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16. Abstract

The retaining walls which are dealt with in this report are drilled shaft retaining walls. For these walls, the resistance to overturning is developed from the embedded part of the wall by flexural rigidity. In a first part the basic finite difference method of solving such retaining wall problems is described and a simple example is given. In a second part a parametric analysis of the solution using a conventional method is performed to point out which of the wall and soil parameters are most important. The pressuremeter method recommended by Menard is presented in the third part. Finally predictions are made for two future drilled shaft walls in Houston, Texas.

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PRESSUREMETER DESIGN OF RETAINING WALLS

by

Jean-Louis Briaud, Michael Meriwether, and Hubert Porwoll

Research Report 340-4F

The Pressuremeter and the Design of Highway Related Foundations
Research Study 2-5-83-340

Sponsored by

State Department of Highways and Public Transportation
In cooperation with the
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Texas Transportation Institute
The Texas A&M University System
College Station, Texas

October 1983

SUMMARY

The retaining walls which are dealt with in this report are drilled shaft retaining walls. For these walls the resistance to overturning is developed from the embedded part of the wall by flexural rigidity. In this study an assessment of the existing pressuremeter method for the design of such walls is attempted.

In a first part the finite difference p-y method to solve such problems is described and a simple example is given to clarify the steps followed by the computer program.

In a second part a conventional method is described which consists of using an elastic plastic p-y curve model using the active and passive earth pressure coefficients. A parametric analysis of the solution using the above method is performed, and it is shown that the pile flexural rigidity and the soil friction angle are two of the most influencial parameters.

In a third part the method proposed by Menard is presented. This method is based on the use of the pressuremeter modulus and the pressuremeter limit pressure to generate the p-y curves for the embedded part of the wall.

In a fourth part, two case histories of drilled shaft walls in Houston, Texas, are reported. The two drilled shaft walls are not yet built but pressuremeter tests were performed at the sites and behavior predictions are presented.

ACKNOWLEDGMENTS

The authors are grateful for the continued support and encouragement of Mr. George Odom of the Texas State Department of Highways and Public Transportation.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views of policies of the Federal Highway Administration or the State Department of Highways and Public Transportation. This report does not constitute a standard, a specification, or a regulation.

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IMPLEMENTATION STATEMENT

This report gives the details of an existing pressuremeter method for the design of drilled shaft retaining walls. This method requires the use of a new piece of equipment: a preboring pressuremeter. This method is directly applicable to design practice and should be used in parallel with current methods for a period of time until a final decision can be taken as to its implementation.

CHAPTER 1. INTRODUCTION

This study is related to the design of drilled shaft retaining walls, sheet pile walls, slurry trench walls, and more generally to the design of walls which develop part or all of the retaining force from the resistance of the embedded portion of the wall (Fig. 1).

There are various types of methods available to design such retaining walls. The first type of method which can be used is the limit equilibrium approach, where the global equilibrium of the wall is considered; the distribution of soil pressure is assumed to be the active pressure behind the wall and the passive pressure in front of the embedded part of the wall multiplied by an appropriate factor of safety. This method does not predict the deformation of the wall.

The second type of method is the finite element method where the wall and the soil surrounding the wall are modeled by finite elements. This method gives the prediction of soil and wall displacements. At the present time, however, this method is rather expensive due to the large number of elements necessary to model the problem properly and the associated cost of computer runs.

The third type of method is the finite difference method where the wall is modeled by a series of elements acted upon by nonlinear spring models representing the soil reaction. This method gives a prediction of wall displacement and can be considered as being of intermediate complexity between the first and second type of method. The nonlinear springs modeling the soil behavior are described by p-y curves where p is the pressure on the wall at depth z and y is the displacement of the wall at the same depth z (Fig. 2). This method is the one which is described and used throughout this report.

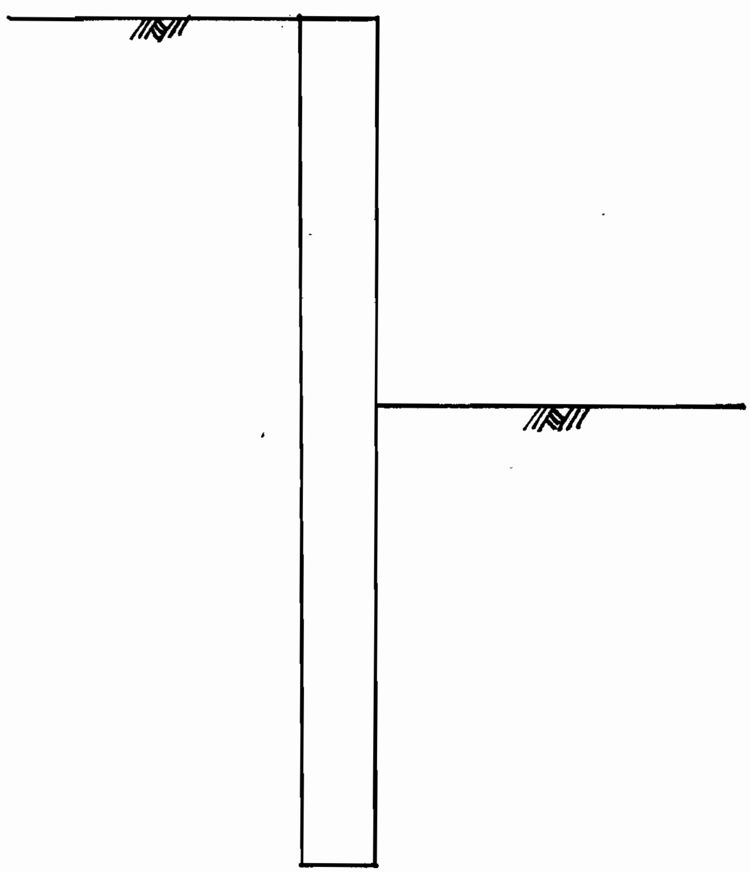


FIG. 1. Retaining Wall

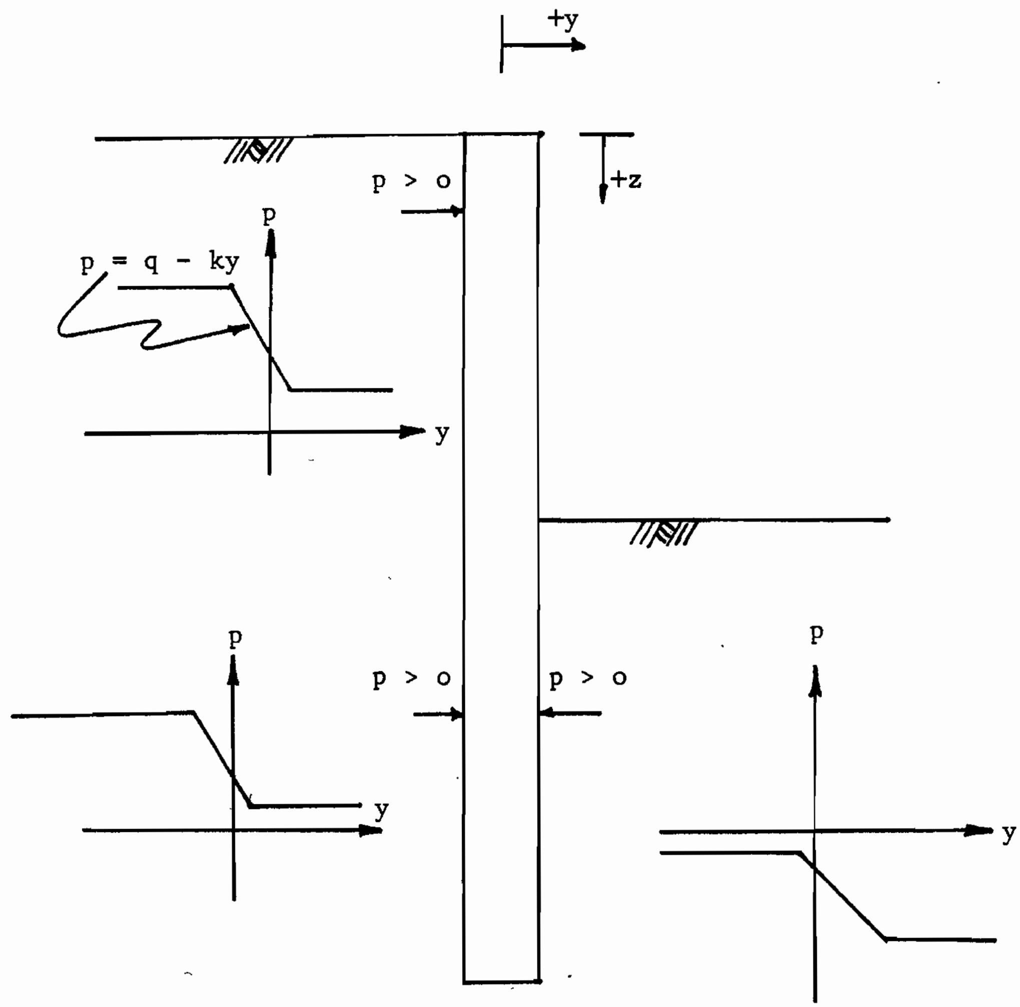


FIG. 2. Pressure-deflection Curves

CHAPTER 2. FINITE DIFFERENCE METHOD

2.1 Theory

The constitutive equation for the wall in bending is

$$M = EI\phi = EI \frac{d^2y}{dz^2} \quad \ldots \quad (1)$$

where M = the bending moment at depth z,

E = the modulus of elasticity of the wall,

I = the moment of inertia of the wall for a unit width of wall,

ϕ = the wall curvature = $1/R = d^2y/dz^2$,

R = the wall radius of curvature,

y = the wall lateral displacement, and

z = the depth.

Considering a unit width of wall, the equilibrium equations of a wall element lead to

$$w = \frac{dV}{dz} \quad \ldots \quad (2)$$

where w = the force per unit length or loading intensity on the wall,

V = the shear force at depth z,

and $$dM = Qdy + Vdz \quad \ldots \quad (3)$$

where Q is the axial load,

thus $$w = \frac{d^2M}{dz^2} + Q\frac{d^2y}{dz^2} \quad \ldots \quad (4)$$

The governing equation is obtained by combining equations 1 and 4:

$$EI \frac{d^2y}{dz^4} + Q \frac{d^2y}{dz^2} - w\ (y,z) = 0 \quad \ldots\ldots\ldots\ldots \quad (5)$$

Generally there is no axial load Q on the wall and the equation reduces to:

$$EI \frac{d^2y}{dz^4} - w\ (y,z) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

Taking a unit width of wall, soil pressure p can be substituted for loading intensity since loading intensity is the product of pressure and width. For simplicity, pressure will be assumed to vary linearly with deflection resulting in the expression (Fig. 2)

$$p = q - ky$$

where q = the pressure at zero deflection, and

k = the slope of the p-y line.

This gives the equation

$$\frac{q - ky}{EI} = \frac{d^2y}{dx^4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

When the wall is divided into a number of discrete segments, the finite difference method can be used to solve this equation.

From finite differences:

$$\frac{d^2y}{dx^2} = \frac{y_{i+1} - 2y_i - y_{i-1}}{h^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

$$\frac{d^2y}{dx^3} = \frac{y_{i+2} - 2y_{i+1} + 2y_{i-1} - y_{i-2}}{2h^3} \quad \ldots\ldots\ldots\ldots \quad (9)$$

$$\frac{d^2y}{dx^4} = \frac{y_{i+2} - 4y_{i+1} + 6y_i - 4y_{i-1} + y_{i+2}}{h^4} \quad \ldots\ldots\ldots \quad (10)$$

where y_i = deflection at node i, and

h = distance between nodes.

Substituting equation (8) into equation (7) gives

$$\frac{q_i - k_i y_i}{EI} = \frac{y_{i+2} - 4y_{i+1} + 6y_i - 4y_{i-1} + y_{i+2}}{h^4}$$

or $$\frac{q_i h^4}{EI} = y_{i+2} - 4y_{i+1} + (6 + \frac{k_i h^4}{EI})\, y_i - 4y_{i-1} + y_{i+2} \quad \ldots \quad (11)$$

For a wall of n nodes, n equations of this form can be written. Since q, h, k, and EI are known for each node, the only unknowns are the n+4 deflections. The four extra deflections come from imaginary nodes: two above the top node and two below the bottom node of the wall. The four additional equations required come from boundary conditions. Shear and moment are known to be zero at both top and bottom of the wall and the resulting equations are:

$$\frac{V_o}{EI} = 0 = \frac{d^3 y_o}{dx^3} = \frac{y_2 - 2y_1 + 2y_{-1} - y_{-2}}{2h^3}$$

$$\frac{M_o}{EI} = 0 = \frac{d^2 y_o}{dx^2} = \frac{y_1 - 2y_o + y_{-1}}{h^2}$$

$$\frac{V_n}{EI} = 0 = \frac{d^3 y_n}{dx^3} = \frac{y_{n+2} - y_{n+1} + 2y_{n-1} - y_{n-2}}{2h^3}$$

$$\frac{M_n}{EI} = 0 = \frac{d^2 y_n}{dx^2} = \frac{y_{n+1} - 2y_n + y_{n-1}}{h^2}$$

These equations can be rewritten as:

$$y_2 - 2y_1 + 2y_{-1} - y_{-2} = 0 \quad \ldots \quad (12)$$

$$y_1 - 2y_o + y_{-1} = 0 \quad \ldots \quad (13)$$

$$y_{n+2} - 2y_{n+1} + 2y_{n-1} - y_{n-2} = 0 \quad \ldots \quad (14)$$

$$y_{n+1} - 2y_n + y_n = 0 \quad \ldots \quad (15)$$

Collecting equations gives

$$0 = y_2 - 2y_1 + 2y_{-1} - y_{-2}$$

$$0 = y_1 - 2y_o + y_{-1}$$

$$\frac{q_o h^4}{EI} = y_2 - 4y_1 + (6 + \frac{k_o h^4}{EI})\ y_o - 4y_{-1} + y_{-2}$$

$$\frac{q_1 h^4}{EI} = y_3 - 4y_2 + (6 + \frac{k_1 h^4}{EI})\ y_1 - 4y_o + y_{-1}$$

$$\vdots$$

$$\frac{q_n h^4}{EI} = y_{n+2} - 4y_{n+1} + (6 + \frac{k_n h^4}{EI})\ y_n - 4y_{n-1} + y_{n-2}$$

$$0 = y_{n+2} - 2y_{n+1} + 2y_{n-1} - y_{n-2}$$

$$0 = y_{n+1} - 2y_n + y_{n-1}$$

Rewriting in matrix form gives

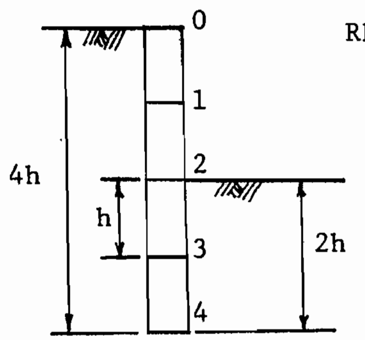
$$
\begin{bmatrix}
-1 & 2 & 0 & -2 & 1 & & \\
 & 1 & -2 & 1 & & & \\
1 & -4 & 6+\frac{k_o h^4}{EI} & -4 & 1 & & \\
 & 1 & -4 & 6+\frac{k_1 h^4}{EI} & -4 & 1 & \\
 & & & \ddots & & & \\
 & & 1 & -4 & 6+\frac{k_n h^4}{EI} & -4 & 1 \\
 & & & 1 & -2 & 1 & \\
 & & -1 & 2 & 0 & -2 & 1
\end{bmatrix}
\begin{Bmatrix}
y_{-2} \\ y_{-1} \\ y_o \\ y_1 \\ \vdots \\ y_n \\ y_{n+1} \\ y_{n+2}
\end{Bmatrix}
=
\begin{bmatrix}
0 \\ 0 \\ \frac{q_o h^4}{EI} \\ \frac{q_1 h^4}{EI} \\ \vdots \\ \frac{q_n h^4}{EI} \\ 0 \\ 0
\end{bmatrix}
$$

This method will not work when the wall has infinite stiffness because the coefficient matrix becomes singular and the solution matrix is uniformly zero.

