	26.03
8	226.000	16.0	81.60	5.04	34.37
9	231.000	20.0	85.00	5.25	42.70
10	234.000	25.0	88.00	5.43	53.13
11	236.000	30.0	89.00	5.49	63.56
12	240.000	35.0	92.00	5.67	73.98
13	246.000	40.0	71.00	4.40	84.40
14	251.000	45.0	53.00	3.30	94.82
15	256.000	50.0	33.00	2.07	105.24
16	265.000	55.0	20.00	1.26	115.64
Po = 10.0 ksf Pl = 310.0 ksf Pl* = 300.0 ksf Eo = 10005 ksf Er = 0 ksf Eo/Pl* = 33.4					



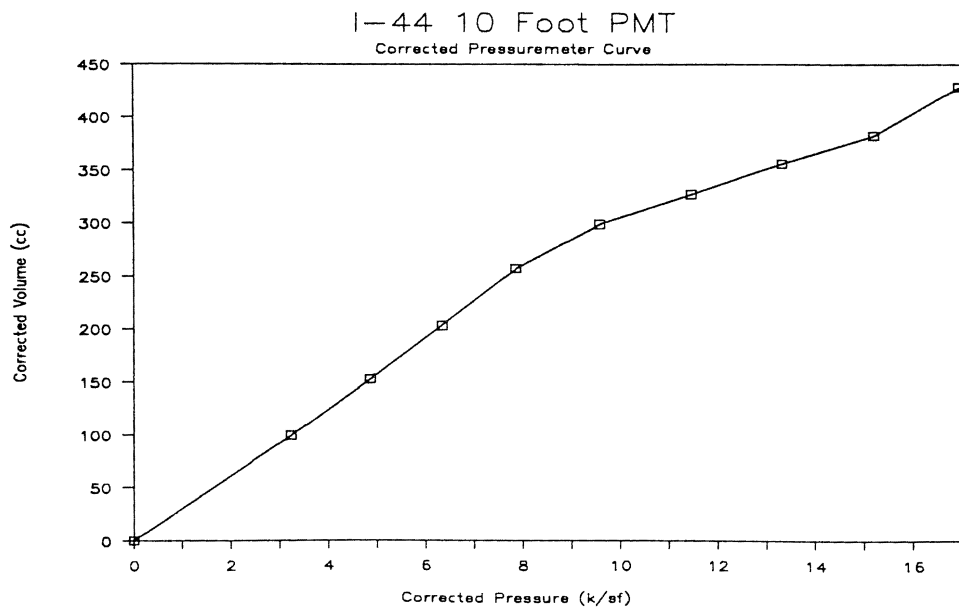
I44 2.5 PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	83.000	0.5	0.00	0.00	1.45
2	172.000	1.0	88.80	5.48	0.84
3	258.000	1.5	174.60	10.51	1.06
4	296.000	2.0	212.40	12.66	1.59
5	323.000	2.5	239.20	14.16	2.34
6	348.000	3.0	264.00	15.53	3.18
7	389.000	4.0	304.60	17.73	4.94
8	445.000	5.0	360.20	20.69	6.60
9	519.000	6.0	433.80	24.49	8.12
=====					
Po =	0.8 ksf	Pl =	18.8 ksf	Pl* =	18.0 ksf
Eo =	112 ksf	Er =	0 ksf	Eo/Pl* =	6.2
=====					



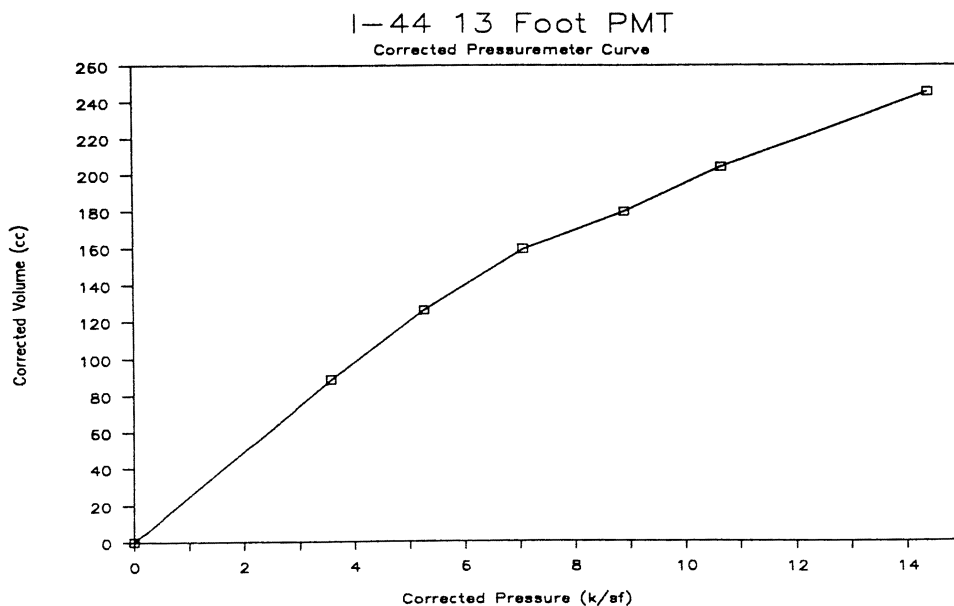
I44 10 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	172.000	1.0	0.00	0.00	2.96
2	273.000	2.0	100.60	6.18	3.24
3	326.000	3.0	153.20	9.28	4.87
4	376.000	4.0	202.80	12.12	6.35
5	431.000	5.0	257.40	15.16	7.87
6	474.000	6.0	300.00	17.48	9.61
7	502.000	7.0	327.60	18.96	11.49
8	531.000	8.0	356.20	20.48	13.35
9	558.000	9.0	382.80	21.87	15.23
10	604.000	10.0	428.40	24.22	16.97
Po =	3.0 ksf	Pl =	37.9 ksf	Pl* =	34.9 ksf
Eo =	87 ksf	Er =	0 ksf	Eo/Pl* =	2.5



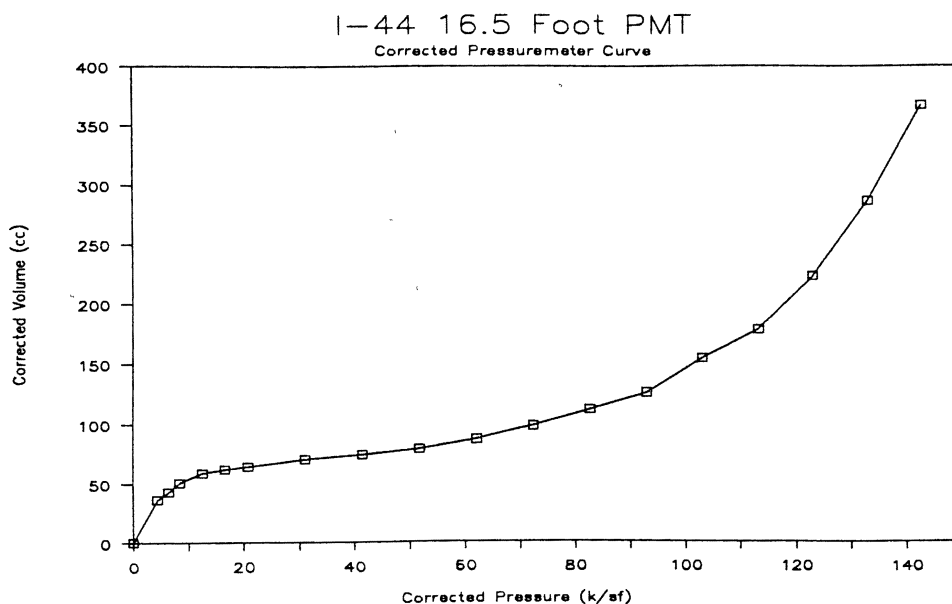
I44 13 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	196.000	1.0	0.00	0.00	3.15
2	285.000	2.0	88.60	5.47	3.58
3	323.000	3.0	126.20	7.70	5.28
4	357.000	4.0	159.80	9.66	7.09
5	378.000	5.0	180.40	10.85	8.93
6	402.000	6.0	204.00	12.19	10.69
7	444.000	8.0	245.20	14.49	14.41
=====					
Po =	3.5 ksf	P1 =	40.6 ksf	P1* =	37.1 ksf
Eo =	226 ksf	Er =	0 ksf	Eo/P1* =	6.1
=====					



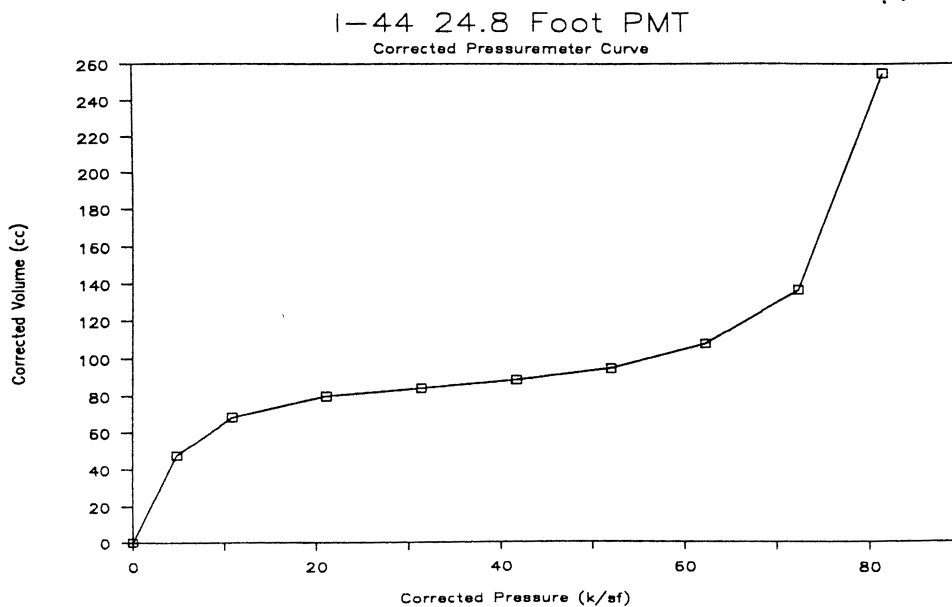
I44 16.5 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	154.000	1.0	0.00	0.00	3.37
2	191.000	2.0	36.60	2.29	4.47
3	198.000	3.0	43.20	2.70	6.47
4	206.000	4.0	50.80	3.17	8.45
5	215.000	6.0	59.00	3.67	12.51
6	219.000	8.0	62.20	3.87	16.64
7	222.000	10.0	64.40	4.00	20.77
8	230.000	15.0	70.40	4.37	31.11
9	236.000	20.0	74.40	4.61	41.47
10	243.000	25.0	79.40	4.91	51.82
11	252.000	30.0	87.20	5.38	62.14
12	264.000	35.0	98.20	6.04	72.45
13	279.000	40.0	111.80	6.85	82.76
14	295.000	45.0	126.54	7.72	93.07
15	325.000	50.0	155.34	9.40	103.24
16	350.000	55.0	179.46	10.79	113.34
17	395.000	60.0	223.50	13.28	123.25
18	459.000	65.0	287.30	16.80	133.17
19	539.000	70.0	367.30	21.06	142.99
Po = 7.5 ksf Pl = 177.0 ksf Pl* = 169.5 ksf Eo = 4677 ksf Er = 0 ksf Eo/Pl* = 27.6					



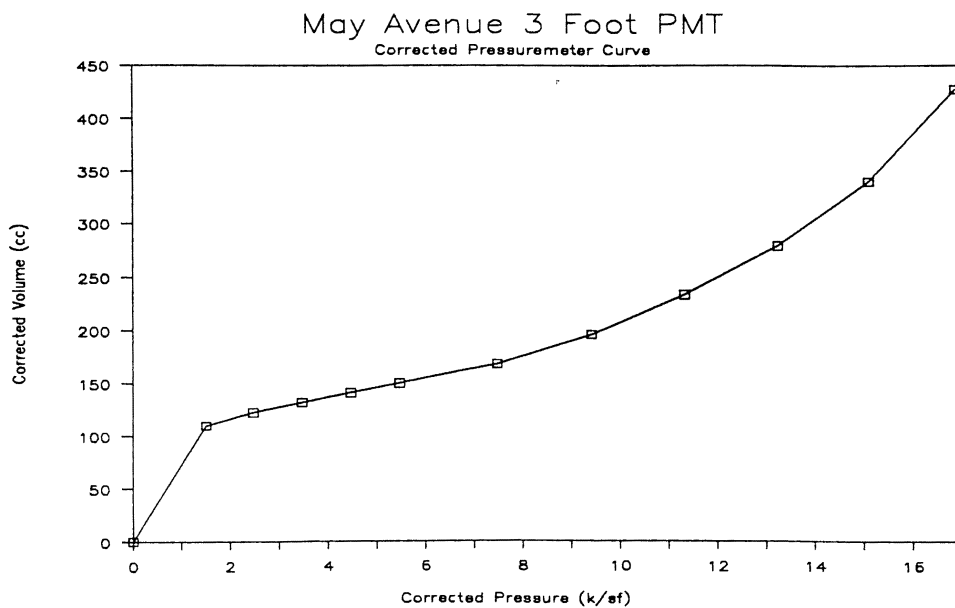
I44 24.8 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	179.000	1.0	0.00	0.00	3.88
2	227.000	2.0	47.60	2.97	4.85
3	249.000	5.0	68.40	4.24	10.83
4	262.000	10.0	79.40	4.91	21.10
5	268.000	15.0	83.40	5.15	31.46
6	274.000	20.0	87.40	5.39	41.82
7	282.000	25.0	93.40	5.75	52.15
8	296.000	30.0	106.20	6.52	62.47
9	326.000	35.0	135.20	8.23	72.66
10	447.000	40.0	254.80	15.02	81.79
Po = 10.0 ksf Pl = 118.0 ksf Pl* = 108.0 ksf Eo = 5165 ksf Er = 0 ksf Eo/Pl* = 47.8					



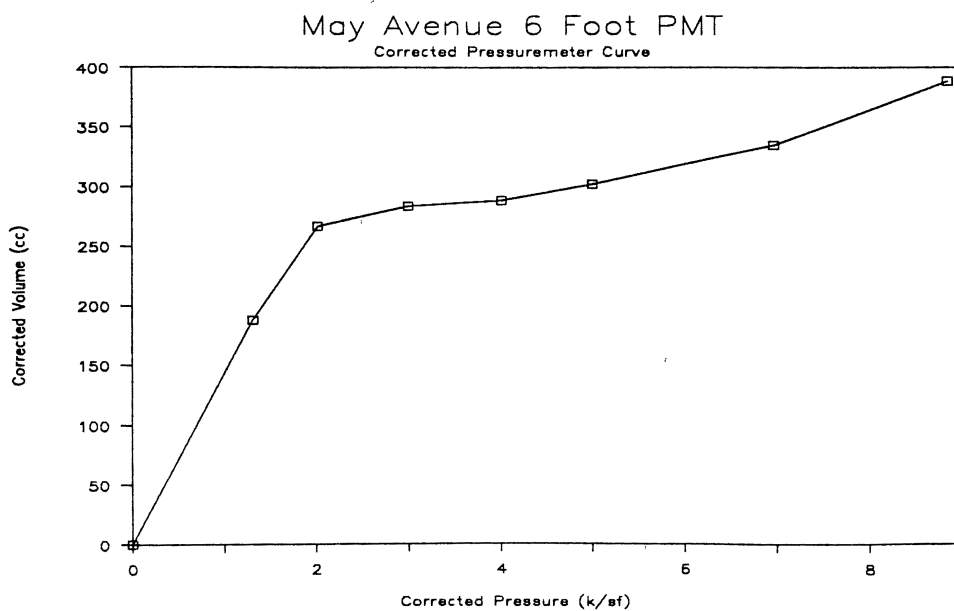
I 240 & May Ave 3 Foot PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	132.000	0.5	0.00	0.00	1.48
2	243.000	1.0	110.70	6.79	1.51
3	256.000	1.5	123.40	7.54	2.48
4	266.000	2.0	133.10	8.11	3.48
5	275.000	2.5	141.80	8.62	4.48
6	284.000	3.0	150.50	9.12	5.48
7	302.000	4.0	167.90	10.13	7.49
8	331.000	5.0	196.30	11.75	9.44
9	369.000	6.0	233.70	13.85	11.36
10	415.000	7.0	279.10	16.35	13.27
11	477.000	8.0	340.50	19.65	15.12
12	564.000	9.0	426.90	24.14	16.89
Po =	1.5 ksf	Pl =	22.7 ksf	Pl* =	21.2 ksf
Eo =	272 ksf	Er =	0 ksf	Eo/Pl* =	12.8