This is the method used in the computer program BMCOL7 with the added feature of being able to handle nonlinear expressions for pressure. BMCOL7 uses the iterative technique to handle nonlinear expressions for pressure. To do this, pressure-deflection curves are simplified into a number of straight line segments and the coordinates of each segment end point are entered into the program. Starting with zero deflection, q and k are computed for each node. Where no p-y curve has been input, the program interpolates to find a curve. Next, the program runs through the finite difference method and computes deflections. If these deflections differ sufficiently from the previous ones, another iteration must be done. Using the new deflections, q and k are recalculated from the p-y curve. K is the slope of the curve and q is the pressure found by extending the tangent to the curve back to where it intercepts the p-axis (Fig. 3).

The iteration process is continued until it closes on the correct deflection for each node. BMCOL7 then uses the deflections to compute the slope, moment, shear, and reaction at each node.

2.2 Example



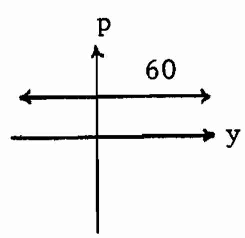
RETAINING WALL OF UNIT WIDTH

$EI = 10{,}000$

$h = 1$

THE P-Y CURVES ARE AS FOLLOWS:

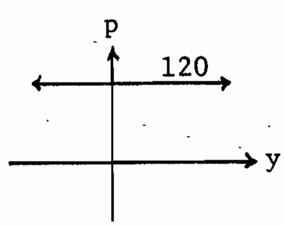
NODE 1



$\omega_1 = p_1 - k_1 y = 60$

$p_1 = 60 \qquad k_1 = 0$

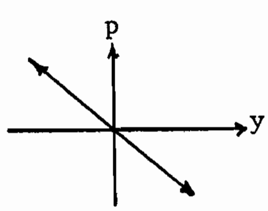
NODE 2



$\omega_2 = 120$

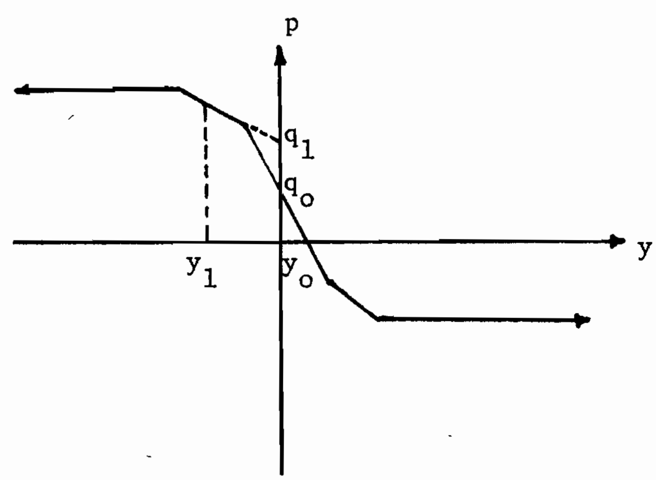
$p_2 = 120 \qquad k_2 = 0$

NODE 3



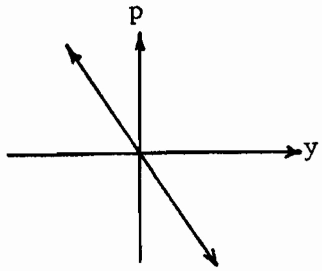
$\omega_3 = p_3 - k_3 y = 0 - 1000y$

$p_3 = 0 \qquad k_3 = 1000$



<u>FIG. 3</u>. Example of a Simplified p-y Curve

NODE 4



$$\omega_4 = p_4 - k_4 y = 0 - 1500y$$

$$p_4 = 0 \qquad k_4 = 1500$$

THERE ARE NO PRESSURES AT NODE 0 SO $p_o = k_o = 0$

BY OBSERVATION $V_o = M_o = V_4 = M_4 = 0$

SET UP THE SET OF EQUATIONS IN MATRIX FORM

$$\begin{bmatrix} -1 & 2 & 0 & -2 & 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & -2 & 1 & 0 & 0 & 0 & 0 & 0 \\ 1 & -4 & 6+\frac{k_o h^4}{EI} & -4 & 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & -4 & 6+\frac{k_1 h^4}{EI} & -4 & 1 & 0 & 0 & 0 \\ 0 & 0 & 1 & -4 & 6+\frac{k_2 h^4}{EI} & -4 & 1 & 0 & 0 \\ 0 & 0 & 0 & 1 & -4 & 6+\frac{k_3 h^4}{EI} & -4 & 1 & 0 \\ 0 & 0 & 0 & 0 & 1 & -4 & 6+\frac{k_4 h^4}{EI} & -4 & 1 \\ 0 & 0 & 0 & 0 & 0 & 1 & -2 & 1 & 0 \\ 0 & 0 & 0 & 0 & -1 & 2 & 0 & -2 & 1 \end{bmatrix} \begin{Bmatrix} y_{-2} \\ y_{-1} \\ y_o \\ y_1 \\ y_2 \\ y_3 \\ y_4 \\ y_5 \\ y_6 \end{Bmatrix} = \begin{bmatrix} \frac{2V_o h^3}{EI} \\ \frac{M_o h^2}{EI} \\ \frac{p_o h^4}{EI} \\ \frac{p_1 h^4}{EI} \\ \frac{p_2 h^4}{} \\ \frac{p_3 h^4}{EI} \\ \frac{p_4 h^4}{EI} \\ \frac{2V_4 h^3}{EI} \\ \frac{M_4 h^2}{EI} \end{bmatrix}$$

INPUT VALUES FOR P, K, V, M, h, AND EI

$$\begin{bmatrix} -1 & 2 & 0 & -2 & 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & -2 & 1 & 0 & 0 & 0 & 0 & 0 \\ 1 & -4 & 6 & -4 & 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & -4 & 6 & -4 & 1 & 0 & 0 & 0 \\ 0 & 0 & 1 & -4 & 6 & -4 & 1 & 0 & 0 \\ 0 & 0 & 0 & 1 & -4 & 6.1 & -4 & 1 & 0 \\ 0 & 0 & 0 & 0 & 1 & -4 & 6.15 & -4 & 1 \\ 0 & 0 & 0 & 0 & 0 & 1 & -2 & 1 & 0 \\ 0 & 0 & 0 & 0 & -1 & 2 & 0 & -2 & 1 \end{bmatrix} \begin{Bmatrix} y_{-2} \\ y_{-1} \\ y_o \\ y_1 \\ y_2 \\ y_3 \\ y_4 \\ y_5 \\ y_6 \end{Bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \\ .006 \\ .012 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

SOLVING THE SET OF 9 SIMULTANEOUS LINEAR EQUATIONS YIELDS THE FOLLOWING DISPLACEMENTS:

$y_{-2} = 4.26$

$y_{-1} = 3.49$

$y_o = 2.72$

$y_1 = 1.95$

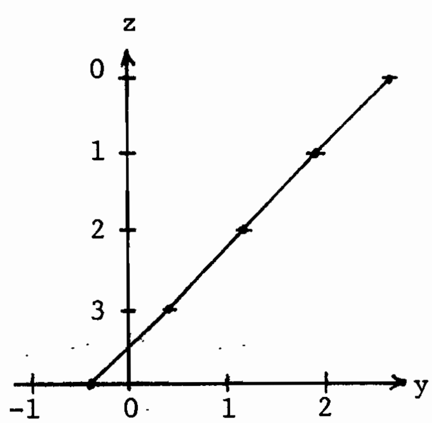
$y_2 = 1.18$

$y_3 = .42$

$y_4 = -.32$

$y_5 = -1.06$

$y_6 = -1.77$



ANY CONSISTENT SET OF UNITS CAN BE USED.

DEFLECTIONS WILL BE IN UNITS OF $\frac{P}{K}$.

KNOWING THE DISPLACEMENTS, THE WALL PRESSURES ARE:

$\omega_o = 0$

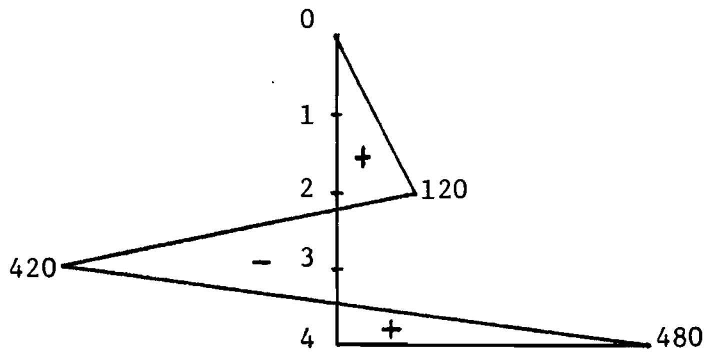
$\omega_1 = 60$

$\omega_2 = 120$

$\omega_3 = -1000\ (.42) = -420$

$\omega_4 = -1500\ (-.32) = 480$

ASSUMING LINEAR DISTRIBUTION, THE WALL PRESSURE DIAGRAM IS:



SUMMING FORCES AND MOMENTS INDICATES THAT THE WALL IS IN EQUILIBRIUM AND THAT THE SOLUTION IS VALID.

CHAPTER 3. PARAMETRIC STUDY

3.1 Conventional Method

3.1.1. The p-y Curve

Conventional soil mechanics gives the active, passive, and at rest soil pressures at a depth, z, as follows for dry conditions:

$$P_{active} = K_a \gamma z$$

$$P_{passive} = K_p \gamma z$$

$$P_{at\ rest} = K_o \gamma z$$

where K_a = the active soil pressure coefficient,

K_p = the passive soil pressure coefficient,

K_o = the at rest soil pressure coefficient, and

γ = the soil unit weight.

K_o is assumed to be 0.5. K_a and K_p are computed from the angle of internal friction, ϕ.

$$K_a = \tan^2 (45° - \frac{\phi}{2})$$

$$K_p = \tan^2 (45° + \frac{\phi}{2})$$

Various recommendations exist concerning the amount of deflection needed to develop these pressures. For this study, deflections of 2 and 10 mm (0.08 and 0.4 in.) were used as the deflections necessary to mobilize the full active and passive pressures, respectively. The p-y curve can then be plotted as shown (Fig. 4). Note that pressure is assumed to vary linearly between the successive points. For BMCOL7, these three points and two more at very large positive and negative deflections are used as input.

Such p-y curves are applicable only to depths above the excavation level where the soil exists only on one side of the wall. Below the

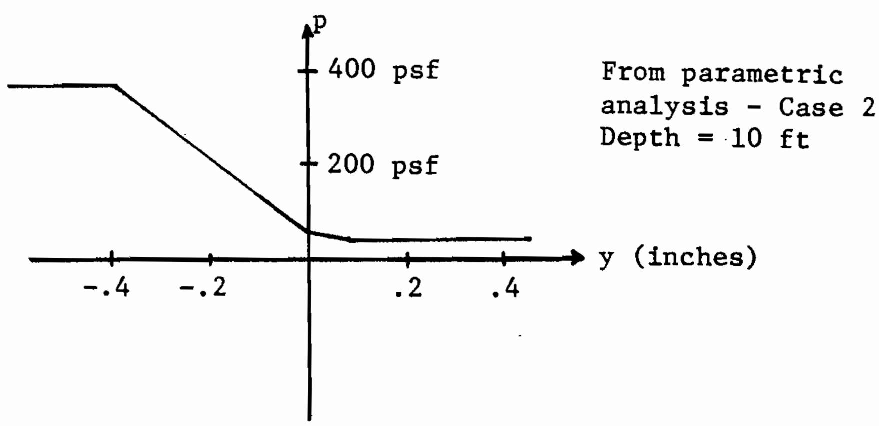


FIG. 4. p-y Curve Above Excavation Level

excavation level the soil on the other side of the wall must be taken into account. The method of superposition is used to find these p-y curves (Fig. 5). Instead of inputting five points, seven p-y values are now required for BMCOL7.

BMCOL7 does not require the input of a p-y curve at each node since it interpolates between curves. The number of p-y curves needed to get a fair representation of a particular problem is determined by discontinuities. Obvious discontinuities are the excavation limit, both ends of the wall, and changes in soil properties.

3.1.2 The Wall Stiffness

The stiffness of reinforced concrete retaining walls does not remain constant, but decreases as loads are applied. This is due to cracking of the concrete which reduces the moment of inertia. The American Concrete Institute recommends the use of the gross moment of inertia until the applied bending moment exceeds the cracking moment. At this point, the effective moment of inertia should be used (Fig. 6):

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

where I_e = the effective moment of inertia,

M_{cr} = cracking moment due to bending,

M_a = the applied bending moment,

I_g = the gross moment of inertia, and

I_{cr} = the cracked moment of inertia.

Unfortunately, BMCOL7 does not have the capability to compute the effective moment of inertia. The user can address this problem by using the iteration method with each computer run being an iteration.

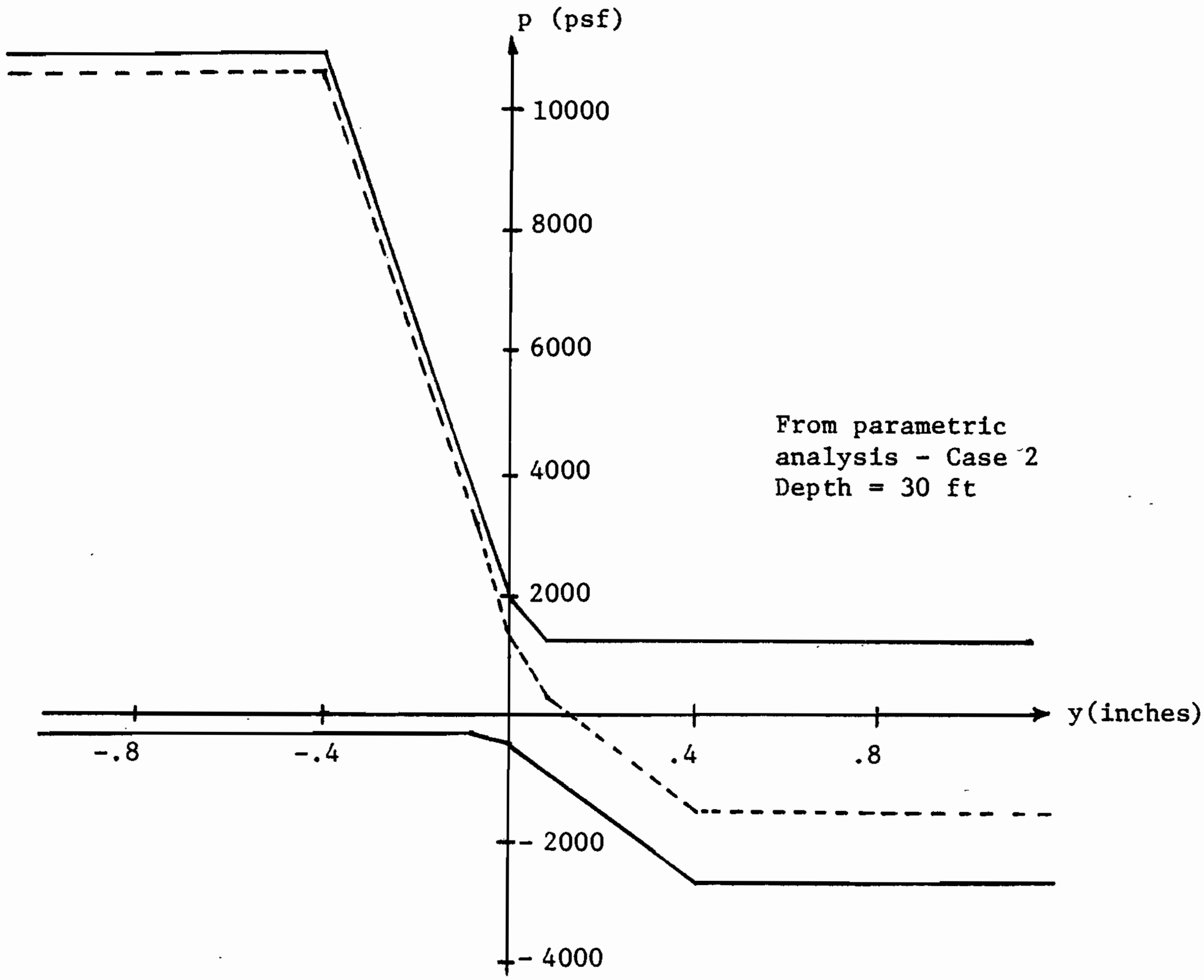
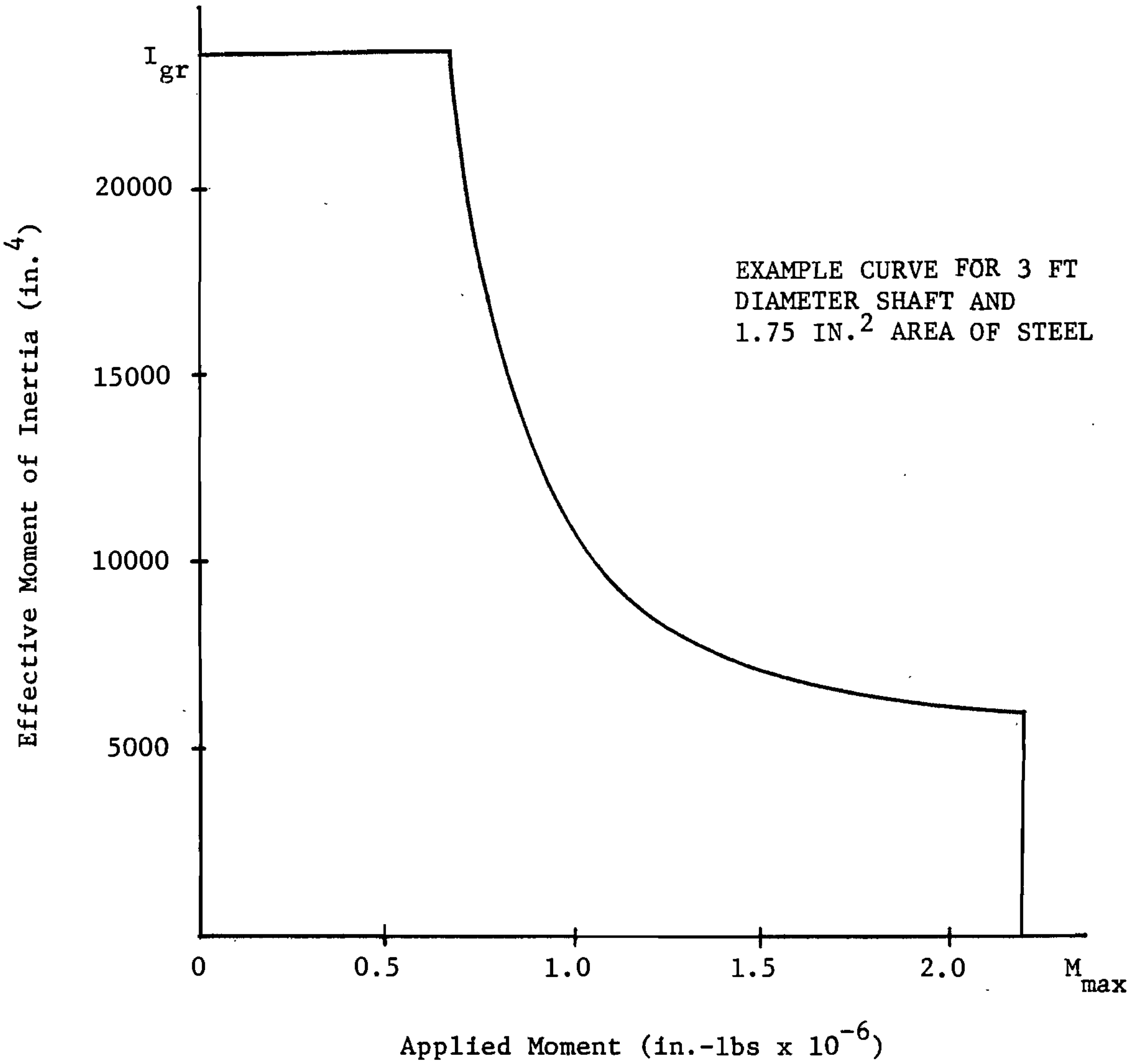


FIG. 5. Combined p-y Curve Below Excavation Level



I_g = 23557 in.^{4}

I_{CR} = 5437 in.^{4}

M_{CR} = .675 x 10^6 in.-lbs

FIG. 6. Effective Moment of Inertia vs. Applied Bending Moment

First, a graph of applied bending moment versus effective moment of inertia is plotted and BMCOL7 is run using a constant EI throughout. Using the results of this run, an EI value can be obtained from the M versus EI plot for each segment of the wall and BMCOL7 is rerun using the new EI values. This process is repeated until the EI values input correspond to the values from the M versus EI plot. By guessing the applied moments and running four or five trials at once, a reasonable answer can be achieved in three iterations. BMCOL7 has the capability to run many problems at once using the same p-y curves with a minimal amount of input. Note that when steel sheet piling is used, EI remains constant up to yielding of the steel.

3.2 Parametric Analysis

A parametric analysis was done on the wall at Liberty and Mesa in Houston with respect to the embedment depth, the wall stiffness, the soil internal friction angle, and the slope of the soil p-y curve. This wall is made of 60-foot long, three-foot diameter, drilled shafts spaced 3.5 feet center to center. The wall has a stiffness of 9.42×10^{10} lb-in.^{2} per foot of wall width. The conventional p-y curves described in the previous section were used with a friction angle of 30°, a unit weight of 120 pcf, and a coefficient of at rest earth pressure of 0.5. The cases studied are summarized in Table 1. Figure 7 shows an example set of p-y curves for one of the cases studied.

An examination of the results (Fig. 8) shows that the slope of the p-y curve has a minor effect on the total displacement at the top of the wall. The reason is that along most of the wall the deflections are large enough to mobilize the full active and passive resistance of

	SOIL		WALL	
CASE NO.	FRICTION ANGLE, ϕ(°) (AT DEPTH = 22')	SLOPE OF P-Y CURVE, K (PSI/IN.)	EMBEDMENT DEPTH (FT)	STIFFNESS, EI LB-IN.^{2} PER FT OF WALL LENGTH
1	25	100	37.5	9.42×10^{10}
2	30	100		
3	35	100		
4	30	50		
5		100		
6		200	37.5	
7		100	22.5	
8			24	
9			25.5	
10			27	
11			30	
12			37.5	
13			45	
14			48	
15			37.5	2.68×10^{10}
16				5.4×10^{10}
17				9.42×10^{10}
18				15.4×10^{10}
19	30	100	37.5	23.35×10^{10}

TABLE 1: Summary of Cases Studies

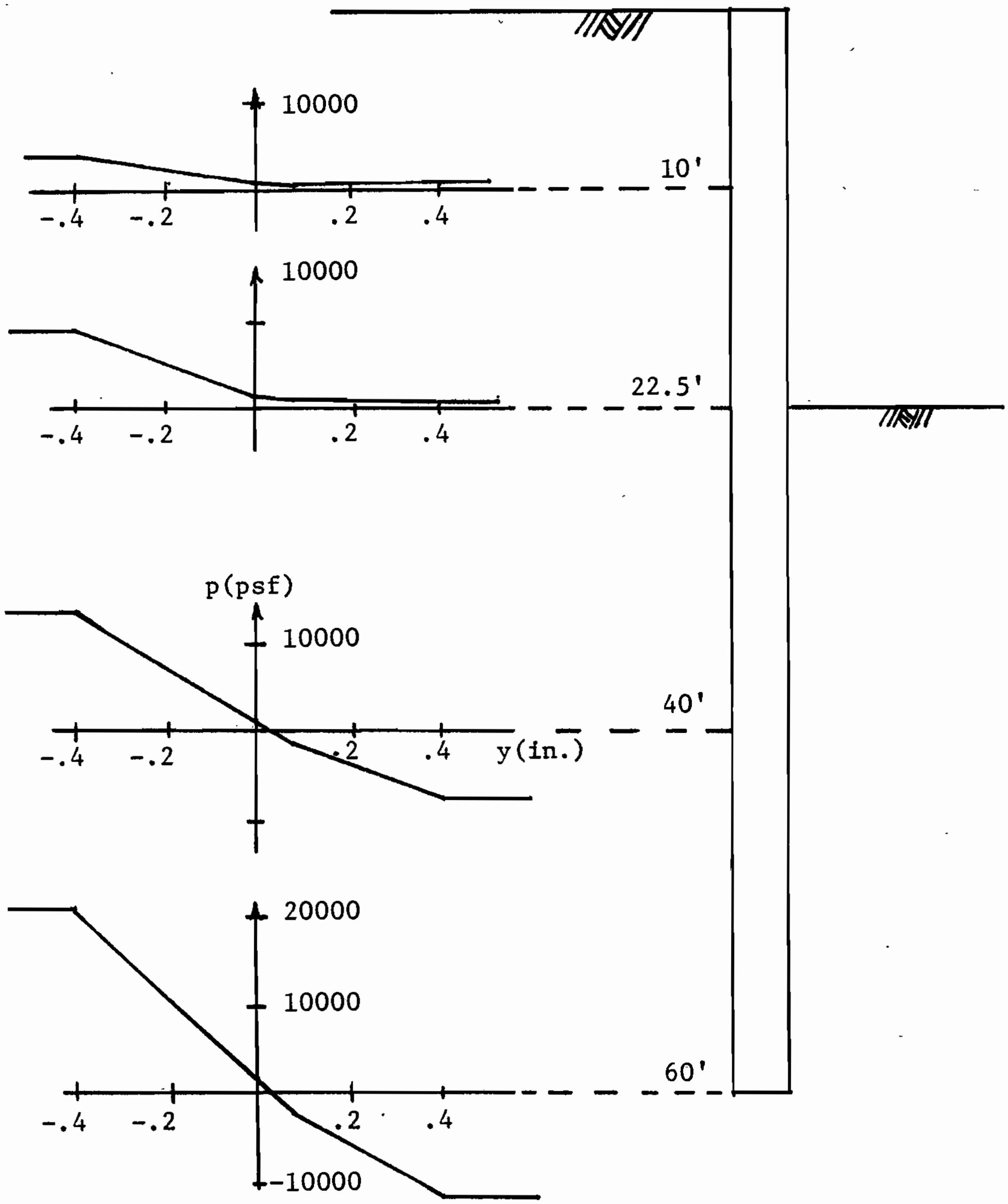


FIG. 7. Case 2 of the Parametric Analysis

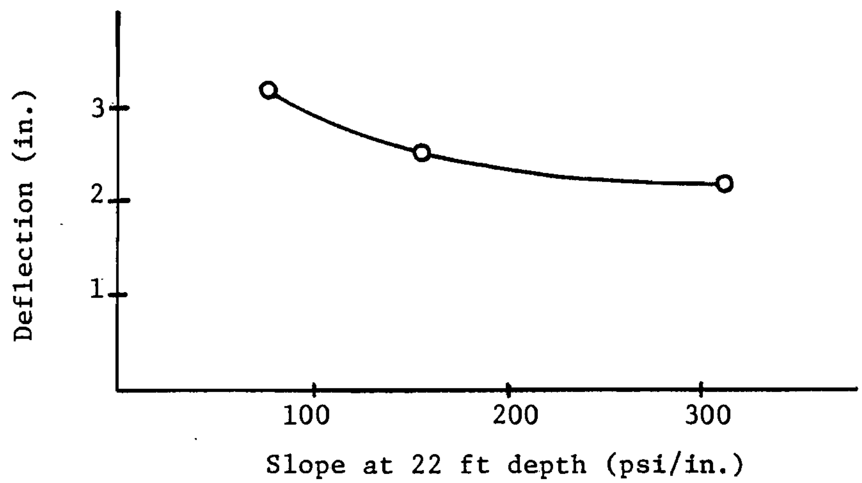


FIG. 8. Influence of the Slope of the p-y Curve on the Wall Top Deflection

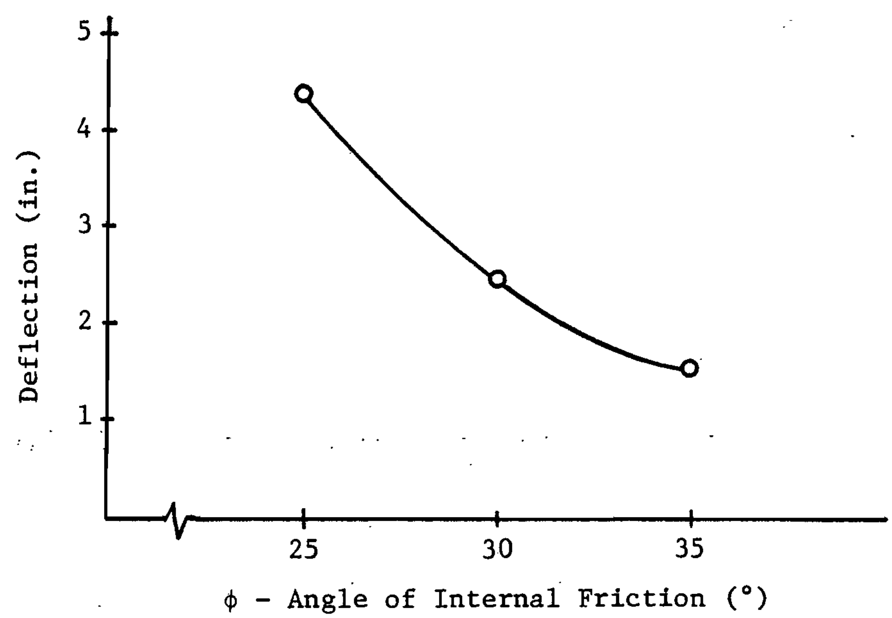


FIG. 9. Influence of φ on the Wall Top Deflection

the soil.