I 240 & May Ave 6 Foot PMT

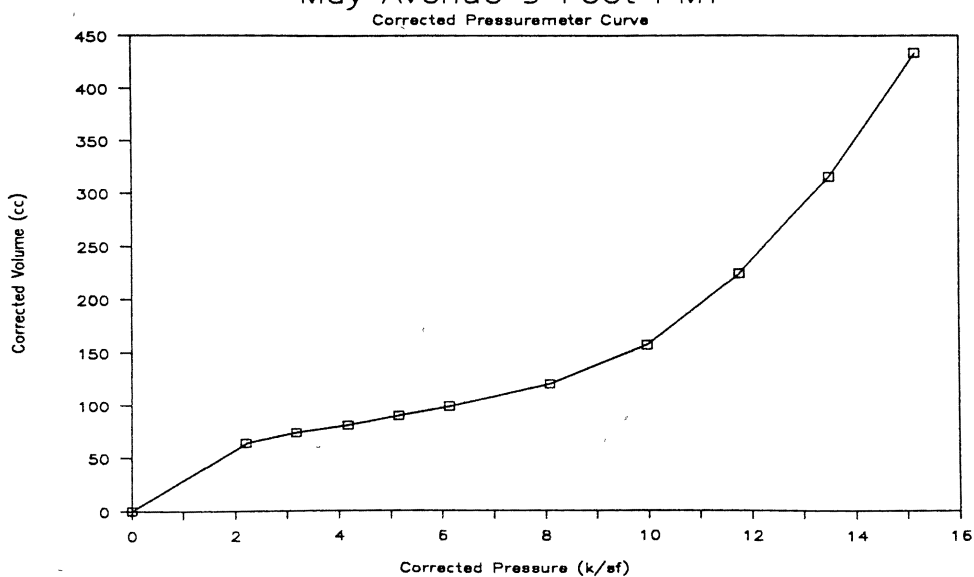
POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	146.000	0.5	0.00	0.00	1.67
2	335.000	1.0	188.70	11.32	1.32
3	414.000	1.5	267.40	15.71	2.03
4	431.000	2.0	284.10	16.62	3.01
5	436.000	2.5	288.80	16.88	4.03
6	450.000	3.0	302.50	17.62	5.02
7	483.000	4.0	334.90	19.35	6.99
8	537.000	5.0	388.30	22.16	8.87
Po = 1.4 ksf Pl = 22.0 ksf Pl* = 20.6 ksf Eo = 243 ksf Er = 0 ksf Eo/Pl* = 11.8					



I 240 & May Ave 9 Foot PMT

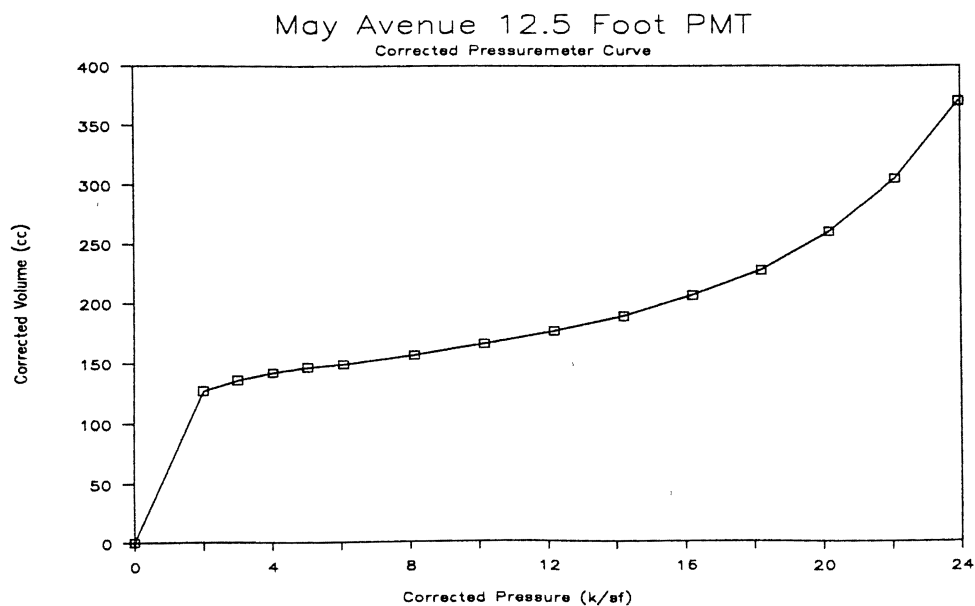
POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	164.000	0.5	0.00	0.00	1.86
2	229.000	1.0	64.70	4.02	2.21
3	239.000	1.5	74.40	4.61	3.18
4	246.000	2.0	81.10	5.01	4.18
5	255.000	2.5	89.80	5.54	5.16
6	264.000	3.0	98.50	6.06	6.14
7	286.000	4.0	119.90	7.33	8.09
8	324.000	5.0	157.30	9.52	9.99
9	391.000	6.0	223.70	13.29	11.77
10	484.000	7.0	316.10	18.35	13.50
11	602.000	8.0	433.50	24.48	15.15
Po = 2.0 ksf Pl = 17.6 ksf Pl* = 15.6 ksf Eo = 269 ksf Er = 0 ksf Eo/Pl* = 17.2					

May Avenue 9 Foot PMT



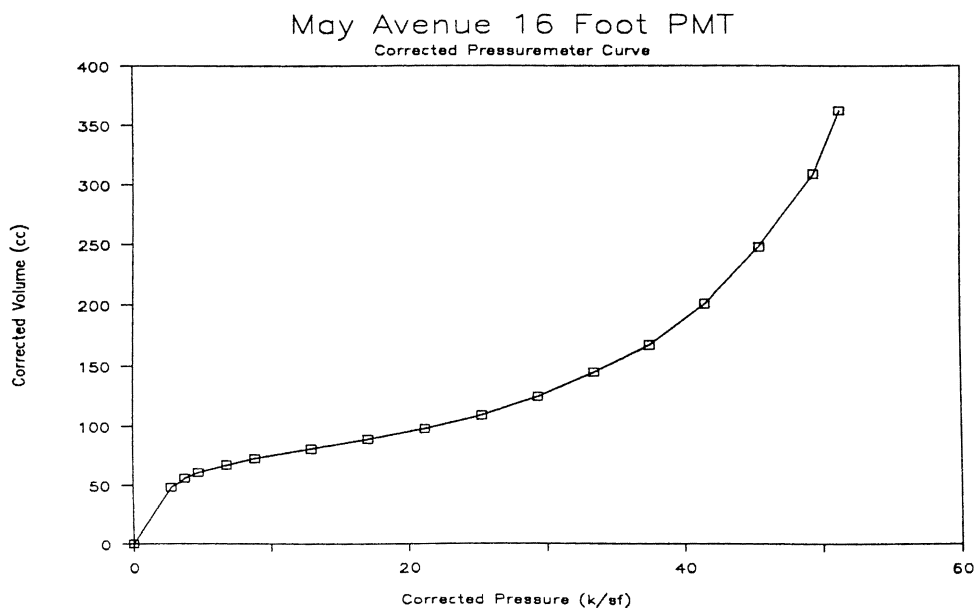
I 240 & May Ave 12.5 Foot PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	183.000	0.5	0.00	0.00	2.07
2	311.000	1.0	127.70	7.79	2.01
3	320.000	1.5	136.40	8.30	3.01
4	326.000	2.0	142.10	8.63	4.03
5	331.000	2.5	146.80	8.91	5.05
6	334.000	3.0	149.50	9.06	6.08
7	342.000	4.0	156.90	9.49	8.13
8	352.000	5.0	166.30	10.04	10.17
9	363.000	6.0	176.70	10.63	12.21
10	376.000	7.0	189.10	11.34	14.23
11	394.000	8.0	206.50	12.33	16.24
12	416.000	9.0	227.90	13.53	18.23
13	449.000	10.0	260.30	15.32	20.19
14	494.000	11.0	304.70	17.74	22.10
15	560.000	12.0	370.10	21.21	23.94
Po =	2.0 ksf	Pl =	32.6 ksf	Pl* =	30.6 ksf
Eo =	621 ksf	Er =	0 ksf	Eo/Pl* =	20.3



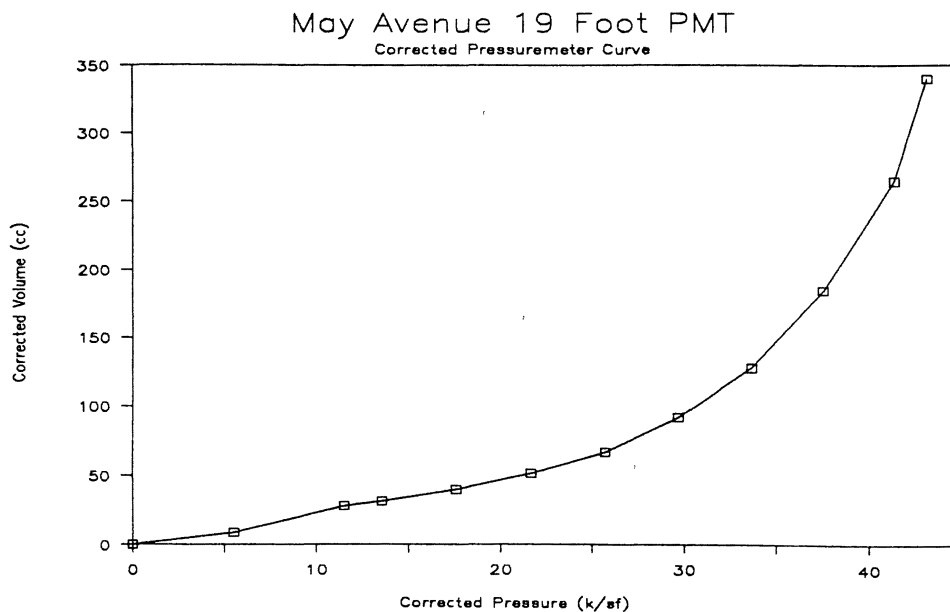
I 240 & May Ave 16 Foot PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	202.000	0.5	0.00	0.00	2.29
2	251.000	1.0	48.70	3.04	2.76
3	259.000	1.5	56.40	3.51	3.74
4	264.000	2.0	61.10	3.80	4.75
5	271.000	3.0	67.50	4.19	6.79
6	277.000	4.0	72.90	4.52	8.84
7	286.000	6.0	80.70	4.99	12.95
8	295.000	8.0	88.50	5.46	17.06
9	305.000	10.0	97.30	5.99	21.17
10	317.000	12.0	108.10	6.63	25.26
11	333.000	14.0	122.90	7.51	29.36
12	354.000	16.0	142.80	8.67	33.43
13	377.000	18.0	165.00	9.96	37.50
14	411.000	20.0	198.20	11.86	41.52
15	459.000	22.0	245.40	14.50	45.49
16	522.000	24.0	307.60	17.89	49.43
17	577.000	25.0	362.20	20.79	51.31
Po = 2.5 ksf Pl = 61.6 ksf Pl* = 59.1 ksf Eo = 1127 ksf Er = 0 ksf Eo/Pl* = 19.1					



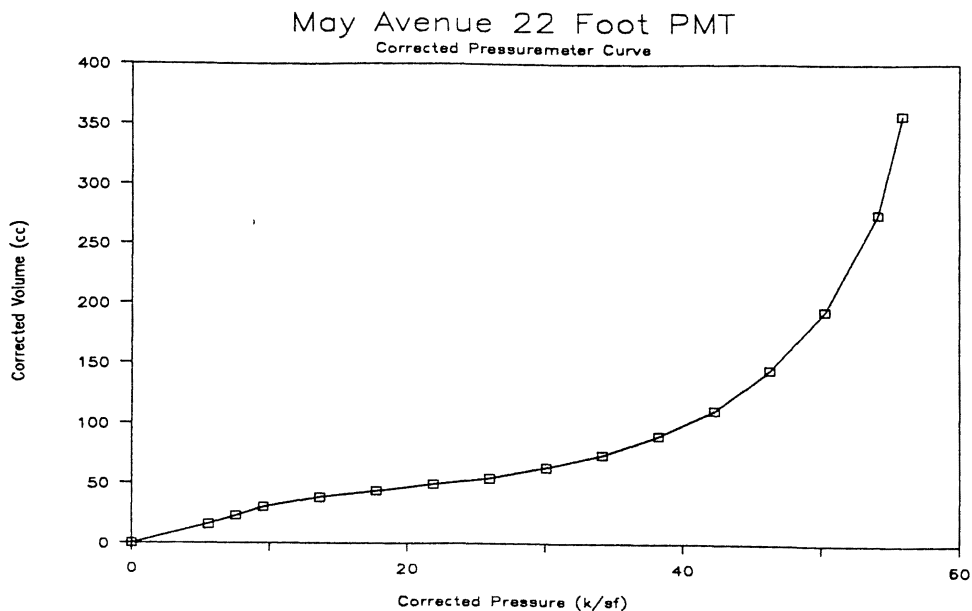
I 240 & May Ave 19 Foot PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	249.000	1.0	0.00	0.00	3.52
2	258.000	2.0	8.40	0.53	5.50
3	279.000	5.0	27.60	1.73	11.49
4	283.000	6.0	31.00	1.95	13.53
5	293.000	8.0	39.80	2.49	17.59
6	306.000	10.0	51.60	3.22	21.68
7	323.000	12.0	67.40	4.18	25.73
8	349.000	14.0	92.20	5.68	29.72
9	386.000	16.0	128.00	7.81	33.69
10	444.000	18.0	185.20	11.12	37.59
11	525.000	20.0	265.40	15.60	41.43
12	600.000	21.0	340.00	19.62	43.23
Po = 4.0 ksf Pl = 51.6 ksf Pl* = 47.6 ksf Eo = 833 ksf Er = 0 ksf Eo/Pl* = 17.5					



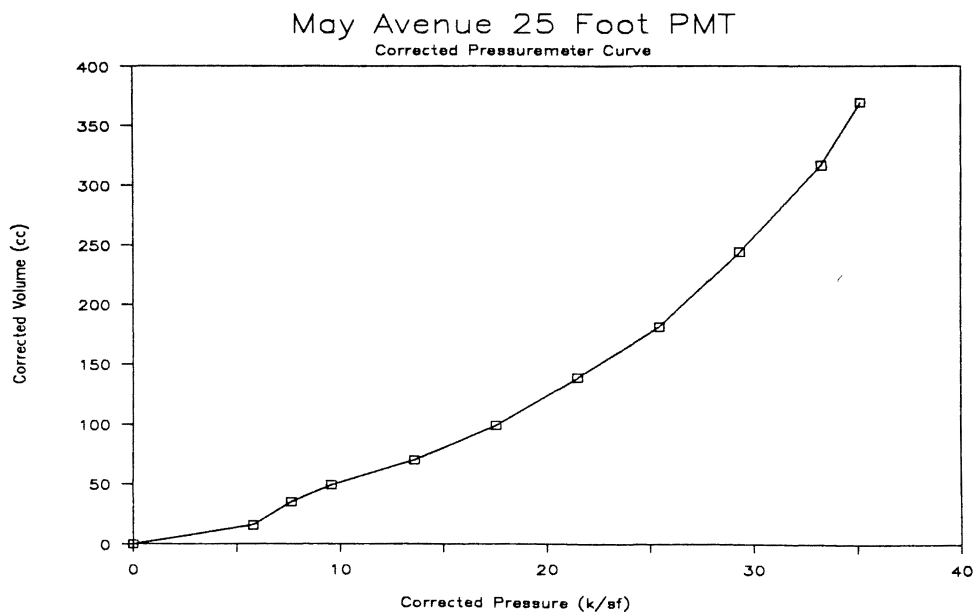
I 240 & May Ave 22 Foot PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	176.000	1.0	0.00	0.00	3.71
2	192.000	2.0	15.40	0.97	5.59
3	200.000	3.0	22.80	1.43	7.58
4	208.000	4.0	30.20	1.90	9.57
5	217.000	6.0	38.00	2.38	13.63
6	224.000	8.0	43.80	2.74	17.75
7	231.000	10.0	49.60	3.10	21.88
8	237.000	12.0	54.40	3.39	26.01
9	247.000	14.0	63.20	3.93	30.12
10	259.000	16.0	74.00	4.58	34.21
11	276.000	18.0	90.20	5.56	38.26
12	298.000	20.0	111.40	6.83	42.30
13	333.000	22.0	145.60	8.84	46.31
14	383.000	24.0	194.80	11.67	50.25
15	464.000	26.0	275.00	16.13	54.10
16	547.000	27.0	357.80	20.56	55.87
Po = 6.5 ksf Pl = 62.4 ksf Pl* = 55.9 ksf Eo = 1502 ksf Er = 0 ksf Eo/Pl* = 26.9					



I 240 & May Ave 25 Foot PMT

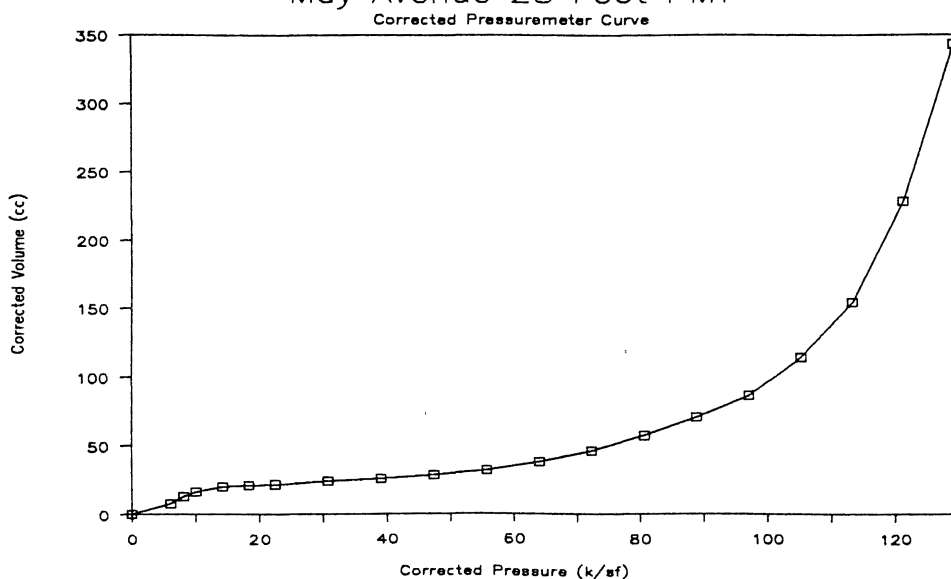
POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	211.000	1.0	0.00	0.00	3.90
2	227.000	2.0	15.40	0.97	5.78
3	247.000	3.0	34.80	2.18	7.61
4	262.000	4.0	49.20	3.07	9.57
5	284.000	6.0	70.00	4.34	13.59
6	314.000	8.0	98.80	6.08	17.55
7	355.000	10.0	138.60	8.43	21.51
8	399.000	12.0	181.40	10.90	25.47
9	463.000	14.0	244.20	14.43	29.37
10	537.000	16.0	317.00	18.40	33.27
11	590.000	17.0	369.60	21.18	35.15
Po =	4.0 ksf	Pl =	45.2 ksf	Pl* =	41.2 ksf
Eo =	249 ksf	Er =	0 ksf	Eo/Pl* =	6.0



I 240 & May Ave 28 Foot PMT

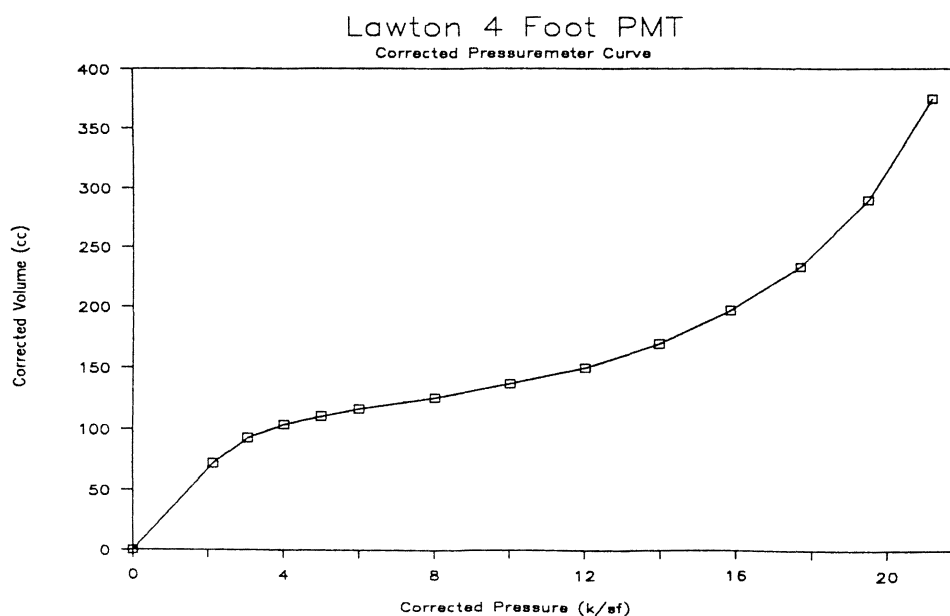
POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	179.000	1.0	0.00	0.00	4.08
2	187.000	2.0	7.40	0.47	6.07
3	193.000	3.0	12.80	0.81	8.08
4	197.000	4.0	16.20	1.02	10.12
5	202.000	6.0	20.00	1.26	14.23
6	204.000	8.0	20.80	1.31	18.38
7	206.000	10.0	21.60	1.36	22.53
8	211.000	14.0	24.20	1.52	30.82
9	215.000	18.0	26.20	1.65	39.12
10	219.000	22.0	28.60	1.80	47.42
11	224.000	26.0	32.00	2.01	55.73
12	231.000	30.0	38.20	2.39	64.03
13	240.000	34.0	46.40	2.90	72.32
14	252.000	38.0	57.60	3.59	80.58
15	268.000	42.0	71.20	4.41	88.82
16	291.000	46.0	87.00	5.37	97.01
17	326.000	50.0	114.80	7.03	105.19
18	373.000	54.0	155.20	9.39	113.32
19	453.000	58.0	228.80	13.58	121.33
20	572.000	62.0	343.00	19.78	129.24
Po = 10.0 ksf Pl = 145.1 ksf Pl* = 135.1 ksf Eo = 7495 ksf Er = 0 ksf Eo/Pl* = 55.5					

May Avenue 28 Foot PMT



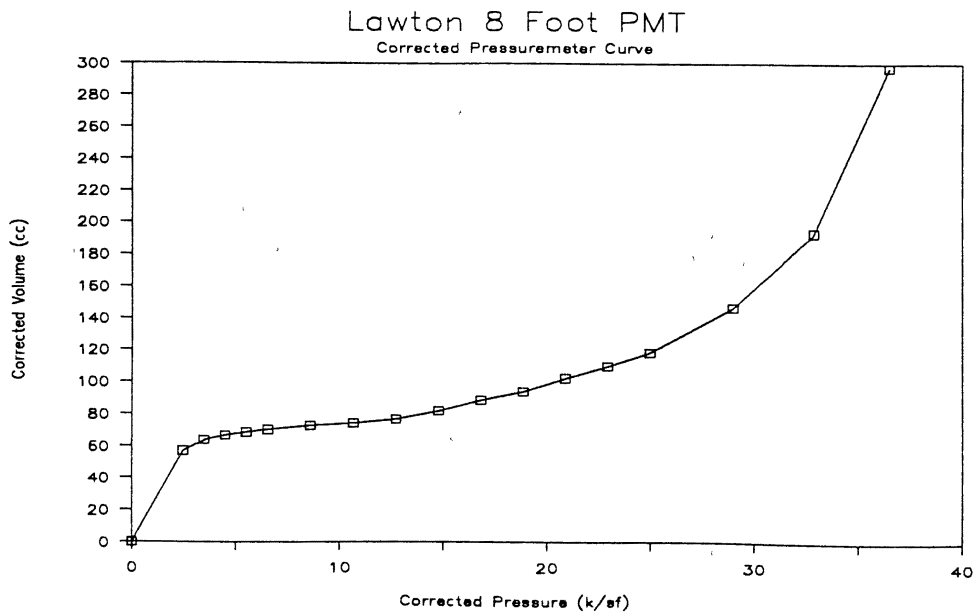
LAWTON 4 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	93.000	0.5	0.00	0.00	1.54
2	166.000	1.0	71.80	4.45	2.14
3	188.000	1.5	92.60	5.71	3.05
4	200.000	2.0	103.40	6.35	4.02
5	208.000	2.5	110.20	6.76	5.01
6	215.000	3.0	116.00	7.10	6.01
7	226.000	4.0	124.60	7.61	8.03
8	240.000	5.0	136.20	8.29	10.03
9	254.000	6.0	148.70	9.02	12.04
10	274.000	7.0	168.10	10.14	14.00
11	302.000	8.0	195.50	11.71	15.92
12	339.000	9.0	231.90	13.75	17.78
13	397.000	10.0	289.30	16.91	19.57
14	483.000	11.0	374.90	21.46	21.27
Po =	3.0 ksf	Pl =	28.8 ksf	Pl* =	25.8 ksf
Eo =	468 ksf	Er =	0 ksf	Eo/Pl* =	18.1



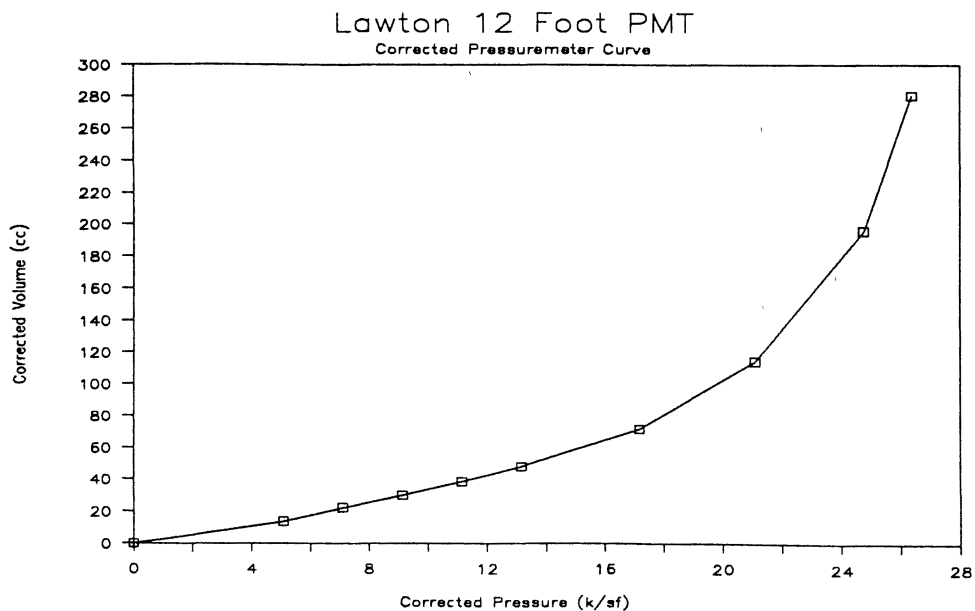
LAWTON 8 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	128.000	0.5	0.00	0.00	1.79
2	186.000	1.0	56.80	3.54	2.48
3	194.000	1.5	63.60	3.95	3.48
4	198.000	2.0	66.40	4.12	4.49
5	201.000	2.5	68.20	4.23	5.52
6	204.000	3.0	70.00	4.34	6.55
7	209.000	4.0	72.60	4.50	8.60
8	213.000	5.0	74.20	4.60	10.67
9	217.000	6.0	76.70	4.75	12.73
10	223.000	7.0	82.10	5.07	14.78
11	230.000	8.0	88.50	5.46	16.82
12	236.000	9.0	93.90	5.78	18.87
13	245.000	10.0	102.30	6.29	20.91
14	253.000	11.0	109.90	6.74	22.95
15	262.000	12.0	118.70	7.26	24.98
16	291.000	14.0	147.30	8.94	28.98
17	338.000	16.0	193.90	11.61	32.86
18	442.000	18.0	297.50	17.35	36.47
Po = 3.5 ksf Pl = 48.5 ksf Pl* = 45.0 ksf Eo = 1829 ksf Er = 0 ksf Eo/Pl* = 40.6					



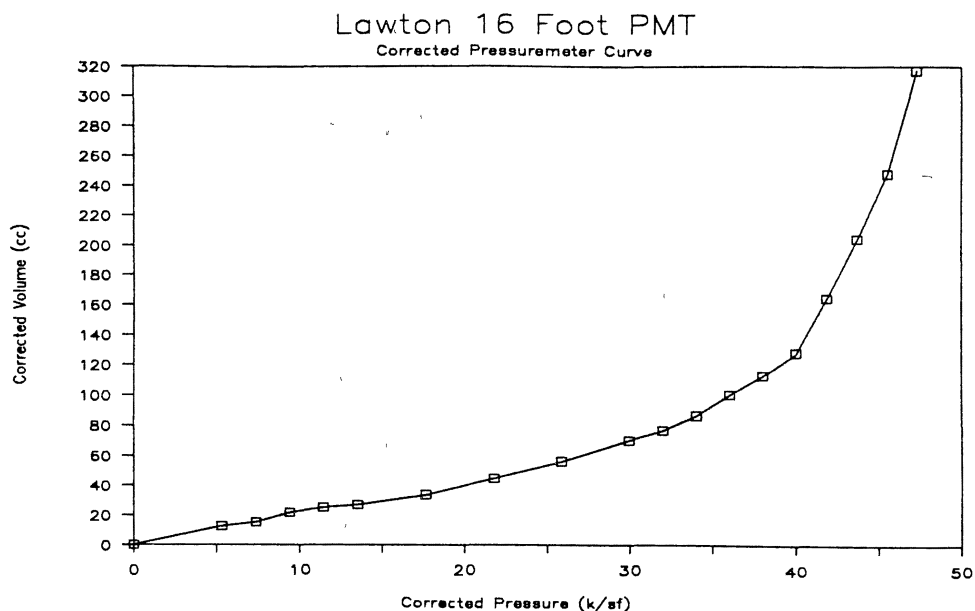
LAWTON 12 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	186.000	1.0	0.00	0.00	3.09
2	202.000	2.0	13.60	0.86	5.08
3	213.000	3.0	22.20	1.40	7.10
4	223.000	4.0	29.80	1.87	9.12
5	234.000	5.0	38.40	2.40	11.14
6	246.000	6.0	48.00	3.00	13.16
7	271.000	8.0	71.80	4.45	17.18
8	315.000	10.0	114.60	7.02	21.08
9	397.000	12.0	195.80	11.72	24.76
10	482.000	13.0	280.60	16.43	26.37
Po =	3.8 ksf	Pl =	34.8 ksf	Pl* =	31.0 ksf
Eo =	530 ksf	Er =	0 ksf	Eo/Pl* =	17.1



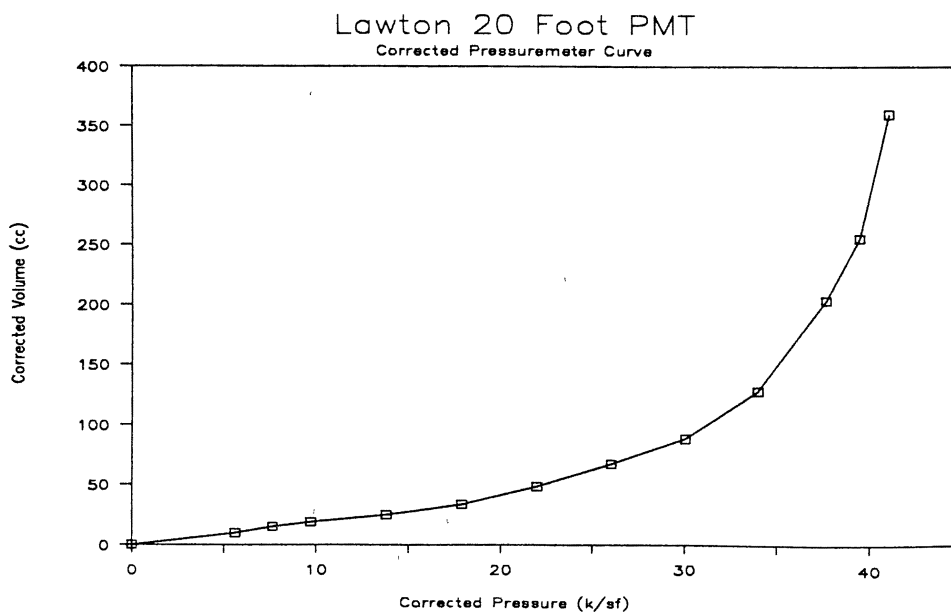
LAWTON 16 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	195.000	1.0	0.00	0.00	3.34
2	210.000	2.0	12.60	0.80	5.33
3	215.000	3.0	15.20	0.96	7.39
4	224.000	4.0	21.80	1.37	9.42
5	230.000	5.0	25.40	1.60	11.47
6	234.000	6.0	27.00	1.70	13.53
7	242.000	8.0	33.80	2.12	17.66
8	254.000	10.0	44.60	2.79	21.76
9	266.000	12.0	55.80	3.48	25.86
10	280.000	14.0	69.40	4.31	29.95
11	287.000	15.0	76.20	4.72	32.00
12	297.000	16.0	86.00	5.31	34.02
13	311.000	17.0	99.80	6.14	36.03
14	324.000	18.0	112.60	6.90	38.03
15	339.000	19.0	127.40	7.77	40.03
16	376.000	20.0	164.20	9.91	41.89
17	416.000	21.0	204.00	12.19	43.73
18	460.000	22.0	247.60	14.62	45.56
19	530.000	23.0	317.20	18.41	47.31
Po =	4.0 ksf	Pl =	56.4 ksf	Pl* =	52.4 ksf
Eo =	1225 ksf	Er =	0 ksf	Eo/Pl* =	23.4



LAWTON 20 FOOT PMT

POINT NUMBER	VOLUME MEASUREMENT	PRESSURE MEASUREMENT	CORR. VOL. INCREASE (cm ³)	dR/Ro (%)	CORRECTED PRESSURE (ksf)
1	210.000	1.0	0.00	0.00	3.59
2	222.000	2.0	9.60	0.61	5.60
3	230.000	3.0	15.20	0.96	7.64
4	236.000	4.0	18.80	1.18	9.69
5	247.000	6.0	25.00	1.57	13.80
6	257.000	8.0	33.80	2.12	17.91
7	273.000	10.0	48.60	3.03	21.99
8	293.000	12.0	67.80	4.21	26.04
9	315.000	14.0	89.40	5.51	30.08
10	355.000	16.0	129.00	7.87	34.01
11	431.000	18.0	204.60	12.22	37.72
12	483.000	19.0	256.40	15.11	39.51
13	587.000	20.0	360.20	20.69	41.11
Po = 5.0 ksf Pl = 47.5 ksf Pl* = 42.5 ksf Eo = 1352 ksf Er = 0 ksf Eo/Pl* = 31.8					



APPENDIX B

MFAD INPUT/OUTPUT REPORT

PROGRAM: MFAD - MOMENT FOUNDATION ANALYSIS AND DESIGN

VERSION 2.85 05/15/87

LOCATION: Interstate Highway 240 & May Avenue
BY: MLH DATE: 03/03/91 STR. NO.: May2
CHKD. BY: DATE: SHEET NO.: 1 OF

INPUT DATA

RUN OPTIONS: NONLINEAR LOAD-DEFLECTION ANALYSIS
WITH: SIDE SHEAR MOMENT SPRING
BASE SHEAR SPRING
BASE MOMENT SPRING
BRITISH UNITS
ALLOWABLE ERROR FOR CONVERGENCE = 0.01000
EMBEDMENT TYPE: DRILLED

SOIL PARAMETERS: 8 LAYERS WITH A DEPTH TO WATER TABLE OF 29.5 FT.