The second soil parameter, the angle of internal friction, has a major effect on the displacement (Fig. 9). The reason is that the active and passive pressures are directly dependent on the angle of internal friction: decreasing ϕ increases the active pressure and decreases the passive pressure.

The effects of the wall properties are shown on Figures 10 and 11. Common sense indicates that a stiff wall will deflect less than a more flexible one. The graph of stiffness versus deflection (Fig. 10) indicates that it is not cost efficient to increase the wall stiffness beyond a certain value.

Figure 11 illustrates the diminishing effect of increasing the wall embedment depth. Beyond a certain point, increasing the embedment depth has no effect on the wall deflection. The wall is essentially fixed and the soil pressures on either side of the wall are nearly equal below the point of fixity.

Varying the stiffness along the cross-section of the wall had a significant effect on the deflection. To reflect the reduction in stiffness due to cracking, the wall was tested with the middle third having a stiffness equal to about one fourth of the regular stiffness. The maximum deflection was nearly double that of the uncracked wall. The stiffness reduction was based on the ACI code and involved an assumption of the amount of reinforcing steel present in the wall. The results are shown in Appendix A as are the detailed results for each case of the parametric analysis.

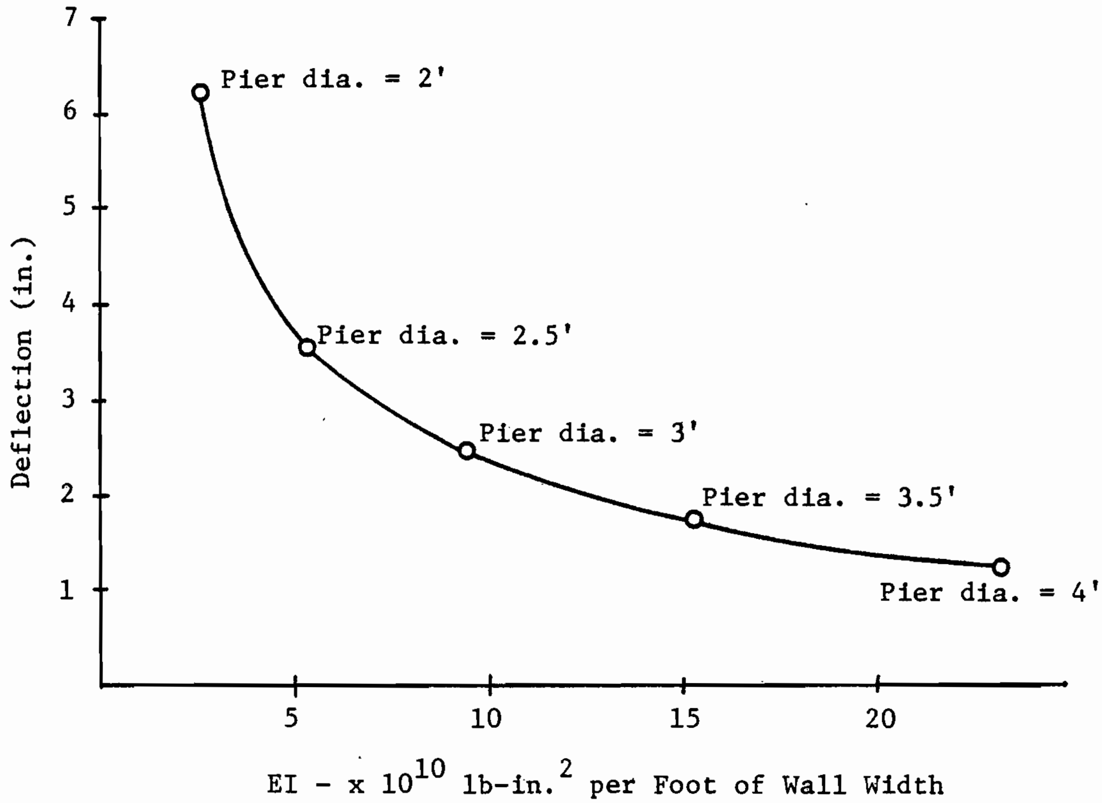


FIG. 10. Influence of Wall Stiffness on Wall Top Deflection

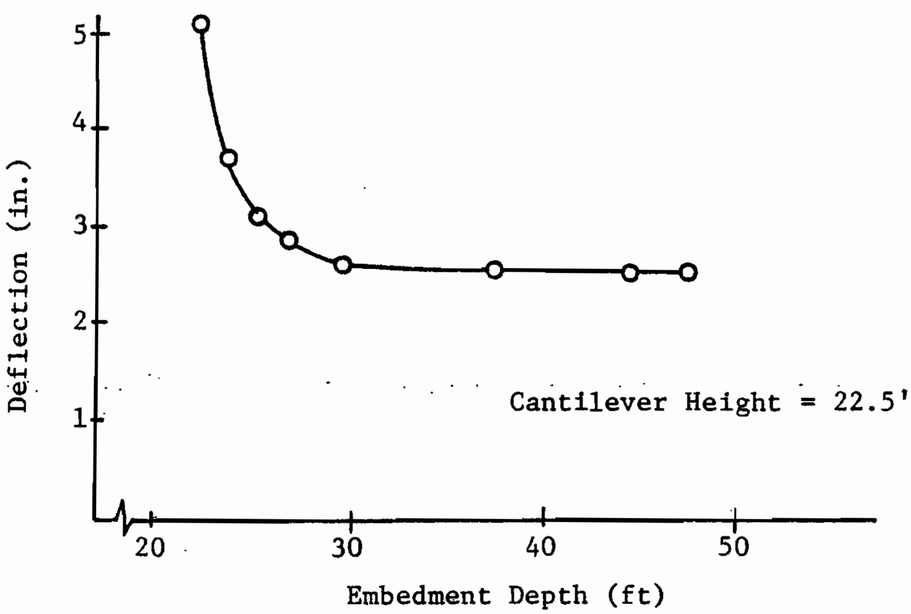


FIG. 11. Influence of Embedment Depth on Wall Top Deflection

CHAPTER 4. THE MENARD PRESSUREMETER METHOD

4.1 Menard's Modulus of Horizontal Subgrade Reaction

The pressure of the soil on a retaining wall can be assumed to be proportional to the horizontal displacement of the wall (Fig. 12) where the constant of proportionality, K, is called the modulus of horizontal subgrade reaction. A value for K can be found from data obtained by a pressuremeter, an in situ testing device described in Chapter 5. Menard (1, 2, 3) gives the following equation for K in tons per cubic foot for a concrete or sheet-pile wall, below excavation level

$$\frac{1}{K} = \frac{1}{E_M} [\frac{\alpha}{2} a + \frac{13}{30.48} (0.09 \times 30.48 \times a)^{\alpha}]$$

where E_M = the arithmetic average of the pressuremeter soil modulus in tons per square foot over the upper two-thirds of the considered embedded length, h (Fig. 13),

a = two-thirds of h in feet, and

α = a dimensionless coefficient (Table 2) depending on E_M and on the limit pressure of the pressuremeter test.

It can be seen that this equation is correct from the point of view of dimensions when $\alpha = 1$. The coefficient α was introduced by Menard in order to match a data set which he has not described in the referenced publications.

4.2 Ultimate Value

The modulus of horizontal subgrade reaction, K, is actually the slope of the p-y curve in the elastic region. In order to get a

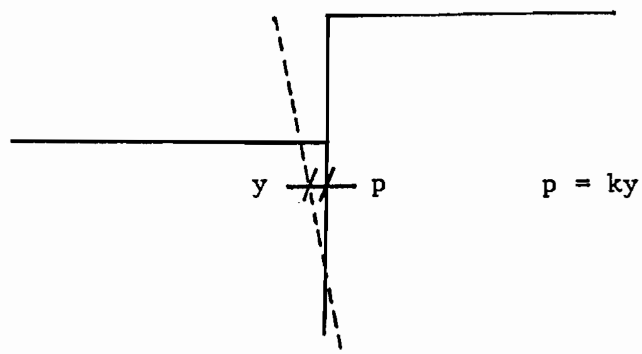


FIG. 12. Assumed Pressure-displacement Relationship

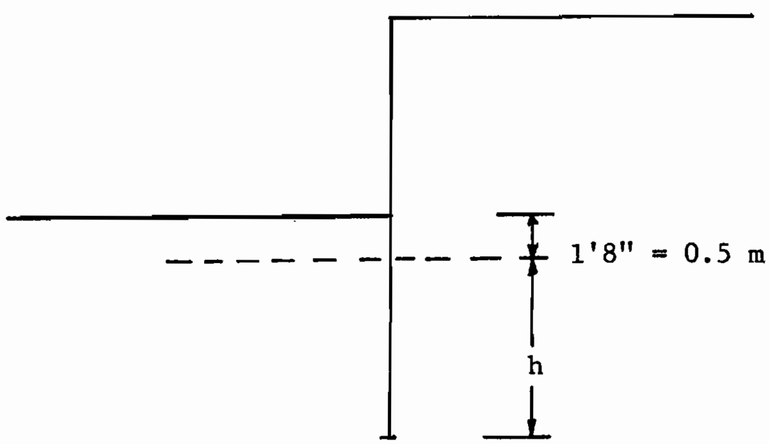


FIG. 13. Definition of Embedment Length

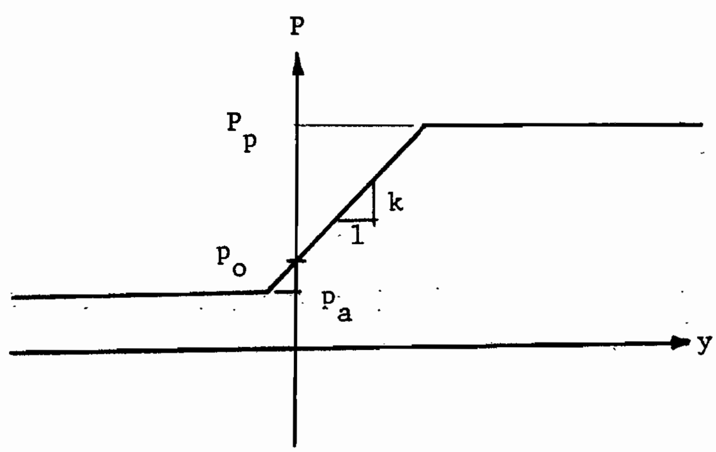


FIG. 14. Pressure-deflection Curve

Degree of Consolidation	Clay		Silt		Sand		Sand and Gravel		Peat	Rock	
	$\frac{E}{p\ell}$	α	$\frac{E}{p\ell}$	α	$\frac{E}{p\ell}$	α	$\frac{E}{p\ell}$	α		Condition	α
Over-consolidated	16	1	14	2/3	12	1/2	10	1/3	1	Very Fractured	1/3
Normally Consolidated	9-16	2/3	8-14	1/2	7-12	1/3	6-10	1/4	1	Normally Fractured	1/2
Under-consolidated or Weathered	9	1/2	8	1/2	7	1/3	6	1/4	1	Not Fractured or Very Weathered	2/3

TABLE 2: Recommended α Values

$E/p\ell$	10	15
Clay	2.7	3.2
Sand	3.5	4.2

TABLE 3: Recommended K_B Values

complete p-y curve, the ultimate values of the active and passive pressures versus deflection are needed.

Menard (1, 2, 3) gives the following equation for the ultimate passive resistance of the soil:

$$p_p = \frac{p_\ell - p_o}{K_B} + p_o$$

where p_ℓ = the limit pressure of the pressuremeter test,

p_o = the at rest pressure in the soil, and

K_B = a dimensionless coefficient dependent upon E_M and p_ℓ (Table 3).

Given the modulus of subgrade reaction and assuming that the at rest coefficient of earth pressure and the coefficient of active earth pressure are one-half and one-third, respectively, a complete p-y curve can be constructed (Fig. 14).

CHAPTER 5. CASE HISTORIES

5.1 Houston Wall: Liberty and Mesa

5.1.1 Purpose and Scope

A geotechnical investigation was undertaken as part of the evaluation of the foundation conditions for the retaining walls of a railroad underpass. The site is located close to the intersection of Liberty Road and Mesa Road (FM 527) in Houston, Texas (Fig. 15). Each retaining wall will be made of a line of drilled piers, 3 feet in diameter with a 42 inches spacing center to center. At the final stage of construction, those piers will have a total length of 60 feet and retain 22.5 feet of soil (Fig. 16).

The work consisted of performing pressuremeter tests at the site in order to obtain the soil properties as follows:

- first loading modulus
- reload modulus
- net limit pressure

A total of eight tests were performed on June 29, 1983.

5.1.2 Pressuremeter Testing

The pressuremeter used was a TEXAM model sold by Roctest, Inc. This is a monocell pressuremeter inflated with water. The probe is 70 mm in diameter and has an initial deflated volume of 1380 cm^3. Eight tests and two calibrations were performed. One of the eight test boreholes was too large and the results are not reported.

All tests were performed in the same boring. The hole was drilled using rotary drilling with axial injection of prepared mud with a 4 inches drilling bit down to a level located 3 feet above testing level.

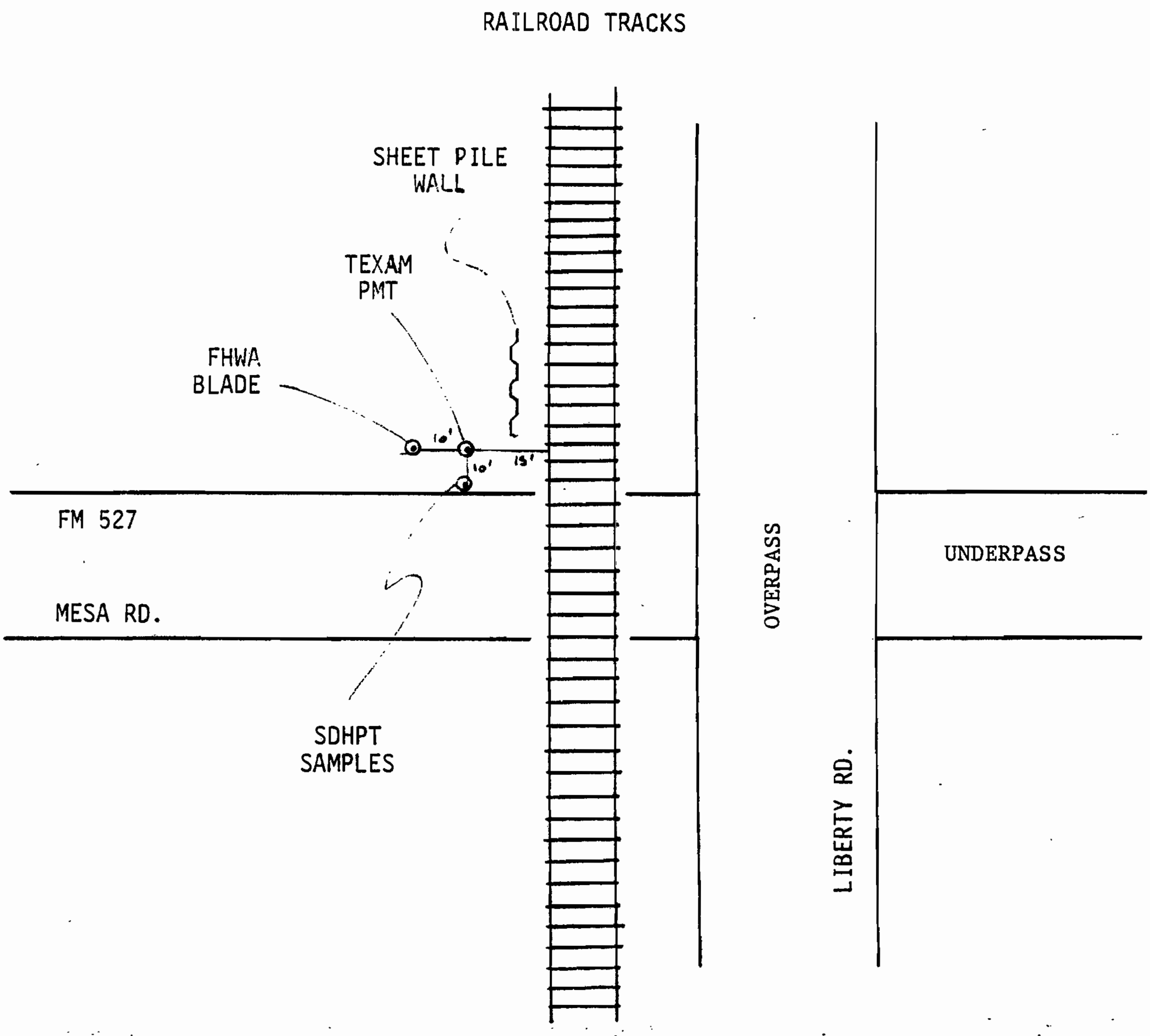


FIG. 15. Location of Pressuremeter Test Hole

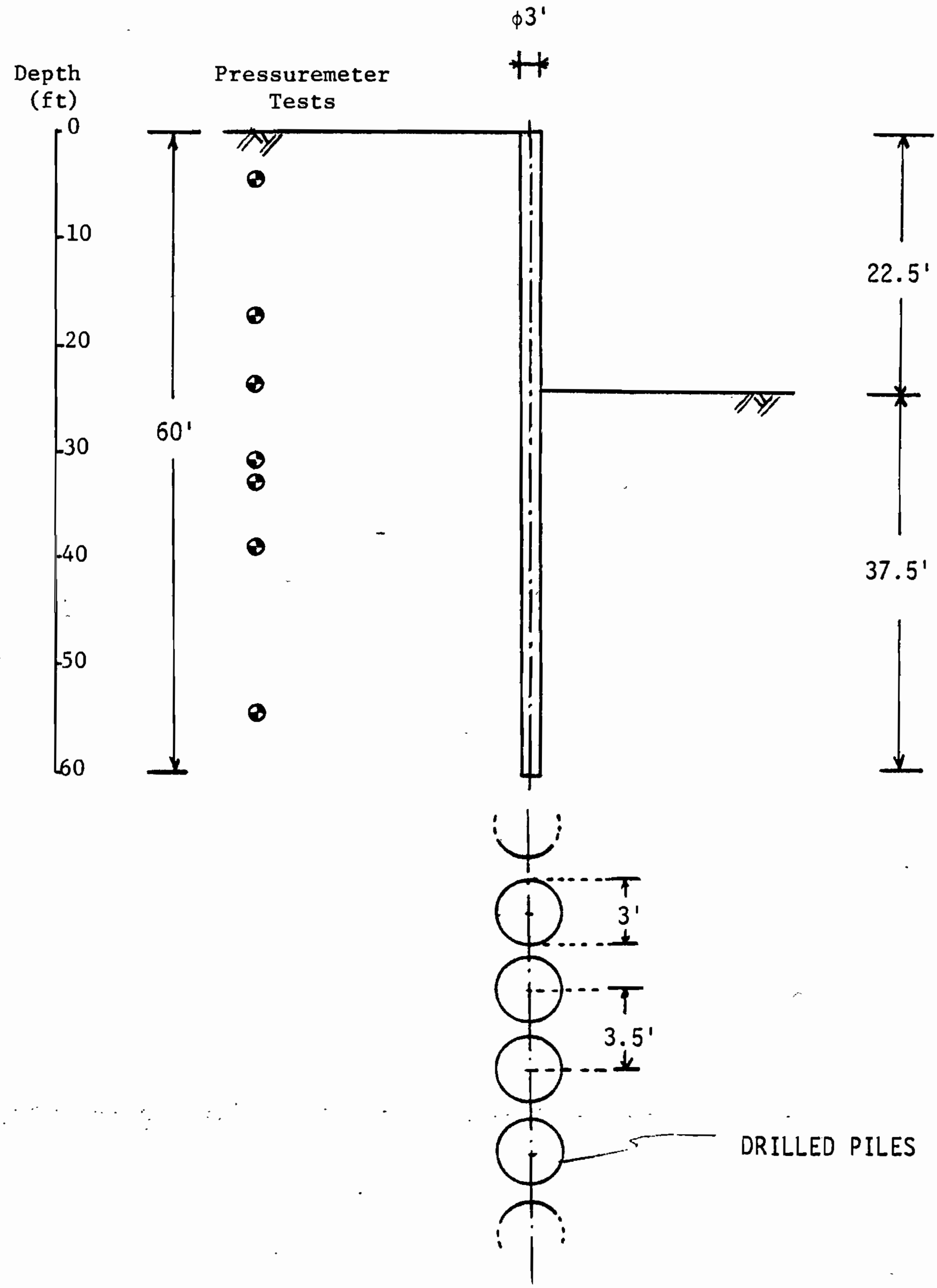


FIG. 16. Depths of Pressuremeter Tests

For the last 3 feet, a 2-15/16 inches bit was used with a slower rotation speed and a lower mud pressure. This procedure gave very satisfactory overall results.

5.1.3 Pressuremeter Results

The raw data obtained in the field was reduced. Corrections were applied for membrane resistance and volume losses in order to obtain the corrected curves.

The corrected p.v. data was then transformed and plotted as a p, $\Delta R/R_o$ curve (Appendix C). The parameter p represents the actual total pressure against the wall of the borehole, ΔR is the increase in probe radius and R_o the deflated probe radius.

The first load modulus E_o was obtained from the straight part of the pressuremeter curve on the first loading, the reload modulus E_R from the unload-reload cycle. The net limit pressure $p\ell^*$ was obtained by manual extension of the curve. The moduli E_o and E_R were computed assuming a Poisson's ratio of 0.33 in all case. The values of the above parameters are shown on the profiles on Figure 17.

5.2 Houston Wall: West Belt and Kimberly

5.2.1 Purpose and Scope

A geotechnical investigation was undertaken as part of the evaluation of the foundation conditions for the retaining walls of a highway underpass. The site is located at the intersection of the West Belt and Kimberly Lane in Houston, Texas (Fig. 18). Each retaining wall will be made of a line of drilled piers, 3 feet in diameter with a 42 inches spacing center to center. At the final stage of construction, those

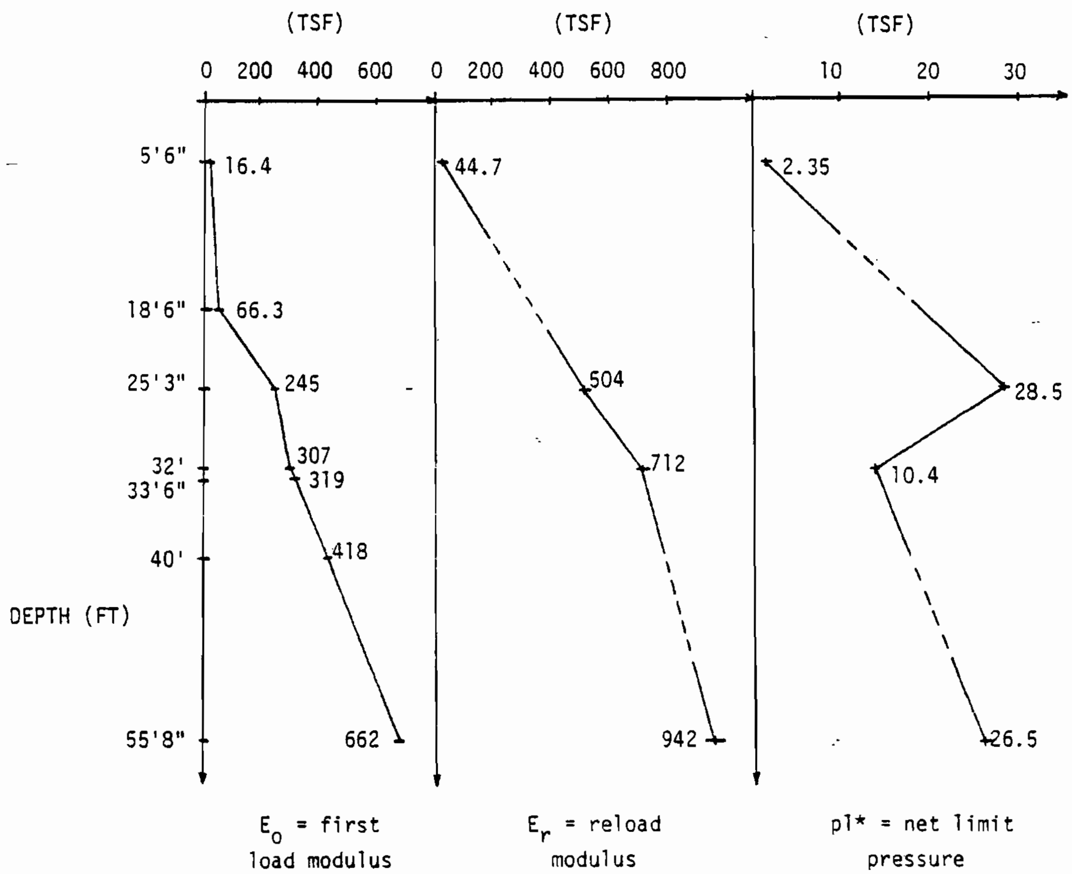


FIG. 17. - Pressuremeter Test Results: Profiles

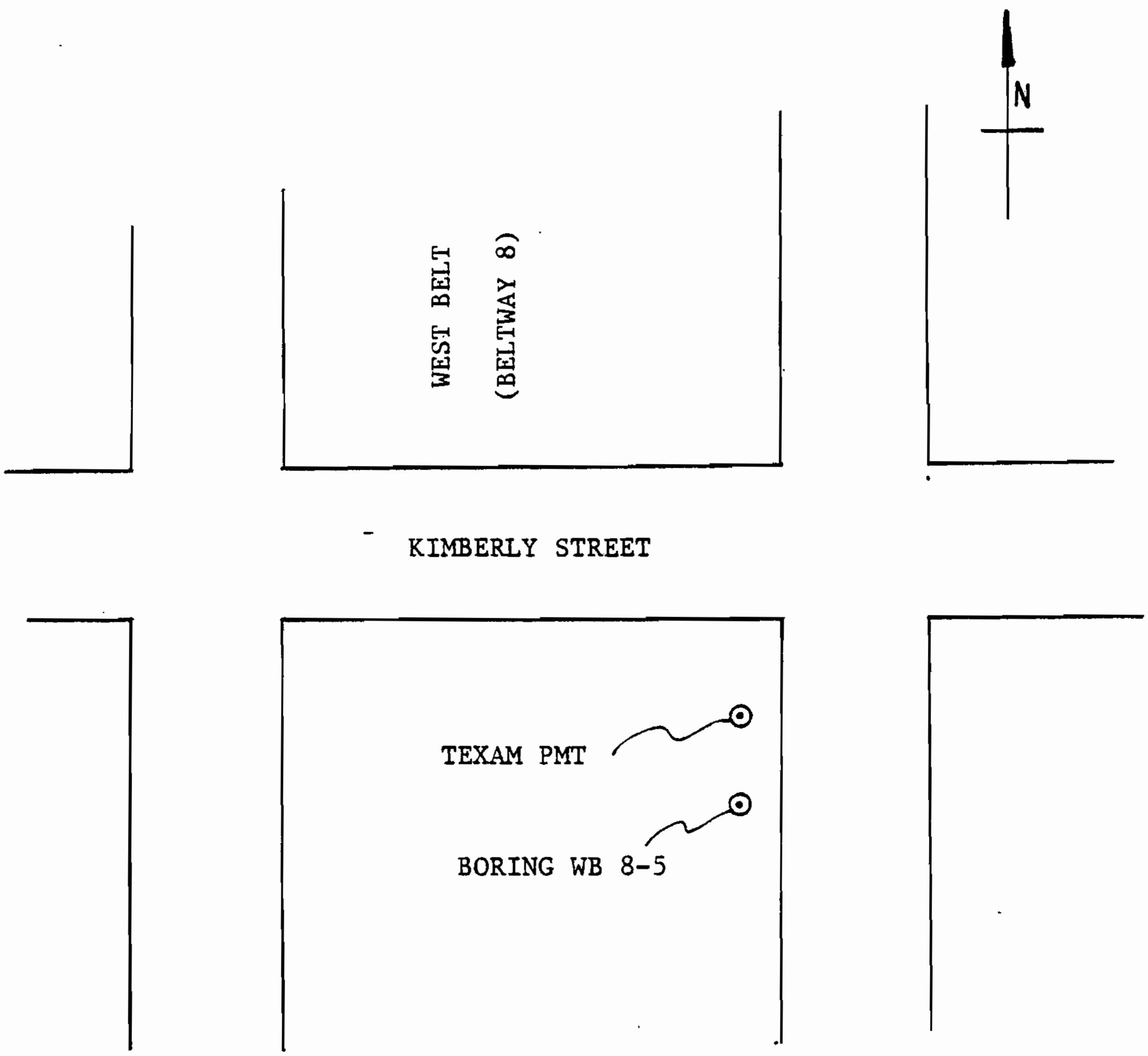


FIG. 18. Location of Pressuremeter Borings

piers will have a total length of 68 feet and retain 22 feet of soil (Fig. 19).