LAYER NUMBER	DEPTH TO BOTTOM OF LAYER (FT.)	TOTAL UNIT WEIGHT (PCF)	PRESSUREMETER MODULUS OF DEFORMATION (KSI)	FRICTION ANGLE (DEG.)	COHESION (KSF)	STRENGTH REDUCTION FACTOR
1	3.0	120.0	1.8900	0.0	2.800	0.40
2	6.0	125.0	1.6900	0.0	2.700	0.40
3	9.0	125.0	1.8600	0.0	2.300	0.40
4	12.5	125.0	4.3100	0.0	4.200	0.40
5	16.0	125.0	7.8300	0.0	7.500	0.40
6	19.0	125.0	5.8000	0.0	6.000	0.40
7	22.0	125.0	10.4300	0.0	8.000	0.40
8	28.5	125.0	52.0500	0.0	13.500	0.40

PIER PARAMETERS:

DIAMETER = 4.0 FT. STICK-UP = 0.0 FT. EMBEDMENT = 18.0 FT.
MODULUS OF ELASTICITY X MOMENT OF INERTIA (EI) = 0.120E+10 K.-SQ. IN.

LOAD PARAMETERS: NUMBER OF LOAD CASES = 9

LOAD CASE NO.	LOAD CASE IDENTIFICATION	APPLIED LOADS AT TOP OF PIER		
		LATERAL (K.)	MOMENT (K.-FT.)	AXIAL (K.)
1	5900	15.7	5900.0	0.0
2	100 kft	15.7	100.0	0.0
3	200	15.7	200.0	0.0
4	400	15.7	400.0	0.0
5	600	15.7	600.0	0.0
6	1000kft	15.7	1000.0	0.0
7	3000kft	15.7	3000.0	0.0
8	5000kft	15.7	5000.0	0.0
9	6000kft	15.7	6000.0	0.0

PROGRAM: MFAD - MOMENT FOUNDATION ANALYSIS AND DESIGN

VERSION 2.85 05/15/87

LOCATION: Interstate Highway 240 & May Avenue
 BY: MLH DATE: 03/03/91 STR. NO.: May2
 CHKD. BY: DATE: SHEET NO.: 2 OF

NONLINEAR LOAD-DEFLECTION ANALYSIS RESULTS

ANALYSIS LOADING AT TOP OF PIER:

LOAD CASE NO.	LOAD CASE IDENTIFICATION	ACTUAL LOADS LATERAL (K.)	ACTUAL LOADS MOMENT (K.-FT.)	ACTUAL ECCENTRICITY (FT.)
1	5900	15.7	5900.0	375.8

ULTIMATE CAPACITY AT TOP OF PIER:

LATERAL LOAD = 17.6 K. MOMENT = 6623.2 K.-FT.

AT ANALYSIS LOADING:

ELEVATION (FT.)	DEFLECTION (IN.)	ROTATION (DEG.)	INTERNAL SHEAR (K.)	INTERNAL MOMENT (K.-FT.)	LATERAL SOIL PRESSURE (KSF)	SIDE SHEAR MOMENT (K.-FT./SF)
0.0	0.277E+01	0.128E+01	15.7	5900.0	-8.895	-1.759
-1.5	0.238E+01	0.121E+01	-38.3	5872.6	-10.138	-1.759
-3.0	0.200E+01	0.115E+01	-107.1	5753.2	-11.380	-1.759
-3.0	LAYER INTERFACE				-12.640	-1.696
-4.5	0.165E+01	0.110E+01	-183.4	5525.1	-13.253	-1.696
-6.0	0.131E+01	0.104E+01	-267.1	5177.1	-13.866	-1.696
-6.0	LAYER INTERFACE				-12.613	-1.445
-7.5	0.995E+00	0.990E+00	-343.1	4710.5	-12.707	-1.445
-9.0	0.692E+00	0.944E+00	-413.9	4132.4	-10.723	-1.445
-9.0	LAYER INTERFACE				-22.512	-2.639
-10.8	0.354E+00	0.899E+00	-549.7	3264.2	-16.249	-2.639
-12.5	0.311E-01	0.866E+00	-630.3	2201.7	-4.853	-2.639
-12.5	LAYER INTERFACE				-8.862	-4.712
-14.3	-0.282E+00	0.846E+00	-524.1	1126.3	26.867	-4.712
-16.0	-0.591E+00	0.838E+00	-290.6	368.3	39.075	-4.712
-16.0	LAYER INTERFACE				30.377	-3.770
-18.0	-0.941E+00	0.836E+00	-15.2	21.9	38.341	-3.770
BASE SHEAR =			15.1 K.			
BASE MOMENT =			-22.0 K.-FT.			

APPENDIX C

P-Y CURVE DATA

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: MAY 3
 SITE: MAY

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 3.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.47 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.41 IN.
 POH, INITIAL PRESSURE 1.50 KSF
 PL, LIMIT PRESSURE 34.70 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 0.94

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 0.95
 BORED PILE

P-Y CURVE FOR 3 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
3.205	4.888	8.094	0.170
9.727	15.899	25.626	0.410
12.996	20.521	33.516	0.530
19.533	22.509	42.042	0.750
25.901	23.613	49.514	1.120
32.163	27.694	59.858	1.590
38.405	30.154	68.558	2.150
44.447	30.239	74.686	2.890
50.221	30.239	80.460	3.900

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: MAY 6
 SITE: MAY

PMT DATA

COHESIVE SOIL		
PRE-BORED PMT		
DEPTH OF TEST	6.00	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.58	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.58	IN.
POH, INITIAL PRESSURE	1.40	KSF
PL, LIMIT PRESSURE	34.70	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 6 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
2.015	2.535	4.550	0.120
5.148	10.521	15.669	0.310
11.597	12.239	23.837	0.520
17.876	15.076	32.952	0.880
23.914	15.076	38.991	1.470

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: MAY 9
 SITE: MAY

PMT DATA

COHESIVE SOIL		
PRE-BORED PMT		
DEPTH OF TEST	9.00	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.43	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.43	IN.
POH, INITIAL PRESSURE	2.00	KSF
PL, LIMIT PRESSURE	34.70	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 9 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
0.000	0.000	0.000	0.000
6.962	9.676	16.637	0.260
10.099	12.634	22.733	0.380
13.236	14.549	27.785	0.500
19.473	16.039	35.512	0.790
25.581	14.927	40.508	1.300
31.257	16.919	48.175	2.170
36.804	21.229	58.033	3.340
42.090	21.229	63.319	4.760

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: MAY 12_5
 SITE: MAY

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 12.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.48 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.48 IN.
 POH, INITIAL PRESSURE 2.00 KSF
 PL, LIMIT PRESSURE 34.70 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 12 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
6.494	11.549	18.034	0.190
19.615	29.457	49.072	0.380
26.145	33.224	59.369	0.500
32.660	36.875	69.535	0.630
39.145	35.501	74.646	0.790
45.555	32.702	84.626	1.280
70.219	28.909	99.128	2.990

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: MAY 16
 SITE: MAY

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 16.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RAOIUS (FRONT REACTION) 1.42 IN.
 BOREHOLE RAOIUS (SHEAR REACTION) 1.42 IN.
 POH, INITIAL PRESSURE 2.50 KSF
 PL, LIMIT PRESSURE 34.70 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 16 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
0.000	0.000	0.000	0.000
13.733	29.511	43.244	0.300
20.278	50.419	70.698	0.370
33.436	74.613	108.050	0.480
46.595	86.903	133.498	0.590
59.730	89.758	149.488	0.720
98.986	82.911	181.897	1.340
124.851	71.167	196.018	2.090
156.188	68.188	224.376	4.170

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: MAY 19
 SITE: MAY

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 19.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.38 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.38 IN.
 POH, INITIAL PRESSURE 4.00 KSF
 PL, LIMIT PRESSURE 51.60 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

FILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 19 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
4.787	7.528	12.315	0.100
30.491	60.087	90.578	0.440
43.499	62.622	106.122	0.570
56.567	62.140	118.708	0.740
69.545	56.115	125.660	0.970
107.499	50.844	158.343	2.630
119.784	51.174	170.958	3.710
125.551	51.174	170.958	4.670

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: LAWTON 8
 SITE: LAWTON

PMT DATA

COHESIVE SOIL		
PREBORED PMT		
DEPTH OF TEST	8.00	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.43	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.43	IN.
POH, INITIAL PRESSURE	3.50	KSF
PL, LIMIT PRESSURE	42.13	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 8 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
3.182	4.810	7.992	0.040
9.745	17.584	27.328	0.090
29.531	27.340	56.871	0.180
42.637	35.328	77.965	0.350
55.703	40.985	96.688	0.540
68.730	37.705	106.435	0.760
93.959	29.939	123.898	1.770
105.507	29.939	135.446	3.090

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: LAWTON 12
 SITE: LAWTON

PMT DATA

COHESIVE SOIL		
PREBORED PMT		
DEPTH OF TEST	12.00	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.39	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.39	IN.
POH, INITIAL PRESSURE	3.80	KSF
PL, LIMIT PRESSURE	42.13	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 12 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
0.000	0.000	0.000	0.000
4.082	6.715	10.797	0.130
17.030	25.694	42.724	0.370
23.495	30.987	54.482	0.500
29.939	32.873	62.812	0.640
42.809	34.554	77.363	0.990
55.306	30.529	85.835	1.610
67.056	30.178	97.234	2.730
72.226	30.178	102.404	3.860

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: LAWTON 16
SITE: LAWTON

PMT DATA

COHESIVE SOIL		
PREBORED PMT		
DEPTH OF TEST	16.00	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.38	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.38	IN.
POH, INITIAL PRESSURE	4.00	KSF
PL, LIMIT PRESSURE	42.13	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

FILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 16 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
0.000	0.000	0.000	0.000
10.842	5.284	16.126	0.060
17.346	19.037	36.383	0.160
52.217	57.451	109.668	0.440
69.966	66.745	136.711	0.660
116.564	42.554	159.118	1.800
127.144	42.621	169.765	2.740
138.576	40.161	178.737	4.220

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: LAWTON 20
 SITE: LAWTON

PMT DATA

COHESIVE SOIL		
PREBORED PMT		
DEPTH OF TEST	20.00	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.38	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.38	IN.
POH, INITIAL PRESSURE	5.00	KSF
PL, LIMIT PRESSURE	47.50	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 20 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
0.000	0.000	0.000	0.000
8.441	15.206	23.648	0.130
28.149	40.959	69.108	0.270
54.360	42.145	96.505	0.620
67.328	52.166	119.494	0.900
92.833	37.410	130.242	1.780
110.429	28.559	138.989	3.510
115.539	28.559	144.099	4.840

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: I44 2_5
 SITE: I44

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 2.50 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.51 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.51 IN.
 POH, INITIAL PRESSURE 0.80 KSF
 PL, LIMIT PRESSURE 63.26 KSF
 DEPTH REDUCTION FACTOR FOR PROBE .86

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 0.85
 BORED PILE

P-Y CURVE FOR 2 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
0.873	1.210	2.083	0.230
2.656	5.458	8.114	0.700
5.196	12.883	18.079	1.030
8.027	22.097	30.123	1.330
13.969	28.248	42.217	1.810
19.560	28.236	47.797	2.460
24.685	28.236	52.921	3.300

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: I44 7_5
 SITE: I44

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 7.50 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.58 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.58 IN.
 POH, INITIAL PRESSURE 2.00 KSF
 PL, LIMIT PRESSURE 63.26 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 7 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
4.692	5.963	10.655	0.290
16.793	37.485	54.278	0.850
23.080	49.152	72.232	1.020
29.367	54.759	84.125	1.200
35.604	57.463	93.067	1.390
41.793	60.691	102.484	1.610
47.932	60.691	108.624	1.840

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: I44 10
 SITE: I44

PMT DATA

COHESIVE SOIL		
PREBORED PMT		
DEPTH OF TEST	10.00	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.46	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.46	IN.
POH, INITIAL PRESSURE	3.00	KSF
PL, LIMIT PRESSURE	63.26	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 10 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
0.000	0.000	0.000	0.000
5.997	7.829	13.826	0.800
10.728	14.196	24.925	1.450
15.589	26.166	41.755	2.140
21.166	52.625	73.791	2.670
39.141	77.066	116.208	3.650
44.690	77.066	121.756	4.200

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: 144 13
 SITE: I44

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 13.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.45 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.45 IN.
 POH, INITIAL PRESSURE 3.50 KSF
 PL, LIMIT PRESSURE 63.26 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 13 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
5.709	8.111	13.820	0.530
11.491	21.243	32.734	0.980
17.363	33.710	51.072	1.250
23.002	44.984	67.986	1.550
34.918	44.984	79.902	2.080

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: I44 16_5
 SITE: I44

PMT DATA

COHESIVE SOIL		
PREBORED PMT		
DEPTH OF TEST	16.50	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.43	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.42	IN.
POH, INITIAL PRESSURE	7.50	KSF
PL, LIMIT PRESSURE	63.26	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 16.5 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
16.041	28.470	44.511	0.140
39.877	93.796	133.673	0.220
75.544	176.080	251.624	0.310
174.840	191.751	366.591	0.540
207.835	182.128	389.963	0.700
275.987	163.543	439.530	1.110
338.683	180.556	519.240	1.800
402.145	169.366	571.511	3.200
433.580	169.804	603.384	4.190

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: I44 24_8
 SITE: I44

PMT DATA

COHESIVE SOIL		
PREBORED PMT		
DEPTH OF TEST	24.80	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.44	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.44	IN.
POH, INITIAL PRESSURE	10.00	KSF
PL, LIMIT PRESSURE	118.00	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.02	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.02	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 24.8 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
2.644	9.267	11.911	0.040
35.508	77.659	113.166	0.190
68.662	189.231	177.804	0.250
200.515	39.198	239.714	0.960
229.729	39.198	268.927	2.530

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: GAI 2.5
 SITE: GAI

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 2.50 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.38 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.38 IN.
 POH, INITIAL PRESSURE 1.80 KSF
 PL, LIMIT PRESSURE 42.50 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 0.86

PILE DATA

PILE DIAMETER 60.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 30.68 FT⁴

 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 0.85
 BORED PILE

P-Y CURVE FOR 2.5 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
5.861	10.689	16.550	0.440
11.542	22.021	33.562	0.720
22.904	30.275	53.179	1.270
28.585	34.997	63.582	1.740
34.265	36.608	70.874	2.180
39.946	36.447	76.394	2.840
45.627	37.791	83.419	3.670
51.308	37.791	89.100	4.770

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: GAI 7.5
 SITE: GAI

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 7.50 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.37 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.37 IN.
 POH, INITIAL PRESSURE 2.80 KSF
 PL, LIMIT PRESSURE 42.50 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 60.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 30.68 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 7.5 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
0.000	0.000	0.000	0.000
3.190	4.015	7.205	0.180
20.470	36.924	57.394	0.740
37.750	35.005	72.756	1.230
43.510	34.042	77.553	1.570
49.270	33.790	83.061	1.940
66.550	25.184	91.734	3.870
72.310	25.184	97.494	5.690