The work consisted of performing pressuremeter tests at the site in order to obtain the soil properties as follows:

- first loading modulus
- reload modulus
- net limit pressure

A total of seven tests were performed on August 31, 1983. Their position is shown on Figure 20 along with the observed soil layers.

5.2.2 Pressuremeter Testing

The pressuremeter used was a TEXAM model sold by Roctest, Inc. This is a monocell pressuremeter inflated with water. The probe is 70 mm in diameter had has an initial deflated volume of 1380 cm^3. Seven tests and two calibrations were performed.

All tests were performed in the same boring. The hole was drilled using rotary drilling with axial injection of prepared mud with a 4 inches drilling bit down to a level located 3 feet above testing level. For the last 3 feet, a 2-15/16 inches bit was used with a slower rotation speed and a lower mud pressure. This procedure gave satisfactory overall results. In the sand layers future drilling might be more successful if a tricone roller bit is used and if the drilling mud is thickened significantly.

5.2.3 Pressuremeter Results

The raw data obtained in the field was reduced. Corrections were applied for membrane resistance and volume losses in order to obtain the corrected curves.

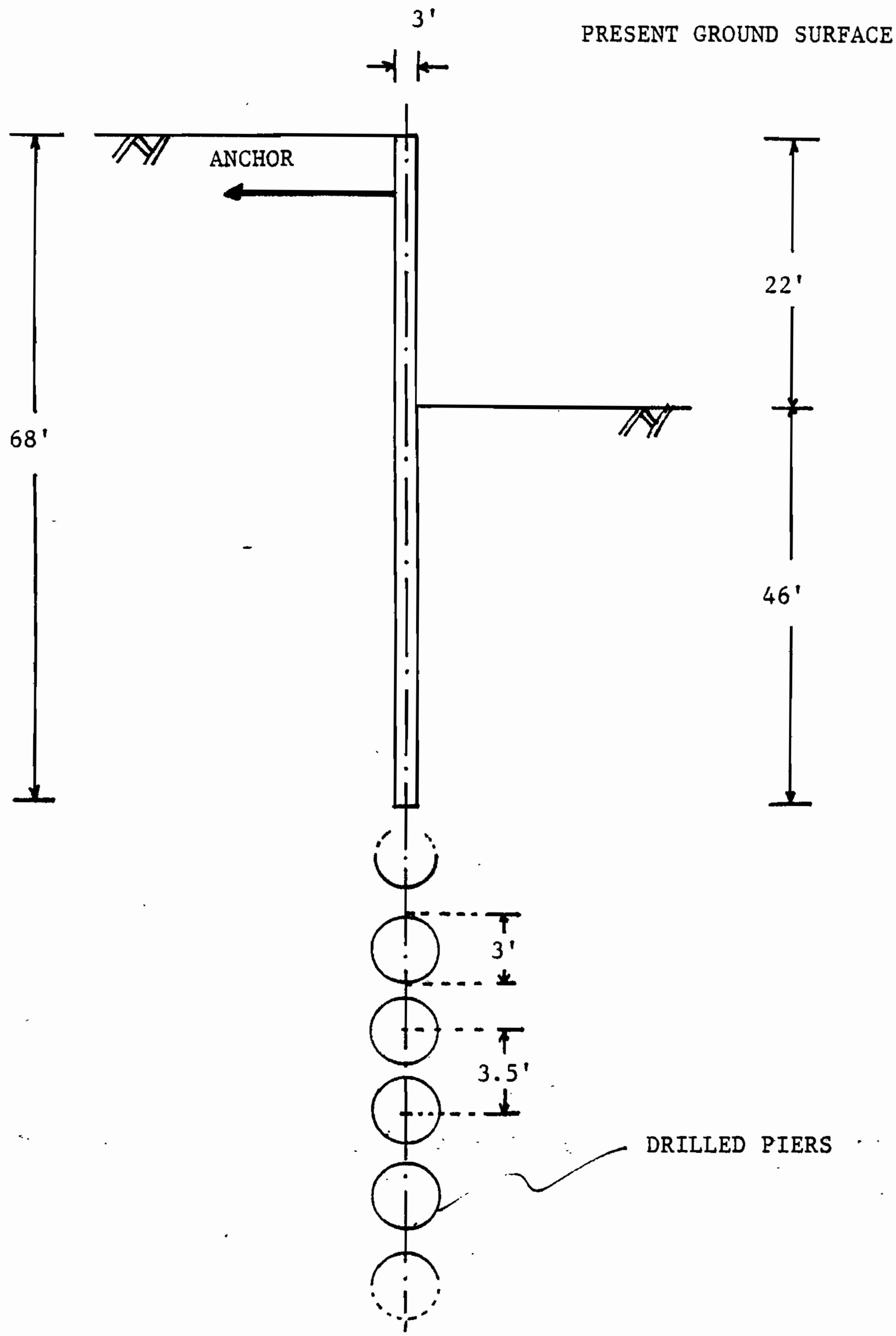


FIG. 19. Wall Cross Section

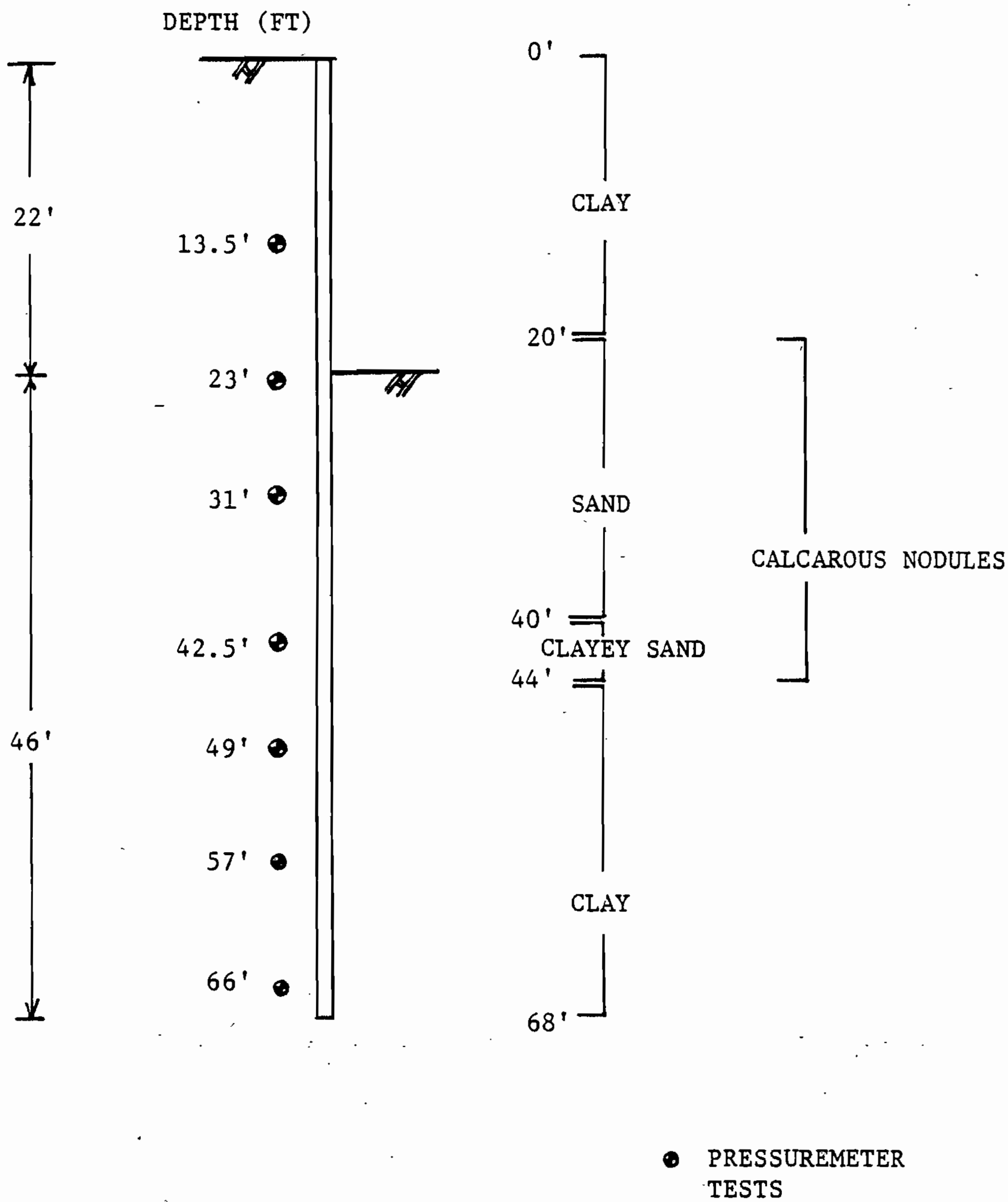


FIG. 20. Depths of Pressuremeter Tests

The corrected p.v. data was then transformed and plotted as a p, $\Delta R/R_o$ curve (Appendix B). The parameter p represents the actual total pressure against the wall of the borehole, ΔR is the increase in probe radius and R_o the deflated probe radius.

The first load modulus E_o was obtained from the straight part of the pressuremeter curve on the first loading, the reload moduli E_{RI} and E_{R2} from the first and second unload-reload cycles, respectively. The net limit pressure $p\ell^*$ was obtained by manual extension of the curve. The moduli E_o, E_{R1} and E_{R2} were computed assuming a Poisson's ratio of 0.33 in all cases. The values of the above parameters are shown on the profiles on Figure 21.

5.3 Assumptions and Analysis

In order to analyze the two retaining walls in Houston, the following assumptions were made:

- Each pier affects a width of soil equal to 3.5 feet.
- The reinforcing steel is such that the cracking moment is not exceeded in each pier.
- The modulus of elasticity for the concrete is four million psi.

For the analysis using conventional p-y curves, the soil properties were assumed as follows:

γ = 120 pcf,

ϕ = 30°,

Displacement needed to develop active pressure is 2 mm.

Displacement needed to develop passive pressure is 10 mm.

For the Menard method, the procedure outlined in Chapter 4 was

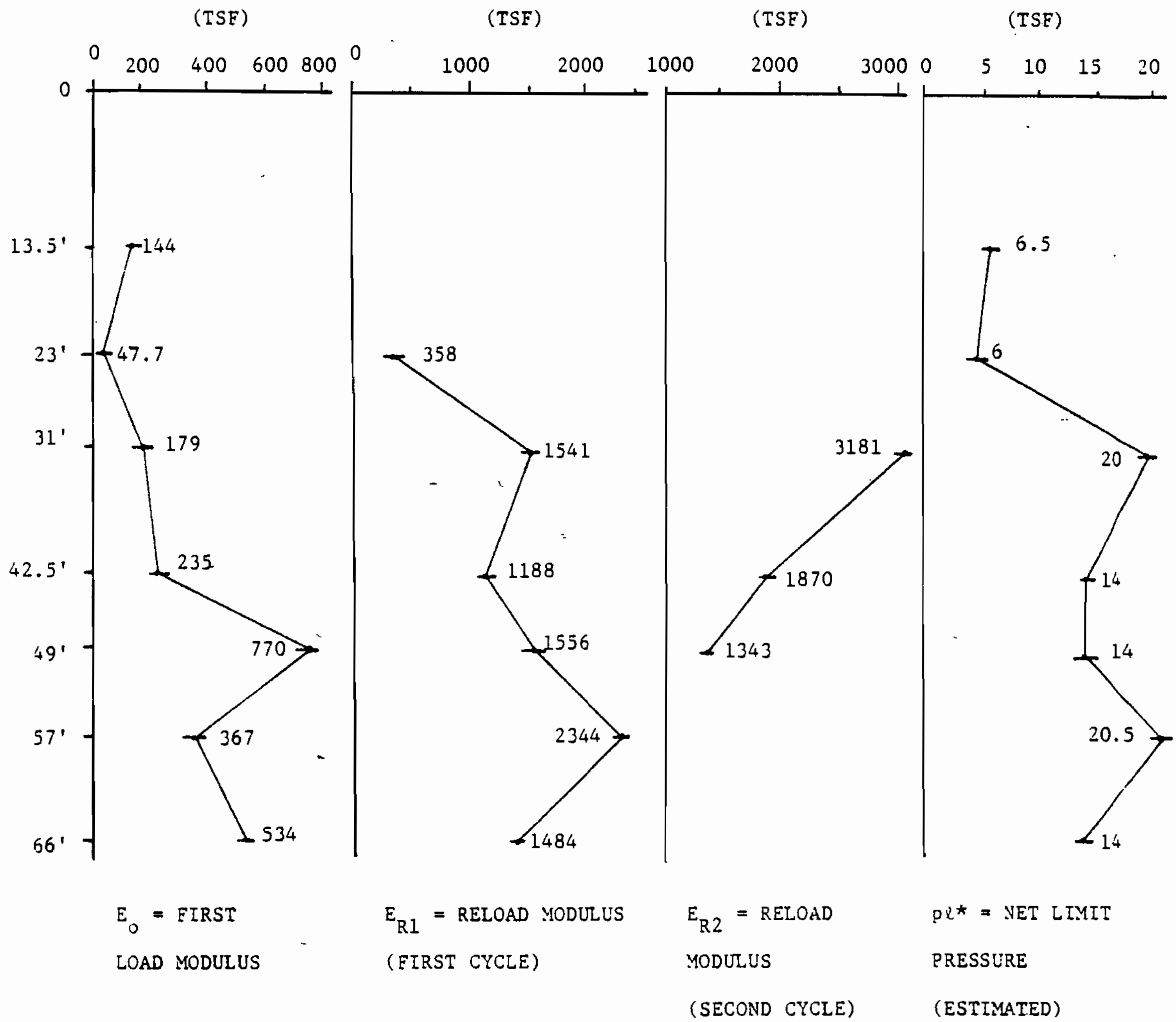
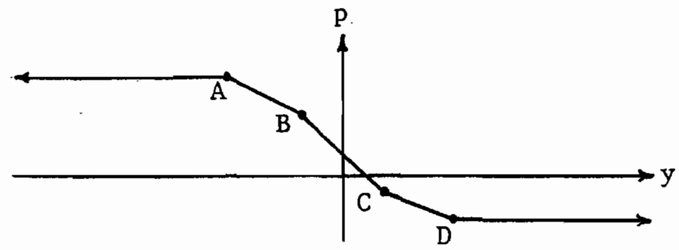


FIG. 21. - Summary of Pressuremeter Results

used below the excavation level. Above the excavation level, the conventional p-y curves were used.