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: GAI 11.4
 SITE: GAI

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 11.40 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.38 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.38 IN.
 POH, INITIAL PRESSURE 4.50 KSF
 PL, LIMIT PRESSURE 42.50 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 60.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 30.68 FT⁴
 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 2.5 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
29.044	36.545	65.589	0.110
57.844	70.002	127.846	0.240
115.444	85.354	200.798	0.540
129.844	88.811	218.655	0.660
201.844	90.957	292.801	1.710
245.044	114.259	359.303	2.960
259.444	116.535	375.978	3.570
273.844	116.535	390.378	4.270

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: Bdwy 3
 SITE: Bdwy

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 3.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.46 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.46 IN.
 POH, INITIAL PRESSURE 1.00 KSF
 PL, LIMIT PRESSURE 78.40 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 0.94

FILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴

 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 0.95
 BORED PILE

P-Y CURVE FOR 3.0 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
3.556	5.354	8.910	0.300
6.765	10.610	17.375	0.510
9.996	16.454	26.451	0.720
13.264	17.300	30.564	0.900
16.453	15.158	33.611	1.180
19.602	16.548	36.150	1.510
28.330	16.043	44.373	3.420
33.932	18.271	52.203	5.710

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: Bdwy 10
 SITE: Bdwy

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 10.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.50 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.50 IN.
 POH, INITIAL PRESSURE 2.00 KSF
 PL, LIMIT PRESSURE 78.40 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴

 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 10.0 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
7.8363	12.693	20.556	0.220
14.456	24.450	38.907	0.330
21.040	35.749	56.789	0.450
73.989	68.190	142.180	1.030
126.104	53.163	179.267	2.680
138.492	53.163	191.655	3.910

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: Bdwy 12
 SITE: Bdwy

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST 12.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.56 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.56 IN.
 POH, INITIAL PRESSURE 5.00 KSF
 PL, LIMIT PRESSURE 78.40 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴

 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 10.0 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
11.3633	14.258	25.622	0.110
37.853	31.616	69.469	0.360
64.119	44.431	108.549	0.880
77.061	36.938	119.814	1.390
89.770	39.847	129.618	2.190
102.435	39.847	142.282	3.370

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: Bdwy 14
 SITE: Bdwy

PMT DATA

COHESIVE SOIL		
PREBORED PMT		
DEPTH OF TEST	12.00	FT
DEFLATED PROBE RADIUS	1.38	IN.
BOREHOLE RADIUS (FRONT REACTION)	1.45	IN.
BOREHOLE RADIUS (SHEAR REACTION)	1.45	IN.
POH, INITIAL PRESSURE	2.20	KSF
PL, LIMIT PRESSURE	78.40	KSF
DEPTH REDUCTION FACTOR FOR PROBE	1.00	

PILE DATA

PILE DIAMETER	48.00	IN.
PILE MODULUS	519120.00	KSF
PILE MOMENT OF INERTIA	12.57	FT ⁴
PILE FRONT REACTION SHAPE FACTOR	0.80	
PILE SHEAR REACTION SHAPE FACTOR	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE	1.00	
PILE REDUCTION FACTOR FOR MOBILIZED SHEAR	1.00	
PILE DEPTH REDUCTION FACTOR	1.00	
BORED PILE		

P-Y CURVE FOR 10.0 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
<hr/>			
0.000	0.000	0.000	0.000
8.826	11.135	19.961	0.120
15.422	19.602	35.024	0.210
22.018	33.474	55.492	0.290
35.246	73.276	108.522	0.430
88.364	87.681	176.045	0.740
153.830	53.392	207.221	2.490

P-Y CURVE GENERATION FROM PMT TEST

TEST TITLE: Bdwy 17
 SITE: Bdwy

PMT DATA

COHESIVE SOIL
 PREBORED PMT
 DEPTH OF TEST (Note: Data used as 19.0) 19.00 FT
 DEFLATED PROBE RADIUS 1.38 IN.
 BOREHOLE RADIUS (FRONT REACTION) 1.43 IN.
 BOREHOLE RADIUS (SHEAR REACTION) 1.43 IN.
 POH, INITIAL PRESSURE 3.00 KSF
 PL, LIMIT PRESSURE 78.40 KSF
 DEPTH REDUCTION FACTOR FOR PROBE 1.00

PILE DATA

PILE DIAMETER 48.00 IN.
 PILE MODULUS 519120.00 KSF
 PILE MOMENT OF INERTIA 12.57 FT⁴

 PILE FRONT REACTION SHAPE FACTOR 0.80
 PILE SHEAR REACTION SHAPE FACTOR 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED PRESSURE 1.00
 PILE REDUCTION FACTOR FOR MOBILIZED SHEAR 1.00
 PILE DEPTH REDUCTION FACTOR 1.00
 BORED PILE

P-Y CURVE FOR 19.0 FEET

FRONT REACTION (k/f)	SIDE REACTION (k/f)	TOTAL REACTION (k/f)	PILE DISPLACEMENT (in.)
0.000	0.000	0.000	0.000
60.370	459.350	519.721	0.300
160.410	658.989	819.399	0.430
193.788	681.244	875.033	0.440
227.143	681.244	908.387	0.480

APPENDIX D

BMCOL76 INPUT/OUTPUT REPORT

MAY AVENUE & INTERSTATE HIGHWAY 240
48 INCH DRILLED SHAFT, 18 FOOT LONG

PROB
1 48 INCH DRILLED SHAFT, 18 FOOT LONG, 100K-FT MOMENT

TABLE 1 - PROGRAM CONTROL DATA

PROBLEM TYPE (1=AX,2=LAT,3=COMB)	2
NUM LATERAL INCREMENTS	72
LATERAL INCREMENT LENGTH	3.000D+00
DATA CARD LISTING (1=NO)	0

TABLE 6 - LATERAL CONTROL DATA

	DEFLS W(I)	TABLE NUMBER		
		7	8	9
PRIOR-DATA OPTIONS (1 = HOLD)	0	0	0	0
NUM CARDS INPUT THIS PROBLEM	0	2	21	
OUTPUT OPTION (0 = TABLE 15 ONLY, 1 = TABLES 15 AND 16)				0
PLOT OPTION (1=PRINTER, 2=CALCOMP, 3=BOTH)				1

LATERAL ITERATION CONTROL DATA

MAX NUMBER OF ITERATIONS				10
DEFL CLOSURE TOLERANCE				1.000D-03
MAX ALLOWABLE DEFLECTION				1.200D+01
LIST OF MONITOR STATIONS	1	12	16	24 72

TABLE 7 - SPECIFIED DEFLECTIONS AND SLOPES

STA	CASE	DEFLECTION	SLOPE
NONE			

TABLE 8 - LATERAL STIFFNESS AND LOAD DATA

FROM	TO	CONTD	EI	Q	S	C	R
0	0	0	0.000D-01	1.570D+04	0.000D-01	1.200D+06	0.000D-01
0	72	0	1.200D+12	0.000D-01	0.000D-01	0.000D-01	0.000D-01

TABLE 9 - LATERAL LOAD AND SUPPORT CURVES

MAY AVENUE & INTERSTATE HIGHWAY 240
48 INCH DRILLED SHAFT, 18 FOOT LONG

PROB (CONTD)

1 48 INCH DRILLED SHAFT, 18 FOOT LONG, 100K-FT MOMENT

TABLE 14 - LATERAL ITERATION MONITOR DATA

ITER NUM	OFF CURVES	NUM STAS NOT CLOSED	DEFLECTIONS AT STATION NUMBERS				
			1	12	16	24	72
1	NO	70	4.939D-02	3.654D-02	3.220D-02	2.409D-02	-1.309D-02
2	NO	26	4.824D-02	3.546D-02	3.114D-02	2.308D-02	-1.334D-02
3	NO	0	4.824D-02	3.546D-02	3.114D-02	2.308D-02	-1.334D-02

MAY AVENUE & INTERSTATE HIGHWAY 240
48 INCH DRILLED SHAFT, 18 FOOT LONG

PROB (CONTD)

1 48 INCH DRILLED SHAFT, 18 FOOT LONG, 100K-FT MOMENT

TABLE 15 - RESULTS OF ITERATION NUM 3

ASTERISKS * INDICATE VALUES AFFECTED BY SPECIFIED SLOPES, DEFLECTIONS,
APPLIED COUPLES, OR ROTATIONAL RESTRAINTS

STA I	DIST ALONG BMCOL	DEFL	SLOPE	BENDING MOMENT	SHEAR	SUPPORT REACTION
-1	-3.000D+00	5.070D-02		0.000D-01		0.000D-01
0	0.000D-01	4.947D-02	-4.109D-04	6.000D+05*	2.000D+05*	-5.844D+02
1	3.000D+00	4.824D-02	-4.079D-04	1.245D+06	2.151D+05*	-1.140D+03
2	6.000D+00	4.703D-02	-4.048D-04	1.287D+06	1.398D+04	-1.111D+03
3	9.000D+00	4.582D-02	-4.016D-04	1.326D+06	1.286D+04	-1.083D+03
4	1.200D+01	4.463D-02	-3.983D-04	1.361D+06	1.178D+04	-1.055D+03
5	1.500D+01	4.344D-02	-3.949D-04	1.393D+06	1.073D+04	-1.027D+03
6	1.800D+01	4.227D-02	-3.914D-04	1.422D+06	9.701D+03	-9.988D+02
7	2.100D+01	4.111D-02	-3.878D-04	1.449D+06	8.702D+03	-9.713D+02
8	2.400D+01	3.995D-02	-3.842D-04	1.472D+06	7.730D+03	-9.441D+02
9	2.700D+01	3.881D-02	-3.805D-04	1.492D+06	6.786D+03	-9.172D+02
10	3.000D+01	3.768D-02	-3.768D-04	1.510D+06	5.869D+03	-8.905D+02
11	3.300D+01	3.656D-02	-3.730D-04	1.525D+06	4.979D+03	-8.640D+02
12	3.600D+01	3.546D-02	-3.692D-04	1.537D+06	4.115D+03	-7.154D+02
13	3.900D+01	3.436D-02	-3.654D-04	1.547D+06	3.399D+03	-5.745D+02
14	4.200D+01	3.327D-02	-3.615D-04	1.556D+06	2.825D+03	-5.564D+02
15	4.500D+01	3.220D-02	-3.576D-04	1.563D+06	2.268D+03	-5.385D+02
16	4.800D+01	3.114D-02	-3.537D-04	1.568D+06	1.730D+03	-5.207D+02
17	5.100D+01	3.009D-02	-3.498D-04	1.571D+06	1.209D+03	-5.032D+02
18	5.400D+01	2.905D-02	-3.459D-04	1.573D+06	7.060D+02	-4.858D+02
19	5.700D+01	2.803D-02	-3.419D-04	1.574D+06	2.202D+02	-4.687D+02
20	6.000D+01	2.701D-02	-3.380D-04	1.573D+06	-2.485D+02	-4.517D+02
21	6.300D+01	2.601D-02	-3.341D-04	1.573D+06	-7.002D+02	-4.350D+02
			-3.301D-04	1.571D+06	-1.135D+03	

22	6.600D+01	2.502D-02	-3.262D-04	1.568D+06	-1.554D+03	-4.184D+02
23	6.900D+01	2.404D-02	-3.223D-04	1.563D+06	-1.956D+03	-4.020D+02
24	7.200D+01	2.308D-02	-3.184D-04	1.557D+06	-2.520D+03	-5.648D+02
25	7.500D+01	2.212D-02	-3.145D-04	1.550D+06	-3.233D+03	-7.130D+02
26	7.800D+01	2.118D-02	-3.107D-04	1.540D+06	-3.916D+03	-6.826D+02
27	8.100D+01	2.025D-02	-3.069D-04	1.528D+06	-4.569D+03	-6.526D+02
28	8.400D+01	1.932D-02	-3.031D-04	1.515D+06	-5.191D+03	-6.229D+02
29	8.700D+01	1.842D-02	-2.993D-04	1.499D+06	-5.785D+03	-5.936D+02
30	9.000D+01	1.752D-02	-2.956D-04	1.482D+06	-6.350D+03	-5.646D+02
31	9.300D+01	1.663D-02	-2.920D-04	1.463D+06	-6.886D+03	-5.360D+02
32	9.600D+01	1.575D-02	-2.884D-04	1.442D+06	-7.393D+03	-5.078D+02
33	9.900D+01	1.489D-02	-2.848D-04	1.420D+06	-7.873D+03	-4.799D+02
34	1.020D+02	1.403D-02	-2.813D-04	1.396D+06	-8.326D+03	-4.524D+02
35	1.050D+02	1.319D-02	-2.779D-04	1.371D+06	-8.751D+03	-4.252D+02
36	1.080D+02	1.236D-02	-2.745D-04	1.345D+06	-9.458D+03	-7.069D+02
37	1.110D+02	1.153D-02	-2.712D-04	1.317D+06	-1.040D+04	-9.453D+02
38	1.140D+02	1.072D-02	-2.680D-04	1.285D+06	-1.128D+04	-8.760D+02
39	1.170D+02	9.916D-03	-2.649D-04	1.252D+06	-1.209D+04	-8.075D+02
40	1.200D+02	9.121D-03	-2.619D-04	1.215D+06	-1.283D+04	-7.398D+02
41	1.230D+02	8.335D-03	-2.589D-04	1.177D+06	-1.350D+04	-6.729D+02
42	1.260D+02	7.559D-03	-2.561D-04	1.136D+06	-1.411D+04	-6.067D+02
43	1.290D+02	6.790D-03	-2.533D-04	1.094D+06	-1.465D+04	-5.412D+02
44	1.320D+02	6.030D-03	-2.507D-04	1.050D+06	-1.512D+04	-4.765D+02
45	1.350D+02	5.278D-03	-2.482D-04	1.005D+06	-1.554D+04	-4.124D+02
46	1.380D+02	4.534D-03	-2.458D-04	9.581D+05	-1.589D+04	-3.490D+02
47	1.410D+02	3.796D-03	-2.435D-04	9.104D+05	-1.617D+04	-2.862D+02
48	1.440D+02	3.066D-03	-2.414D-04	8.619D+05	-1.640D+04	-2.239D+02
49	1.470D+02	2.341D-03	-2.394D-04	8.127D+05	-1.656D+04	-1.623D+02
50	1.500D+02	1.623D-03	-2.374D-04	7.630D+05	-1.672D+04	-1.653D+02
51	1.530D+02	9.110D-04	-2.357D-04	7.129D+05	-1.685D+04	-1.288D+02
52	1.560D+02	2.040D-04	-2.340D-04	6.623D+05	-1.688D+04	-2.884D+01
53	1.590D+02	-4.981D-04	-2.325D-04	6.117D+05	-1.681D+04	7.043D+01
54	1.620D+02	-1.195D-03	-2.311D-04	5.612D+05	-1.664D+04	1.691D+02

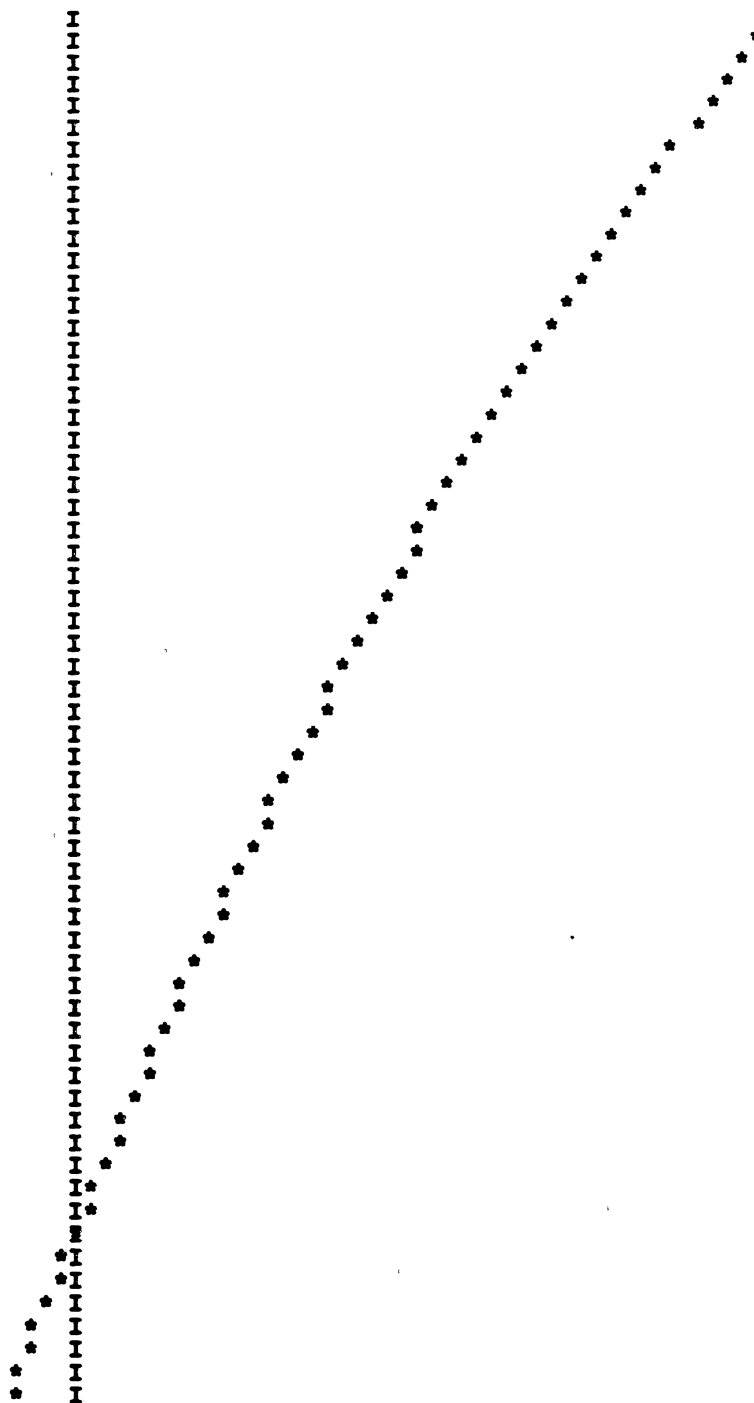
55	1.650D+02	-1.889D-03		5.113D+05		2.671D+02
56	1.680D+02	-2.578D-03	-2.298D-04	4.622D+05	-1.637D+04	3.646D+02
57	1.710D+02	-3.264D-03	-2.286D-04	4.142D+05	-1.601D+04	4.616D+02
58	1.740D+02	-3.947D-03	-2.276D-04	3.675D+05	-1.555D+04	5.582D+02
59	1.770D+02	-4.627D-03	-2.267D-04	3.226D+05	-1.499D+04	6.543D+02
60	1.800D+02	-5.305D-03	-2.259D-04	2.796D+05	-1.434D+04	7.502D+02
61	1.830D+02	-5.980D-03	-2.252D-04	2.388D+05	-1.358D+04	8.457D+02
62	1.860D+02	-6.654D-03	-2.246D-04	2.006D+05	-1.274D+04	9.410D+02
63	1.890D+02	-7.326D-03	-2.241D-04	1.652D+05	-1.180D+04	1.036D+03
64	1.920D+02	-7.997D-03	-2.237D-04	1.329D+05	-1.076D+04	1.043D+03
65	1.950D+02	-8.667D-03	-2.233D-04	1.037D+05	-9.719D+03	1.035D+03
66	1.980D+02	-9.336D-03	-2.231D-04	7.769D+04	-8.684D+03	1.115D+03
67	2.010D+02	-1.001D-02	-2.229D-04	5.498D+04	-7.569D+03	1.195D+03
68	2.040D+02	-1.067D-02	-2.227D-04	3.586D+04	-6.374D+03	1.275D+03
69	2.070D+02	-1.134D-02	-2.227D-04	2.056D+04	-5.100D+03	1.354D+03
70	2.100D+02	-1.201D-02	-2.226D-04	9.323D+03	-3.745D+03	1.434D+03
71	2.130D+02	-1.268D-02	-2.226D-04	2.391D+03	-2.311D+03	1.514D+03
72	2.160D+02	-1.334D-02	-2.226D-04	0.000D-01	-7.969D+02	7.969D+02
73	2.190D+02	-1.401D-02	-2.226D-04	0.000D-01	0.000D-01	0.000D-01
STA I	DIST ALONG BMCOL	DEFL	SLOPE	BENDING MOMENT	SHEAR	SUPPORT REACTION

THE MAXIMUM ARITHMETIC ROUND-OFF ERROR CHECK WAS 7.704D-07 FORCE UNITS

LATERAL DEFLECTION ALONG BNCOL

STA
NUM LATERAL
DEFLECTION

-1	5.070E-02
0	4.947E-02
1	4.824E-02
2	4.703E-02
3	4.582E-02
4	4.463E-02
5	4.344E-02
6	4.227E-02
7	4.111E-02
8	3.995E-02
9	3.881E-02
10	3.768E-02
11	3.656E-02
12	3.546E-02
13	3.436E-02
14	3.327E-02
15	3.220E-02
16	3.114E-02
17	3.009E-02
18	2.905E-02
19	2.803E-02
20	2.701E-02
21	2.601E-02
22	2.502E-02
23	2.404E-02
24	2.308E-02
25	2.212E-02
26	2.118E-02
27	2.025E-02
28	1.932E-02
29	1.842E-02
30	1.752E-02
31	1.663E-02
32	1.575E-02
33	1.489E-02
34	1.403E-02
35	1.319E-02
36	1.236E-02
37	1.153E-02
38	1.072E-02
39	9.916E-03
40	9.121E-03
41	8.335E-03
42	7.559E-03
43	6.790E-03
44	6.030E-03
45	5.278E-03
46	4.534E-03
47	3.796E-03
48	3.066E-03
49	2.341E-03
50	1.623E-03
51	9.110E-04
52	2.040E-04
53	-4.981E-04
54	-1.195E-03
55	-1.889E-03
56	-2.578E-03
57	-3.264E-03
58	-3.947E-03
59	-4.627E-03



60	-5.305E-03		*	I
61	-5.980E-03		*	I
62	-6.654E-03		*	I
63	-7.326E-03		*	I
64	-7.997E-03		*	I
65	-8.667E-03		*	I
66	-9.336E-03		*	I
67	-1.001E-02		*	I
68	-1.067E-02		*	I
69	-1.134E-02	*		I
70	-1.201E-02	*		I
71	-1.268E-02	*		I
72	-1.334E-02	*		I
73	-1.401E-02	*		I

60	7.502E+02
61	8.457E+02
62	9.410E+02
63	1.036E+03
64	1.043E+03
65	1.035E+03
66	1.115E+03
67	1.195E+03
68	1.275E+03
69	1.354E+03
70	1.434E+03
71	1.514E+03
72	7.969E+02
73	0.000E-01

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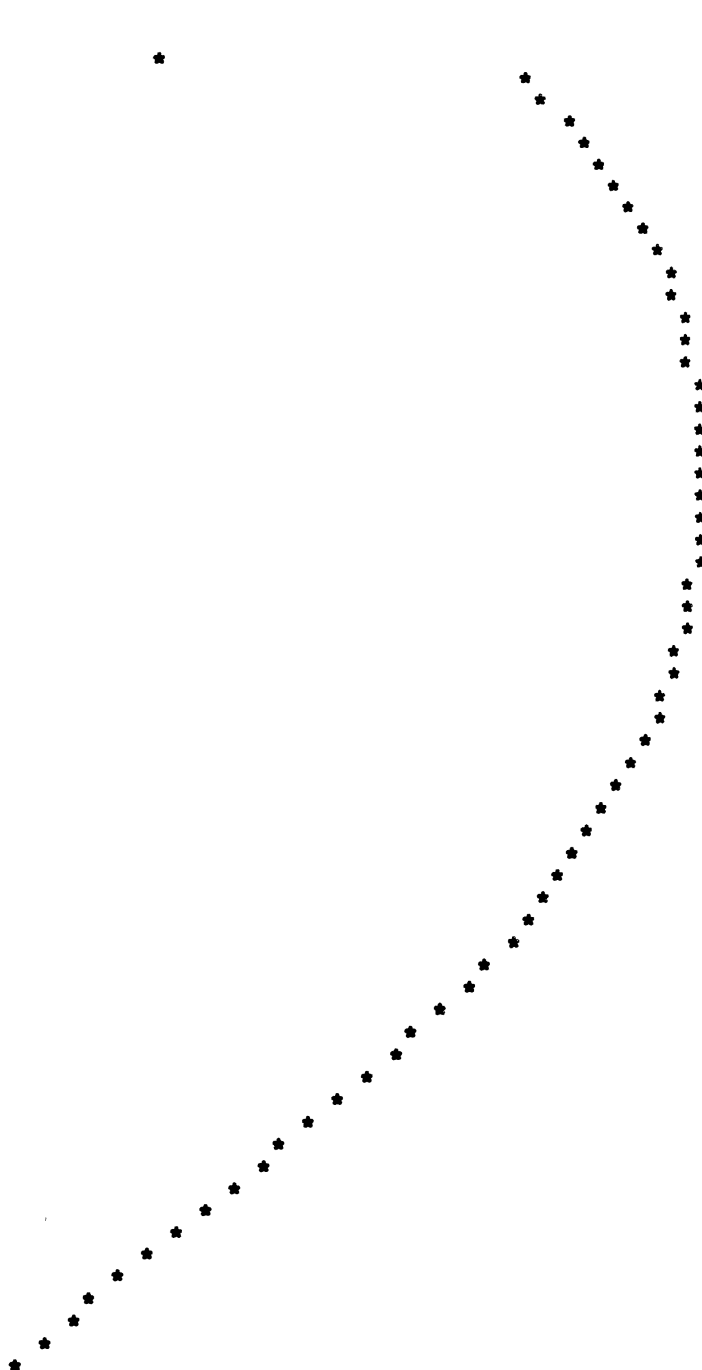
SHEAR ALONG BMCOL

BAR NUM	SHEAR FORCE	
-1	0.000E-01	I
0	2.000E+05	I
1	2.151E+05	I
2	1.398E+04	I *
3	1.286E+04	I *
4	1.178E+04	I *
5	1.073E+04	I *
6	9.701E+03	I *
7	8.702E+03	I *
8	7.730E+03	I *
9	6.786E+03	I *
10	5.869E+03	I *
11	4.979E+03	I *
12	4.115E+03	I*
13	3.399E+03	I*
14	2.825E+03	I*
15	2.268E+03	I*
16	1.730E+03	I*
17	1.209E+03	I*
18	7.060E+02	I
19	2.202E+02	I
20	-2.485E+02	I
21	-7.002E+02	I
22	-1.135E+03	I
23	-1.554E+03	I
24	-1.956E+03	I
25	-2.520E+03	I
26	-3.233E+03	*I
27	-3.916E+03	*I
28	-4.569E+03	*I
29	-5.191E+03	*I
30	-5.785E+03	*I
31	-6.350E+03	*I
32	-6.886E+03	*I
33	-7.393E+03	* I
34	-7.873E+03	* I
35	-8.326E+03	* I
36	-8.751E+03	* I
37	-9.458E+03	* I
38	-1.040E+04	* I
39	-1.128E+04	* I
40	-1.209E+04	* I
41	-1.283E+04	* I
42	-1.350E+04	* I
43	-1.411E+04	* I
44	-1.465E+04	* I
45	-1.512E+04	* I
46	-1.554E+04	* I
47	-1.589E+04	* I
48	-1.617E+04	* I
49	-1.640E+04	* I
50	-1.656E+04	* I
51	-1.672E+04	* I
52	-1.685E+04	* I
53	-1.688E+04	* I
54	-1.681E+04	* I
55	-1.664E+04	* I
56	-1.637E+04	* I
57	-1.601E+04	* I
58	-1.555E+04	* I
59	-1.499E+04	* I

60	-1.434E+04	*	I
61	-1.358E+04	*	I
62	-1.274E+04	*	I
63	-1.180E+04	*	I
64	-1.076E+04	*	I
65	-9.719E+03	*	I
66	-8.684E+03	*	I
67	-7.569E+03	*	I
68	-6.374E+03	*	I
69	-5.100E+03	*	I
70	-3.745E+03	*	I
71	-2.311E+03	*	I
72	-7.969E+02	*	I
73	0.000E-01	*	I

BENDING MOMENT ALONG BMCOL

STA NUM	BENDING MOMENT	I
-1	0.000E-01	I
0	6.000E+05	I
1	1.245E+06	I
2	1.287E+06	I
3	1.326E+06	I
4	1.361E+06	I
5	1.393E+06	I
6	1.422E+06	I
7	1.449E+06	I
8	1.472E+06	I
9	1.492E+06	I
10	1.510E+06	I
11	1.525E+06	I
12	1.537E+06	I
13	1.547E+06	I
14	1.556E+06	I
15	1.563E+06	I
16	1.568E+06	I
17	1.571E+06	I
18	1.573E+06	I
19	1.574E+06	I
20	1.573E+06	I
21	1.571E+06	I
22	1.568E+06	I
23	1.563E+06	I
24	1.557E+06	I
25	1.550E+06	I
26	1.540E+06	I
27	1.528E+06	I
28	1.515E+06	I
29	1.499E+06	I
30	1.482E+06	I
31	1.463E+06	I
32	1.442E+06	I
33	1.420E+06	I
34	1.396E+06	I
35	1.371E+06	I
36	1.345E+06	I
37	1.317E+06	I
38	1.285E+06	I
39	1.252E+06	I
40	1.215E+06	I
41	1.177E+06	I
42	1.136E+06	I
43	1.094E+06	I
44	1.050E+06	I
45	1.005E+06	I
46	9.581E+05	I
47	9.104E+05	I
48	8.619E+05	I
49	8.127E+05	I
50	7.630E+05	I
51	7.129E+05	I
52	6.623E+05	I
53	6.117E+05	I
54	5.612E+05	I
55	5.113E+05	I
56	4.622E+05	I
57	4.142E+05	I
58	3.675E+05	I
59	3.226E+05	I



60	2.796E+05	I					*
61	2.388E+05	I					*
62	2.006E+05	I				*	
63	1.652E+05	I			*		
64	1.329E+05	I		*			
65	1.037E+05	I	*				
66	7.769E+04	I	*				
67	5.498E+04	I	*				
68	3.586E+04	I*					
69	2.056E+04	I*					
70	9.323E+03	I					
71	2.391E+03	I					
72	0.000E-01	I					
73	0.000E-01	*					

APPENDIX E

STANDARD TEST METHOD FOR PRESSURE- METER TESTING IN SOILS



Designation: D 4719 - 87

Standard Test Method for Pressuremeter Testing in Soils¹

This standard is issued under the fixed designation D 4719, the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ϵ) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This test method covers pressuremeter testing of soils. A pressuremeter test is an in-situ stress-strain test performed on the wall of a borehole using a cylindrical probe that is expanded radially. To obtain viable test results, disturbance to the borehole wall must be minimized.

1.2 This test method includes the procedure for drilling the borehole, inserting the probe, and running pressuremeter tests in both granular and cohesive soils, but does not include high pressure testing in rock. Knowledge of the type of soil in which each pressuremeter test is to be made is necessary for assessment of (1) the method of boring or probe placement, or both, and (2) the reasonableness of results and interpretation of the test.

1.3 This test method does not cover the self-boring pressuremeter, for which the hole is drilled by a mechanical tool inside the hollow core of the probe. This test method is limited to the pressuremeter which is inserted into predrilled boreholes or, under certain circumstances, is inserted by driving.

1.4 Two alternate testing procedures are provided as follows.

1.4.1 *Procedure A*—The Equal Pressure Increment Method

1.4.2 *Procedure B*—The Equal Volume Increment Method

NOTE 1—A standard for the self-boring pressuremeter is scheduled to be developed separately. Pressuremeter testing in rock may be standardized as an adjunct to this test method.

1.5 The values stated in SI units are to be regarded as the standard.

1.6 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use. See Note 5.*

2. Referenced Documents

2.1 *ASTM Standards*

D 1587 Practice for Thin-Walled Tube Sampling of Soils²

D 2113 Practice for Diamond Core Drilling for Site Investigation²

3. Summary of Test Method

3.1 A pressuremeter cavity is prepared either by drilling a

borehole, or by advancing some type of sampler. Under certain circumstances, the pressuremeter probe is driven into place, usually within a casing. The various tools and methods available to prepare the cavity produce different degrees of disturbance. The recommended methods to be used at a site depend on the soil and the conditions met. The proper choice of tools and methods is covered by this test method.

NOTE 2—It is recommended that several drilling techniques be available on the site to determine which method will provide the most suitable test hole.

3.2 The pressuremeter test basically consists of placing an inflatable cylindrical probe in a predrilled hole and expanding this probe while measuring the changes in volume and pressure in the probe. The probe is inflated under equal pressure increments (Procedure A) or equal volume increments (Procedure B) and the test is terminated when yielding in the soil becomes disproportionately large. A limit pressure is estimated from the last few readings of the test and a pressuremeter modulus is calculated from pressure-volume changes read during the test. It is of basic importance that the probe be inserted in a borehole with a diameter close to that of the probe to ensure adequate volume change capability. If this requirement is not met, the test could terminate without reaching sufficient probe expansion in the soil to permit evaluation of the limit pressure. The instrument may be either of the type where the change in volume of the probe is directly measured by an incompressible liquid or the type where feelers are used to determine the change in diameter in the probe. The volume measuring system must be well protected and calibrated against any volume losses throughout the system while the feeler operated probe must be sensitive enough to measure relatively small displacement.