Tables 4 and 5 contain a summary of the p and y coordinates used to analyze the two walls. Z is the depth in feet from the top of the wall, P is the pressure in psf, and Y is the corresponding lateral displacement in feet. A, B, C, and D refer to the four points on the p-y curve shown on Tables 4 and 5.

The results of the analysis are shown on Figures 22 and 23 for both the pressuremeter and conventional method.



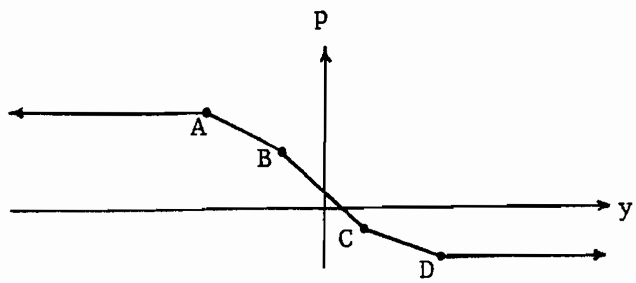
P-Y Coordinates from Conventional Method

Z(ft)	A		B		C		D	
	P(psf)	Y(ft)	P(psf)	Y(ft)	P(psf)	Y(ft)	P(psf)	Y(ft)
0	0	-1000	-	-	-	-	0	1000
22.5	8064	-.033	1296	0	-	-	864	.007
60	20160	-.033	1296	0	-2160	.007	-11088	.033

P-Y Coordinates from Menard Method

Z	A		B		C		D	
	P	Y	P	Y	P	Y	P	Y
0	0	-1000	-	-	-	-	0	1000
23	8280	-.0328	1380	-	-	-	920	.0066
25	22900	-.0337	1350	-.00008	390	.00078	-22100	.0343
32	3720	-.0136	2510	-.00121	20	.00408	- 3120	.0241
56	10460	-.0872	4010	-.00691	850	.01155	- 9860	.10444
60	10460	-.0872	4010	-.00691	850	.01155	- 9860	.10444

TABLE 4: P-y Curve Data for Liberty and Mesa Wall



P-Y Coordinates from Conventional Method

Z	A		B		C		D	
	P	Y	P	Y	P	Y	P	Y
0	0	-1000	-	-	-	-	0	1000
22	7920	-.033	1320	0	-	-	880	.007
68	22644	-.033	1320	0	-2796	.007	9999	.033

P-Y Coordinates from Menard Method

Z	A		B		C		D	
	P	Y	P	Y	P	Y	P	Y
0	0	-1000	-	-	-	-	0	1000
22	7920	-.033	1320	0	-	-	880	.007
31	16640	-.14	1670	-.0016	230	.0056	-9999	.14
42	10000	-.39	2930	-.019	-2100	.04	-7820	.39
57	13800	-.62	4090	-.032	- 940	.06	-9999	.62
68	7280	-.23	4900	-.04	-1354	.06	-5060	.23

TABLE 5: P-y Curve Data for West Belt and Kimberly Wall

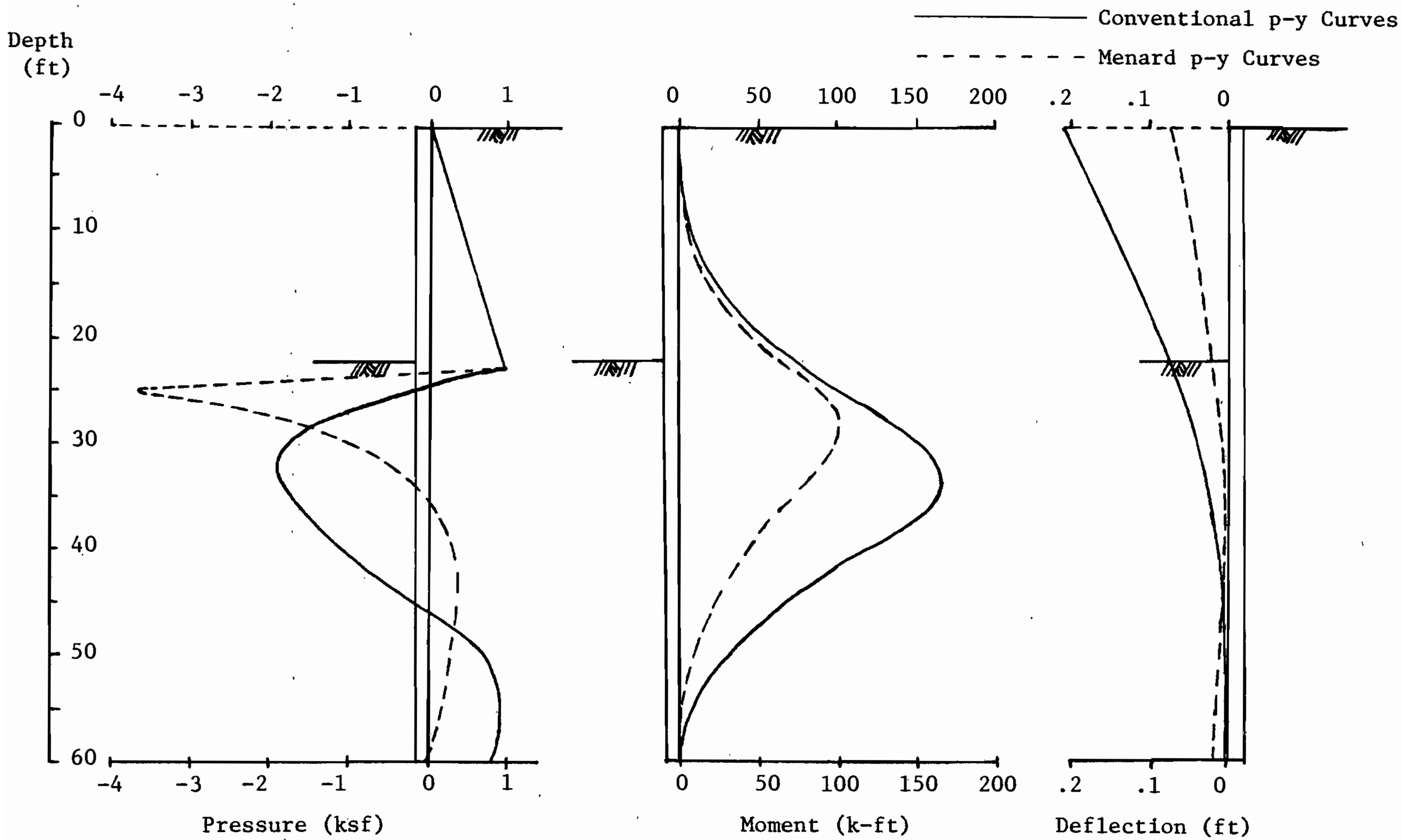


FIG. 22. Results of Analysis for Liberty and Mesa Wall

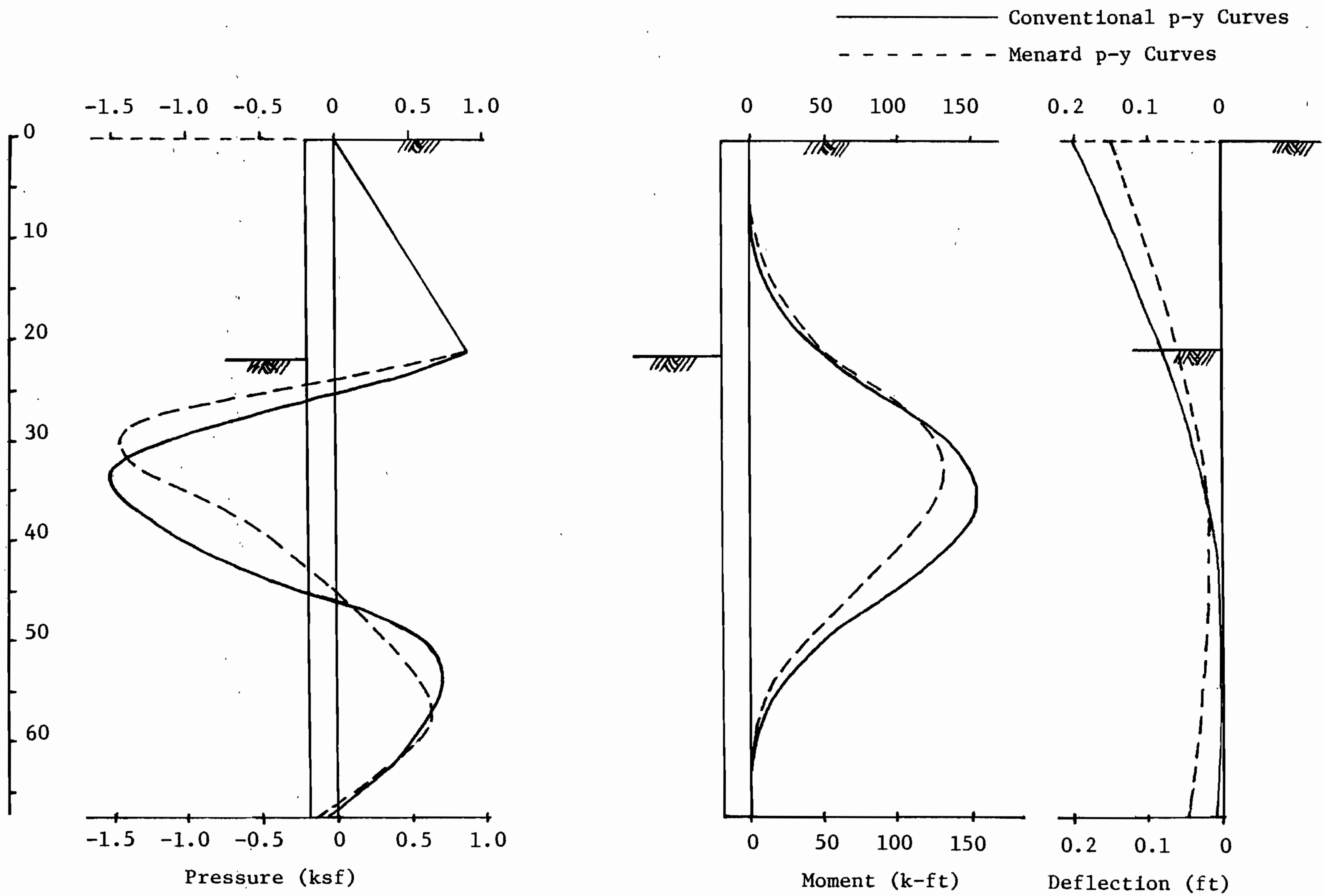


FIG. 23. Results of Analysis for Kimberly and West Belt Wall

CHAPTER 6: CONCLUSIONS

The results of the parametric analysis show that, for the cases studied, the influence of various parameters on the top deflection of the wall is as follows:

1. When the slope of the p-y curve is multiplied by 3, the top deflection of the wall is multiplied by 0.65 and the maximum bending moment is practically unchanged.
2. When the angle of internal friction of the soil is multiplied by 1.40 the top deflection of the wall is multiplied by 0.35 and the maximum bending moment is multiplied by 0.5.
3. When the drilled shaft diameter is multiplied by 2, the top deflection of the wall is multiplied by 0.2 and the maximum bending moment is practically unchanged.
4. Maximum benefit is obtained for an embedment depth equal to 1.4 times the cantilever height (height of retained soil).

The analysis of the two walls in Houston was made using a conventional analysis and the Menard pressuremeter method. The results of the predictions show that, in these two cases:

1. The Menard pressuremeter method predicted 25% and 50% less top deflection of the walls than the conventional method.
2. The Menard pressuremeter method predicted maximum bending moments which were 15% and 39% less than the maximum bending moment predicted by the conventional method.

The above results need to be further investigated at full scale in the field at the time of construction and also at small scale in the laboratory.

CHAPTER 7. REFERENCES

1. Menard, L., Bourdon, G., and Hcuy, A., "Etude experimentale de l'encastrement d'un rideau en fonction des caracteristiques pressiometriques du sol de fondation," Sols-Soils No. 9, pp. 11-27, 1964.