NOTE 3—This test method is based on the type of apparatus where volume changes are recorded during the test. For the system measuring probe diameters, alternate evaluation methods are given in the notes.

4. Significance and Use

4.1 This test method provides a stress-strain response of the soil in-situ. A pressuremeter modulus and a limit pressure is obtained for use in geotechnical analysis and foundation design.

4.2 The results of this test method are dependent on the degree of disturbance during drilling of the borehole and insertion of the pressuremeter probe. Since disturbance cannot be completely eliminated, the interpretation of the test results should include consideration of conditions during drilling. This disturbance is particularly significant in very

¹ This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.02 on Sampling and Related Field Testing for Soil Investigations.

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² *Annual Book of ASTM Standards*, Vol. 04.08.



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TABLE 1 Typical Probe and Borehole Dimensions

Hole Diameter Designation	Probe Diameter, mm	Borehole Diameter	
		Nominal, mm	Max., mm
Ax	44	45	53
Bx	58	60	70
Nx	74	76	89

soft clays and very loose sands

5. Apparatus

5.1 *Hydraulic or Electric Probe*—The apparatus shall consist of a probe to be lowered in the borehole and a measuring or readout device to be located on the ground adjacent to the boring. The probe may be either the hydraulic type or the electric type. The hydraulic probe may be of a single cell or triple cell design. In the latter case, the measuring cell is located between two guard cells. The combined height of the measuring and guard cells, if any, shall be at least six diameters. The design of the probe shall be such that the drilling liquid may flow freely past the probe without disturbing the sides of the borehole during insertion or removal. For both systems, the nominal hole diameter shall not be more than 1.2 times the nominal probe diameter. Typical probe dimensions and corresponding borehole diameters are indicated in Table 1.

5.1.1 *Probe Walls*—The walls of the probe may consist of an inner rubber membrane and an outer flexible sheath which will take up the shape of the borehole as pressure is applied. In a coarse-grained material like gravel, a steel sheath made of thin overlapping metal strips is often used. The accuracy of the test will be impaired when the probe cannot take up the shape of the borehole accurately.

NOTE 4—Various membrane and sheath materials may be used to better accommodate soil types; identify the membrane used in the report.

5.1.2 *Measuring Devices*—Changes in volume of the measuring portion of the probe are measured in the hydraulic apparatus, and the probe diameter is measured by the use of feelers in the electric apparatus. Provisions to measure the diameter in directions at a 120° angle shall be provided with the electric apparatus. The measuring cell shall be prevented from expanding in the vertical direction by guard cells or other effective restraints in the hydraulic apparatus. The accuracy of the readout device shall be such that a change of 0.1% in the probe diameter is measurable.

5.1.3 *Lines*—Lines connecting the probe with the readout device consist of plastic tubing in the hydraulic apparatus. To reduce measuring errors, a coaxial tubing is used, whereby the inner tubing is prevented from expanding by a gas pressure at its perimeter. By applying the correct gas pressure, expansion of the inner tubing is reduced to a minimum. Single tubing can also be used. In both cases, requirement for volume losses given in 6.3 should apply. Electric lines need special protection against groundwater.

5.1.4 *Readout Device*—The readout device includes a mechanism to apply pressure (Procedure A) or volume (Procedure B) in equal increments to the probe and readout of volume change (Procedure A) or pressure change (Procedure B). The equipment using the hydraulic system and guard cells shall also include a regulator whereby the pressure



FIG. 1 Slotted Tube with Probe

in the gas circuit is kept below the fluid pressure in the measuring cell. The magnitude of pressure difference between gas and fluid must be adjustable to compensate for hydrostatic pressures developing in the probe. In the electrical system the volume readings are substituted by an electrical readout on the diameter of the probe.

5.2 *Sampling Tube*, similar to the thin-wall sampler described in Practice D 1587.

5.3 *Iwan-Type Auger*

5.4 *Pneumatic or Hydraulic Drifter*

5.5 *Slotted Tube*—A steel tube, (Fig. 1) that has a series of longitudinal slots (usually six) cut through it to allow for lateral expansion, is used as a protective housing when the probe is driven, vibrodriven, or pushed into the soil.

6. Calibration

6.1 The instrument shall be calibrated before each use to compensate for pressure losses (P_c) and volume losses (V_c).

6.2 *Pressure Losses*—Pressure losses (P_c) occur due to the rigidity of the probe walls. The pressure readings obtained during the test on the readout device include the pressure required to expand the probe walls; this membrane resistance must be deducted to obtain the actual pressure applied to the soil. Calibrations for membrane resistance shall be performed by inflating the probe, completely exposed to the atmosphere, with the probe placed at the level of the pressure gauge.

NOTE 5 **Warning**—The performance of the pressuremeter test, and particularly the calibration procedures, may present a safety hazard to the operator and persons assisting in the test. The blowout of the probe if on the ground or at shallow depth in the hole may cause injuries from flying debris. Wearing protective devices over the eyes and face or other



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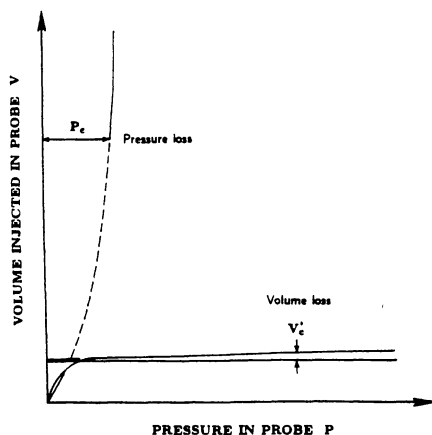


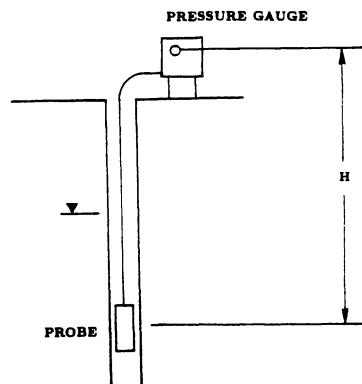
FIG. 2 Calibration for Volume and Pressure Losses

measures such as putting the probe in a protective cylinder during calibration are recommended

6.2.1 Apply pressures in 10-kPa increments for Procedure A and hold for 1 min. Make volume readings after 1-min elapsed time. When Procedure B is used, increase the volume of the probe in increments equal to 5 % of the nominal volume of the measuring portion of the uninflated probe (V_0). Apply the volume increase in about 10 s and hold constant for 1 min. Continue steps in both procedures until the maximum probe volume is reached. Plot results using a pressure versus volume plot. The obtained curve is the pressure calibration curve. The pressure correction is the pressure loss obtained for any particular volume reading from this graph (Fig. 2).

6.2.2 The pressure correction (P_c) must be deducted from the pressure readings obtained during the test. The maximum value of P_c should be less than 50 % of the limit pressure as defined in 9.6.

6.3 Volume Losses—Volume losses (V_c) occur due to expansion of tubing and compressibility of any part of the testing equipment, including the probe and the liquid. Calibration is made by pressurizing the equipment with the probe in heavy duty steel casing. The resulting volume versus pressure plot is the volume calibration curve. The zero volume calibration is obtained by a straight line extension of the curve to zero pressure, as shown in Fig. 2. The volume loss (V_c) of the instrument for a particular pressure is obtained as shown in Fig. 2. The volume correction is the volume loss (V_c) obtained for any particular pressure reading from the graph. This volume loss must be deducted from the measured volumes during the test. This correction is relatively small in soils and can be neglected if the correction is less than 0.1 % of the nominal volume of the measuring portion of the uninflated probe (V_0) per 100 kPa (1 tsf) of pressure. In very hard soils or rock, the correction is significant and must be applied. In no case should this correction exceed 0.5 % of the nominal volume of the measuring portion of the deflated probe (V_0) per 100 kPa (1 tsf) of pressure.

FIG. 3 Depth H for Determination of Hydrostatic Pressure in Probe

6.4 Corrections for temperature changes and head losses due to circulating liquid are usually small and may be disregarded in routine tests for soils. For tests at depths greater than 50 m (150 ft), special procedures are required to account for head losses.

6.5 The amount of hydrostatic pressure (P_h) exerted on the probe by the column of liquid in the testing equipment must be determined. This is accomplished by measuring the test depth (H) and multiplying the unit weight of the test liquid (δt) by the test depth (H), $R_h = H \times \delta t$ (Fig. 3). The test depth (H) is the distance from the center of the pressure gauge to the center of the probe. The obtained pressure is exerted on the probe but is not registered by the pressure gages. This pressure must accordingly be added to the pressure readings obtained on the readout device.

6.6 For triple cell pressuremeters, the pressure of the guard cells (P_G) must be set below the actual pressure generated in the probe to provide effective end restraint. This is obtained by subtracting this pressure from the test pressures as follows:

$$P_G = P_R + P_h - P_d$$

where:

P_R = pressure reading on control unit, kPa,

P_h = hydrostatic pressure between control unit and probe, kPa (see 5.1.2), and

P_d = pressure difference between guard cells and measuring cell, kPa (usually twice the limit pressure of the membrane).

6.6.1 A tabulation of gas and liquid pressures for a pressure difference of $P_d = 100$ kPa for various test depths is shown by Table 2.

7. Drilling

7.1 Whenever possible, place the pressuremeter probe by lowering it into a prebored hole. Two conditions are necessary to obtain a satisfactory test cavity: the diameter of the hole should meet the specified tolerances, and the equipment and method used to prepare the test cavity should cause the least possible disturbance to the soil and the wall of the hole.



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TABLE 2 Pressure Compensation for Guard Cells Based on Test Depth

Test Depth (H)		Liquid Pressure From Head of Test Liquid on Probe P, kPa	Gas Pressure Reduction On Readout Gages ^A P, 100 (kPa)
m	ft		
0	0	0	-100
5	17	50	-50
10	33	100	0
15	50	150	+50
20	67	200	+100

^A To maintain guard cell pressure 100 kPa below the measuring cell pressure, deduct (-) or add (+), these pressures to the guard cell circuit

When testing soils, the pressuremeter tests must be performed immediately after the hole is formed

7.2 The preparation of a satisfactory borehole is the most important step in obtaining an acceptable pressuremeter test. An indication of the quality of the test hole is given by the magnitude of scatter of the test points and by the shape of the pressuremeter curve obtained. Figure 4 shows the typical shape of a pressuremeter curve obtained from a prebored test cavity. Figure 5 shows a pressuremeter curve obtained when the borehole is too small or when the test is performed in a swelling soil. Figure 6 shows a curve obtained when the borehole is too large.

NOTE 6—The shape of the pressuremeter test curve is not sufficient to ensure that the test is reliable. The hole diameter requirements developed in 7.3.1 should also be met.

7.3 Requirements of Test Cavity with Respect to Probe Diameter

7.3.1 Hole Diameter—Dimensions used in this test method are as follows:

7.3.1.1 Diameter of the Pressuremeter Probe, D —The typical diameter D of the pressuremeter varies from approximately 24.5 to 127 mm (1 to 5 in.).

7.3.1.2 Diameter of Test Cavity, D_H —The diameter of the test cavity D_H should satisfy the following condition derived from experience:

$$1.03D < D_H < 1.2D$$

7.3.2 Cutting Tool Diameter

7.3.2.1 When determining the diameter of the necessary cutting tool for a bored hole, three factors must be considered: (a) the required diameter of the cavity, (b) the

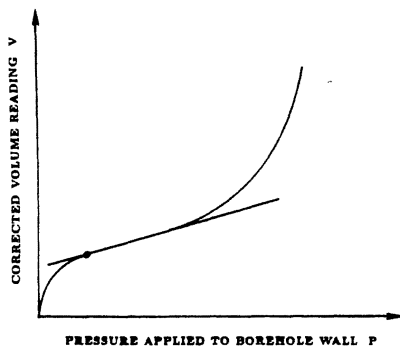


FIG. 4 Ideal Shape of the Pressuremeter Corrected Curve

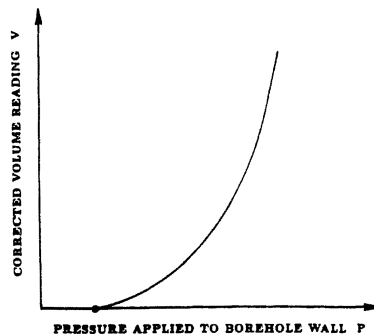


FIG. 5 Pressuremeter Corrected Curve When the Borehole is too Small

overcutting of the cavity resulting from the wobble of the cutting tool or the wall erosion by the mud circulation in medium to large-grained soils, or both, and (c) the inward yielding that occurs between the removal of the cutting tool and the probe placement. Inward yielding can be reduced by the use of drilling mud.

7.3.2.2 When selecting equipment for the site, several bits of various sizes should be available so as to adjust the size of the bit depending on whether overcutting or inward yielding prevails.

7.3.2.3 When selecting the tool consider also that the wall of the test cavity should be as smooth as possible and the diameter D_H should be as constant as possible over the length of the hole.

NOTE 7—If D_H varies significantly over the length of the probe, because of raveling for example, or if the borehole is noncylindrical, the quality of the test will be impaired.

7.4 Methods and Tools Used to Prepare the Test Cavity

7.4.1 Any method and tool that can satisfy the general requirements of 7.1 through 7.3 may be used.

7.4.2 The following methods are used to prepare the test cavity for the pressuremeter probe:

7.4.2.1 Rotary Drilling—The drill bits used are usually finger bits in clays and roller bits in sands and gravels.

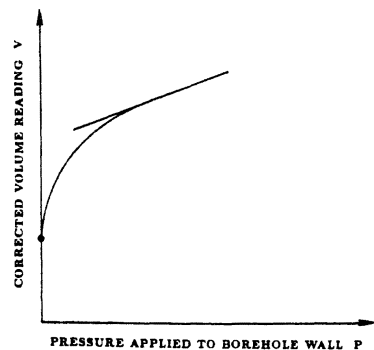


FIG. 6 Pressuremeter Corrected Curve When the Borehole is too Large

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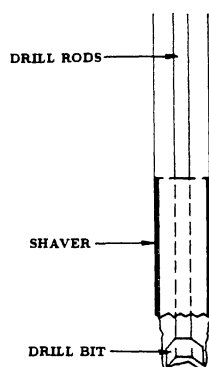


FIG. 7 Preparing the Test Cavity by the Pilot Hole Drilling and Simultaneous Shaving Technique

Advance the rotating drill bit into the soil while satisfying the following conditions: low vertical pressure on the drilling tool (200 kPa (30 psi)), slow rotation (less than 60 rotations per minute) and a regulated low drilling fluid flow (to less than 15 L/min (4 gal/min)). Inject the drilling fluid by axial bottom discharge to cause the least damage to the borehole wall. The fluid must have a viscosity high enough to remove the cuttings at low pumping rates.

7.4.2.2 Tube Sampling—Thin wall samplers similar to those described in Practice D 1587 are used. The sampling tube must be long enough to ensure that the length of cavity to be tested is obtained with a single push. If the tube plugs or if full recovery is not obtained, then another method of preparing the test cavity should be considered. Withdraw the tube slowly to limit inward yielding of the cavity wall due to suction. If thick wall samplers are used, an inward bevel cutting edge must be provided to minimize pre-testing stressing of the borehole wall.

7.4.2.3 Continuous Flight Augering—Use a single 1.52-m (5-ft) length of auger at the bottom of a drill string to advance the borehole to the testing level. The cutting head must be slightly greater in diameter than the auger flight to prevent smearing the borehole wall. Rotate the auger during withdrawal. The same rotation and penetration pressure parameters as in 7.4.2.1 apply to continuous flight augering.

7.4.2.4 Hand Augering—Use an Iwan-Type auger with or without a hand pump for bottom discharge injection of mud.

NOTE 8—The use of hand auger is difficult below a depth of 6 m (20 ft), and should accordingly be considered only for testing at shallow depths.

7.4.2.5 Driving or Vibrodriving a Sampler—Drive a split barrel sampler into the soil. Driving or vibrodriving a flush sampling tube may also be used. The requirements of 7.4.2.2 apply.

7.4.2.6 Core Drilling—This method is described in Practice D 2113.

7.4.2.7 Rotary Percussion—Use a pneumatic or hydraulic drifter working with a bottom discharge bit. The removal of cuttings can be done by compressed air in dry formations, or by mud in wet soils.

7.4.2.8 Pilot Hole Drilling and Subsequent Tube Sampling—Drill a pilot hole smaller in diameter than the

pressuremeter probe. Trim the hole to the proper diameter by a pushed or driven sampler. The requirements of 7.4.2.2 apply.

7.4.2.9 Pilot Hole Drilling and Simultaneous Shaving—Drill a pilot hole smaller in diameter than the pressuremeter probe. Immediately behind the drill bit, (Fig. 7) on the string of the drilling rods is a thin hollow cylinder that trims the cavity. Advance the drill bit and cylinder with high viscosity drilling fluid.

7.4.2.10 Driving, Vibrodriving, or Pushing a Slotted Tube—A slotted tube is generally used as a protective housing for the probe when the probe is driven, vibrodriven, or pushed into the soil. The slotted tube (Fig. 1) is a steel tube that has a series of longitudinal slots (usually six) cut through it to allow for lateral expansion. Place the probe in the slotted tube and drive, vibrodrive, or push the whole assembly into the soil to the testing depth. This method is a full displacement method and should only be used when non-displacement methods cannot be employed. Calibrate the probe within the slotted tube prior to testing.

7.5 Selecting Methods for Hole Preparation

7.5.1 Make the proper choice from the previously mentioned or other acceptable methods. This choice depends on the type of soil to be tested. The major influencing factors are:

7.5.1.1 Particle size distribution.

7.5.1.2 Plasticity.

7.5.1.3 Strength.

7.5.1.4 Degree of saturation.

7.5.2 Table 3 gives guidelines for selecting methods for borehole preparation in typical soils classified according to the factors mentioned in 7.5.1.1 through 7.5.1.4. Table 3 does not cover all possible methods of borehole preparation or probe placement, or both, and is included as a guide for selecting drilling methods.

8. Procedure

8.1 Perform the drilling of the borehole in accordance with Section 7.

8.2 Advance the hole to the test level and clean any debris or cuttings.

8.3 Before the probe is positioned in the hole for testing, make an accurate determination of the 0 volume reading (V_0). The volume V_0 is the volume of the measuring portion of the uninflated probe at atmospheric pressure. Accomplish this by deairing all circuits and adjusting all gages of the instrument to 0 while the probe is at atmospheric pressure. Close the volume circuit, preventing any further change in the volume of the measuring circuit. Lower the probe to test depth in this condition. Determine the test depth as the depth of the midpoint of the probe.

8.4 When using Procedure A, place the probe in test position and apply the pressure on the control unit in about equal increments, until the expansion of the probe during one load increment exceeds about $\frac{1}{4}$ of V_0 as defined in 8.3 (typically 200 cm³ for a 800-cm³ probe). Generally, 25, 50, 100, or 200-kPa pressures are selected for testing soils. Too small steps will result in an excessively long test, too large steps may yield results with inadequate accuracy. The pressure steps should be determined in such a way that about 7 to 10 load increments are obtained.



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TABLE 3 Guidelines for Selection of Borehole Preparation Methods and Tools^A

Soil	Type	Rotary Drilling With Bottom Discharge of Prepared Mud	Pushed Thin Wall Sampler	Pilot Hole Drilling and Subsequent Sampler Pushing	Pilot Hole Drilling and Simultaneous Shaving	Continuous Flight Auger	Hand Auger in the Dry	Hand Auger With Bottom Discharge of Prepared Mud	Driven or Vibro-driven Sampler	Core Barrel Drilling	Rotary Percussion	Driven Vibro-driven or Pushed Slotted Tube
Clayey soils	Soft	2 ^B	2 ^B	2	2	NR	NR	1	NR	NR	NR	NR
	Firm to stiff	1 ^B	1	2	2	1 ^B	1	1	NR	NR	NR	NR
	Stiff to hard	1	2	1	1	1 ^B	NA	NA	NA	1 ^B	2 ^B	NR
Silty soils	Above GWL ^C	1 ^B	2 ^B	2	2 ^B	1	1	2	2	NR	NR	NR
	Under GWL ^C	1 ^B	NR	NR	2 ^B	NR	NR	1	NR	NR	NR	NR
Sandy soils	Loose and above GWL ^C	1 ^B	NR	NR	2	2	2	1	2	NA	NR	NR
	Loose and below GWL ^C	1 ^B	NR	NR	2	NR	NR	1	NR	NA	NR	NR
	Medium to dense	1 ^B	NR	NR	2	1	1	1	2	NR	2 ^B	NR
Sandy gravel or gravelly sands below GWL	Loose	2	NA	NA	NA	NA	NA	NA	NR	NA	2	2
	Dense	NR	NA	NA	NA	NR	NA	NA	NR	NA	2	1 ^D
Weathered rock		1	NA	2 ^B	NA	1	NA	NA	1	2	2	NR

^A 1 is first choice, 2 is second choice, NR is not recommended, and NA is nonapplicable^B Method applicable only under certain conditions (see text for details)^C GWL is ground water level^D Pilot hole drilling required beforehand

8.5 When using Procedure B, increase the volume of the probe in volume increments of 0.05 to 0.1 times the volume V_0 (as defined in 8.3) until the limit of the equipment is reached.

8.6 For both procedures, take readings after 30 s and 1 min after the pressure or volume increments have been applied. Volume readings are recorded to an accuracy of 0.2 % of V_0 (as defined in 8.3) and pressure readings to an accuracy of 5 % of the limit pressure.

8.7 Once the test has reached the maximum test step as determined in 8.4 and 8.5, terminate the test by deflating the

probe to its original volume and removing the probe from the hole.

8.8 One or several load-unload cycles may also be performed in this test within the elastic expansion range (see Fig. 8). This test, if a probe with guard cells is used, requires the accurate control of gas pressure in the guard cells to obtain a representative reading on decreased volumes.

NOTE 9—Strain-controlled tests can also be performed whereby the probe volume is increased at a constant rate and corresponding pressures are measured. This method shall be applied only if special requirements must be met, and is not covered by this test method. Strain-controlled tests may yield different results than the procedure described in this test method.

9. Spacing and Testing Sequence

9.1 Minimum spacing between consecutive tests (center to center of probe) should not be less than 1½ times the length of the inflatable part of the probe. Common spacings vary from 1 to 3 m (3 to 10 ft).

9.2 In soft, loose, and sensitive soils, the hole should be predrilled ahead of the testing depth only far enough so that the cuttings settling at the bottom of the hole will not interfere with the test.

9.3 In stiff soils and weathered rocks where degradation due to exposure is not significant, the hole can be predrilled to several test depths.

9.4 When the probe is driven into the soil, testing can take place continuously, while observing the minimum spacing requirements indicated in 9.1. No withdrawal is required between tests.

9. Calculations

9.1 The pressure transmitted to the soil by the probe from the pressure readings is calculated as follows:

$$P = P_R + P_\delta - P_c$$

where

P = pressure exerted by the probe on the soil, kPa,

P_R = pressure reading on control unit, kPa,

P_δ = hydrostatic pressure between control unit and probe, kPa, and

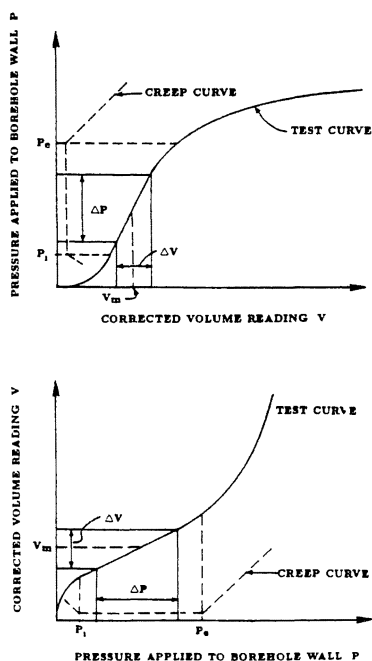


FIG. 8 Pressuremeter Test Curves for Procedure A

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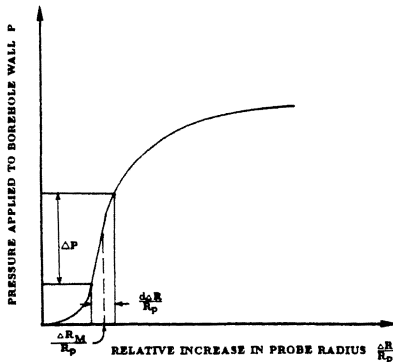


FIG. 9 Pressure Versus Relative Increase in Radius

P_c = pressure correction due to stiffness of instrument at corresponding volume, kPa, determined in accordance with 6.2

9.1.1 The pressure P_δ shall be the hydrostatic pressure as follows

$$P_\delta = H \times \delta_i$$

where

H = depth of probe below the control unit, m,

δ_i = unit weight of measuring liquid in instrument, kN/m³

9.2 Calculate the corrected volume reading of the probe from the volume readings as follows

$$V = V_R - V_c$$

where,

V = corrected increase in volume of the measuring portion of the probe, cm³,

V_R = volume reading on readout device, cm³, and

V_c = volume correction at the corresponding pressure reading, cm³, determined in accordance with 6.3

9.3 Plot the pressure-volume increase curve by entering the corrected volume and the corrected pressure on a coordinate system. Connect the points by a smooth curve. This curve is the corrected pressuremeter test curve and is used in the determination of the results (Fig. 8(a) and 8(b)). Other plots, such as pressure versus relative increase in radius, may also be used (Fig. 9)

NOTE 10—Historically, pressures were plotted on the horizontal axis and volume on the vertical axis. Considering the stress-strain nature of this test, it has become increasingly customary to reverse the coordinates. According to this test method, both presentations are acceptable.

9.4 For Procedure A, plot the volume increase readings (V_{60}) between the 30 s and 60 s reading on a separate graph. Generally, a part of the same graph is used, see Fig. 8. For Procedure B, plot the pressure decrease reading between the 30 s and 60 s reading on a separate graph. The test curve shows an almost straight line section within the range of either low volume increase readings (V_{60}) for Procedure A or low pressure decrease for Procedure B. In this range, a constant soil deformation modulus can be measured. Past the so-called creep pressure, plastic deformations become prevalent.

9.5 The pressuremeter modulus is determined as follows

$$E_p = 2(1 + \gamma)(V_0 + V_m) \frac{\Delta P}{\Delta V}$$

where

E_p = pressuremeter modulus, kPa, an arbitrary modulus of deformation as related to the pressuremeter based on data reduction included herein,

γ = poisson ratio,

NOTE 11—For compatibility with tests performed with this instrument earlier, a value of 0.33 is recommended by this test method. Other values may be used, but the value must be reported.

V_0 = volume of the measuring portion of the uninflated probe at 0 volume reading at ground surface, cm³,

V = corrected volume reading of the measuring portion of the probe,

ΔP = corrected pressure increase in the center part of the straight line portion of the pressure-volume curve (see Fig. 8),

ΔV = corrected volume increase in the center part of the straight line portion of the pressure-volume curve, corresponding to ΔP pressure increase (see Fig. 8), and

V_m = corrected volume reading in the center portion of the ΔV volume increase

$V_0 + V$ = current volume of inflated probe

NOTE 12—If a break in the straight line portion of the pressuremeter curve is observed, calculations shall include a pressuremeter modulus for each straight line section of the pressuremeter test curve.

NOTE 13—A pressuremeter modulus can also be calculated from an unload-reload cycle. This modulus should be identified as the reload pressuremeter modulus (Fig. 10).

NOTE 14—For tests where the probe diameter (radius) is measured, the pressuremeter modulus can be determined by converting the measurements into volume changes of the probe, in which case the formula given in this test method will apply (9.5). The pressuremeter modulus may also be calculated from diameter measurements directly as follows

$$E_p = (1 + \gamma)(R_p + \Delta R_m) \Delta P / d\Delta R$$

where

R_p = radius of probe in uninflated condition, mm,

ΔR_m = increase in radius of probe up to the point corresponding to the pressure where E_p is measured, mm,

$d\Delta R$ = increase of probe radius corresponding to ΔP pressure increase, mm,

ΔR = increase in probe radius, mm, and

$R_p + \Delta R$ = current radius of inflated probe, mm

9.6 The limit pressure is determined as follows: the limit pressure (P_l) is defined as the pressure where the probe volume reaches twice the original soil cavity volume, defined as the volume $V_0 + V_l$, (Fig. 8) where V_l is the corrected volume reading at the pressure where the probe made contact with the borehole. The volume reading at twice the original soil cavity volume is $2(V_0 + V_l)$. The limit pressure is usually not obtained by direct measurements during the test due to limitation in the probe expansion or excessively high pressure. If the test was conducted to read sufficient plastic deformation, the limit pressure can be determined by a $1/V$ to P plot, as shown by Fig. 11.

9.6.1 Points from the plastic range of the test generally fall in an approximate straight line. The extension of this line to



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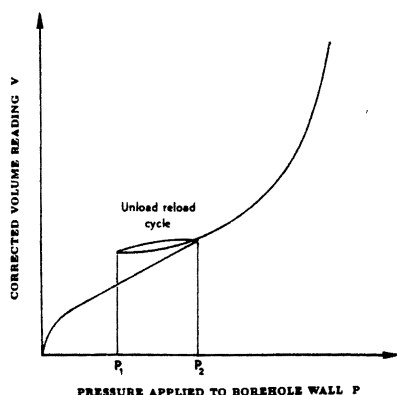


FIG. 10 Cyclic Pressuremeter Test Curve

twice the original probe volume will give the limit pressure (P_l) on the plot

NOTE 15—The theoretical limit pressure is defined as the pressure where infinite expansion of the probe occurs. For practical purposes the definition outlined in 9.6 is recommended. Several methods are used to estimate the limit pressure from points measured during the test. These methods may also be used but should be properly reported.

NOTE 16—When the requirement of 7.3.1 about hole diameter tolerances is not met, only part of the test curve may be suitable for interpretation. The limit pressure is relatively insensitive to borehole size.

10. Report

10.1 For each pressuremeter test the following observations shall be recorded:

- 10.1.1 Type of test (Procedure A or B) and date
- 10.1.2 Boring number.
- 10.1.3 Size of probe
- 10.1.4 Description of membrane and sheath on probe and calibration
- 10.1.5 Depth of center point of probe
- 10.1.6 Pressure or volume steps
- 10.1.7 Volume readings at 30 and 60-s elapsed time for each load increment for Procedure A, pressure readings at 30 and 60-s elapsed time for Procedure B
- 10.1.8 Notes on any deviation from standard test procedure

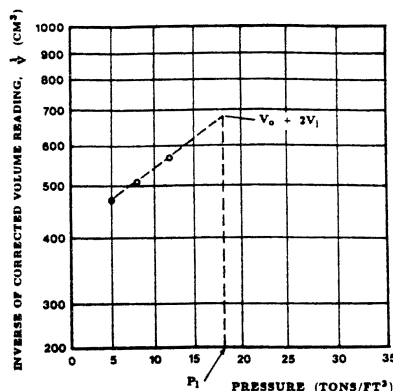


FIG. 11 Determination of Limit Pressure from Inverse of Volume Versus Pressure

10.1.9 Volume versus pressure graph, pressuremeter modulus, limit pressure.

10.1.10 Calibration curves

10.2 In addition, the following observations shall be recorded for the boring:

- 10.2.1 Boring number
- 10.2.2 Log of soil conditions
- 10.2.3 Reference elevation
- 10.2.4 Depth of water in the hole at the time of test
- 10.2.5 Method of making the hole and method of preparing the cavity
- 10.2.6 Type of testing equipment used
- 10.2.7 Notes on driving resistance in the boring (SPT test N value)
- 10.2.8 Weather and temperature
- 10.2.9 Name of drilling foreman

11. Precision and Bias

11.1 The single most important factor in the successful completion of a preboring pressuremeter test is the preparation of a good hole. A good hole is very difficult to prepare in very soft clays and very loose sands. The pressuremeter limit pressure is relatively insensitive to the quality of the borehole, however, the pressuremeter modulus is much more sensitive to the quality of the borehole.

11.2 The subcommittee is seeking pertinent data from users of this test method to develop a precision statement.

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This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, PA 19103.

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VITA

Michael L. Hughes

Candidate for the Degree of
Doctor of Philosophy

Thesis: THE FEASIBILITY OF USING THE MENARD-TYPE PRESSUREMETER IN OKLAHOMA PERMIAN RED SHALE-CLAYS FOR THE DESIGN OF LATERALLY LOADED DRILLED SHAFTS

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Tulsa, Oklahoma, July 21, 1945, the son of Mr. and Ms. Vernon L. Hughes.

Education: Graduated from Davenport High School, Davenport, Oklahoma, in May, 1963; received the Bachelor of Science degree in Civil Engineering from Oklahoma State University in 1969; received the Master of Science degree in Civil Engineering from Oklahoma State University in 1970; completed requirements for the Doctor of Philosophy degree at Oklahoma State University in May, 1991.

Professional Experience: Undergraduate Assistant, School of Civil Engineering, Oklahoma State University, September, 1968, to August, 1969; Graduate Assistant, School of Civil Engineering, Oklahoma State University, August, 1969, to July, 1970; Civil Engineer; Senior Civil Engineer; Senior Project Engineer; Project Supervisor; Supervisor, Facility Engineering; Manager, Facility Engineering; Manager, Environmental Affairs; Oklahoma Gas and Electric Company, Oklahoma City, Oklahoma, August, 1970, to present.

Professional Societies: Member, American Society of Civil Engineers; Member, Society of American Military Engineers; Professional Engineer: Oklahoma (No. 9612), Arkansas (No. 6257).