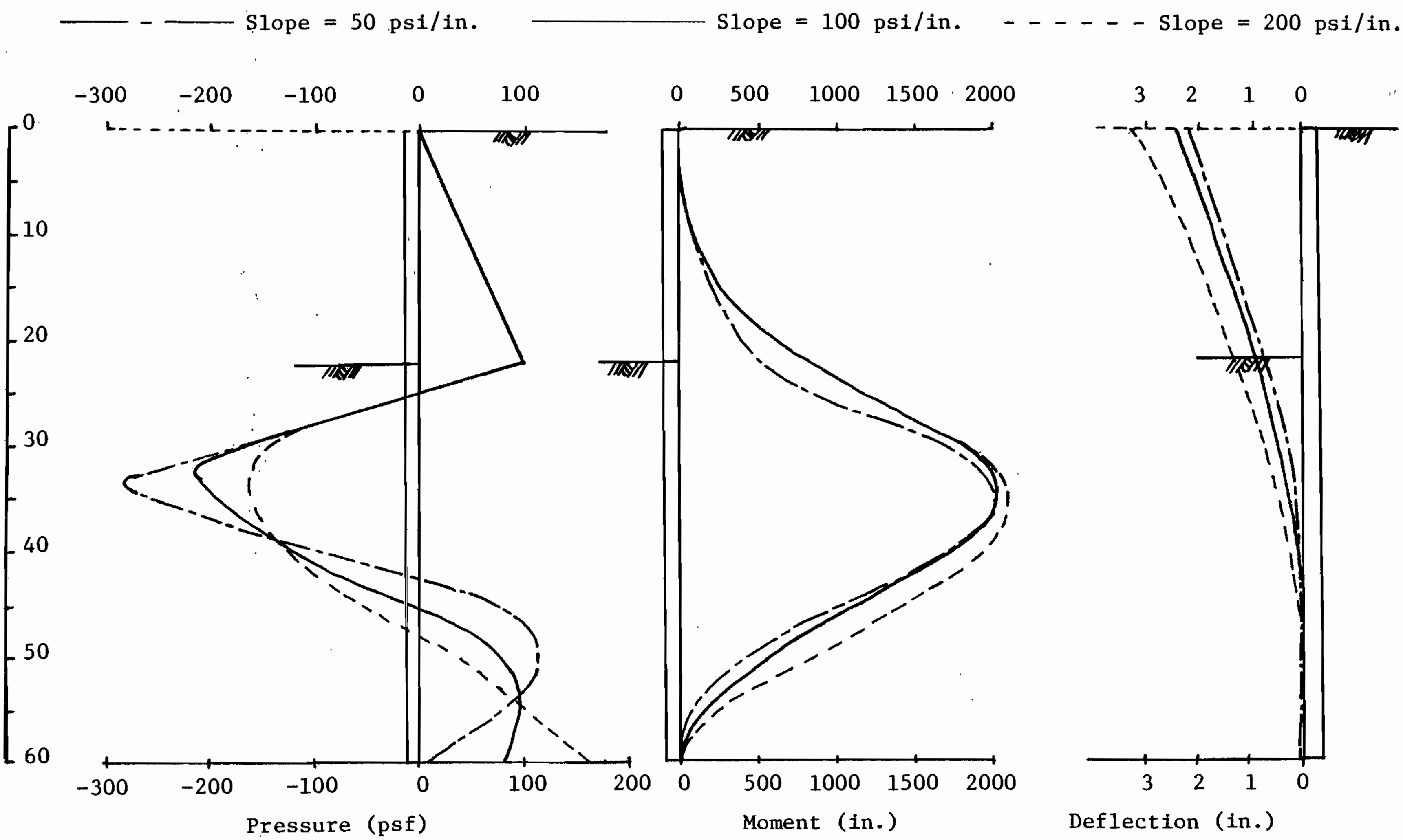
2. Menard, L., and Bourdon, G., "Calcul des rideaux de soutenement: methode nouvelle prenant en compte les conditions reeles d'encastrement," Sols-Soils No. 12, pp. 18-32, 1965.

3. Menard, L., Bourdon, G., and Gambin, M., "Methode generale de calcul d'un rideau ou d'un pieu sollicite horizontalement en fonction des resultats pressiometriques," Sols-Soils No. 22-23, pp. 16-29, 1969.

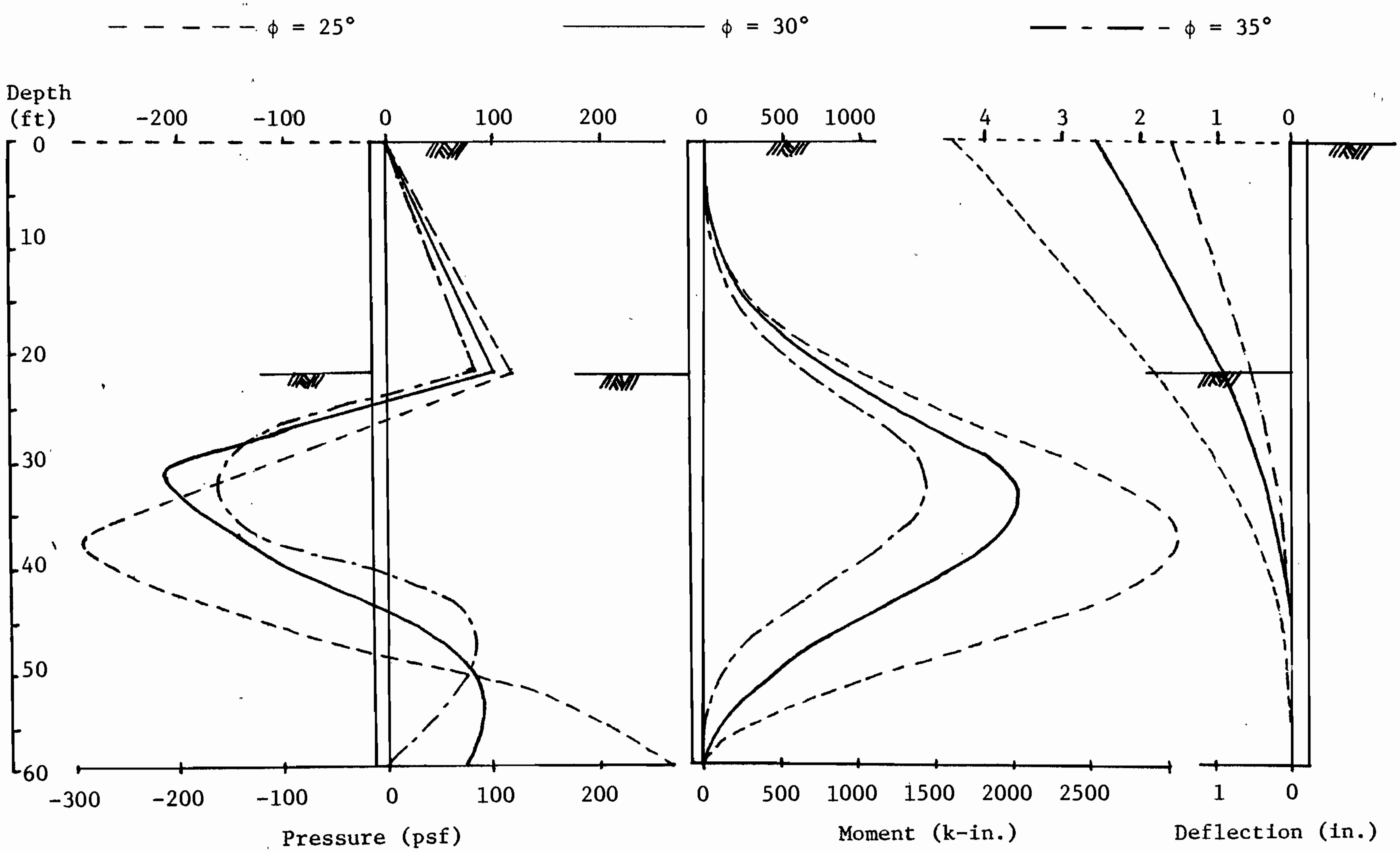
APPENDIX A

RESULTS OF PARAMETRIC STUDY

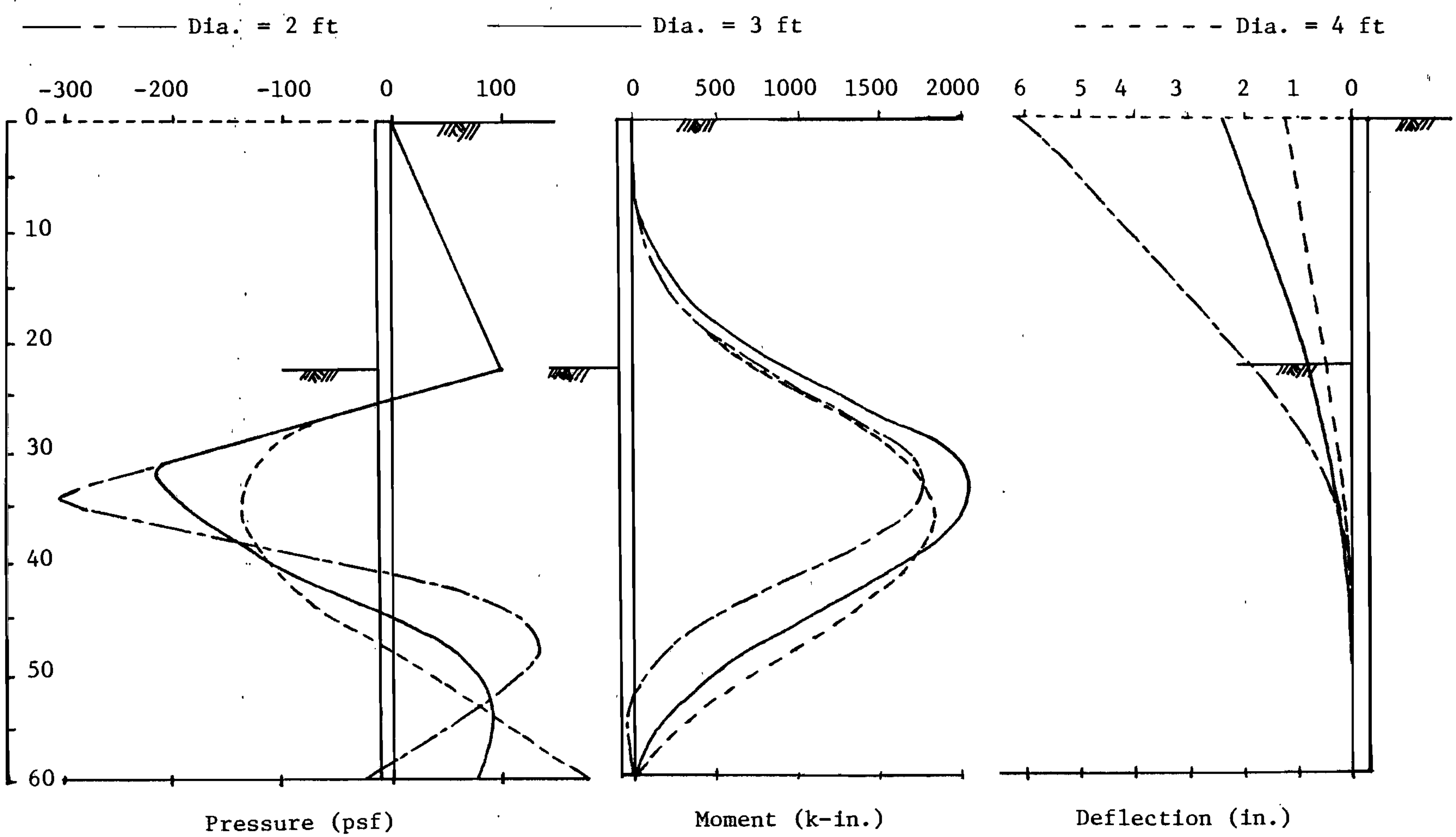
Influence of the Slope of the p-y Curve on the Wall Response



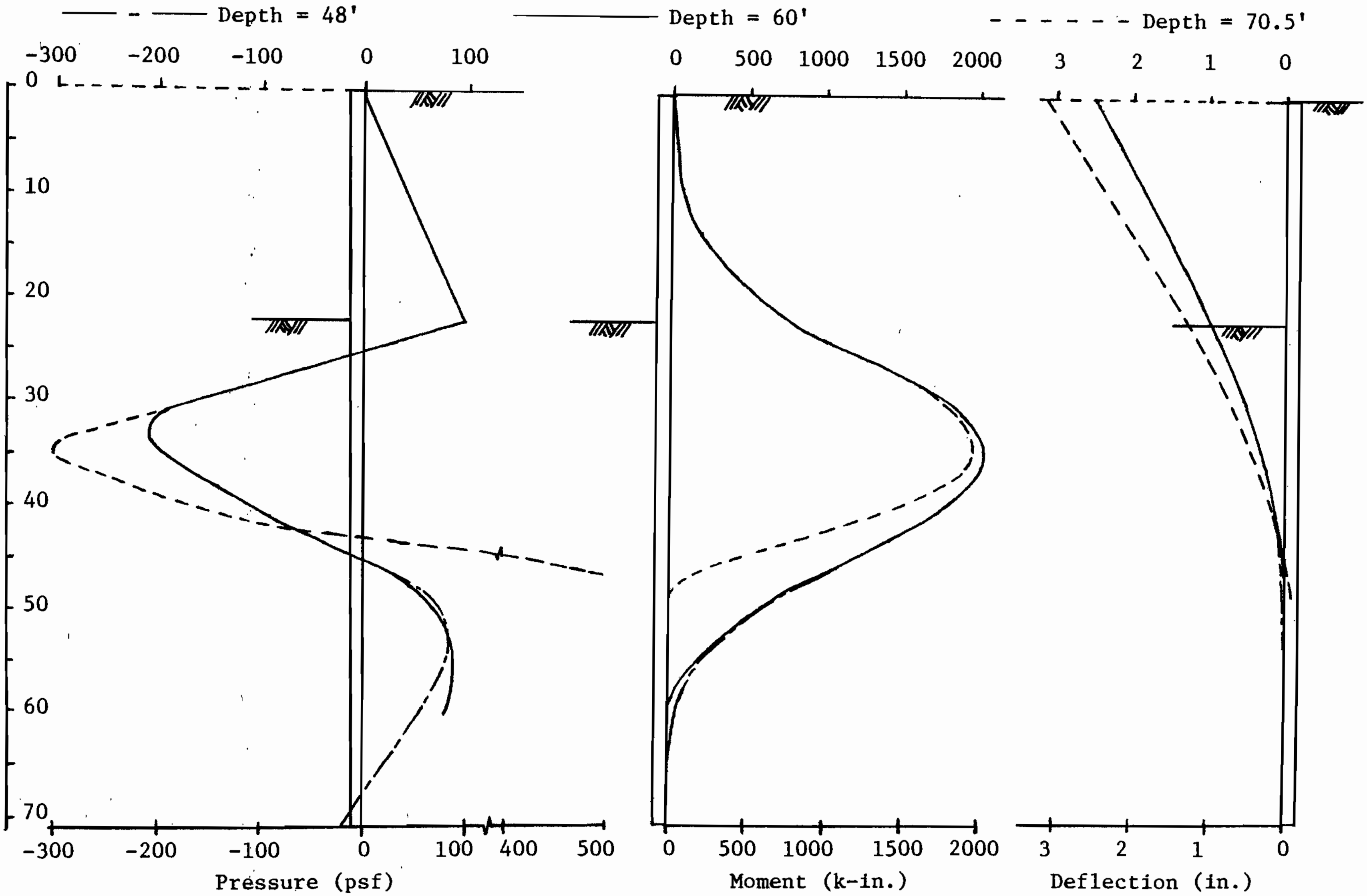
Influence of Angle of Internal Friction on Wall Response



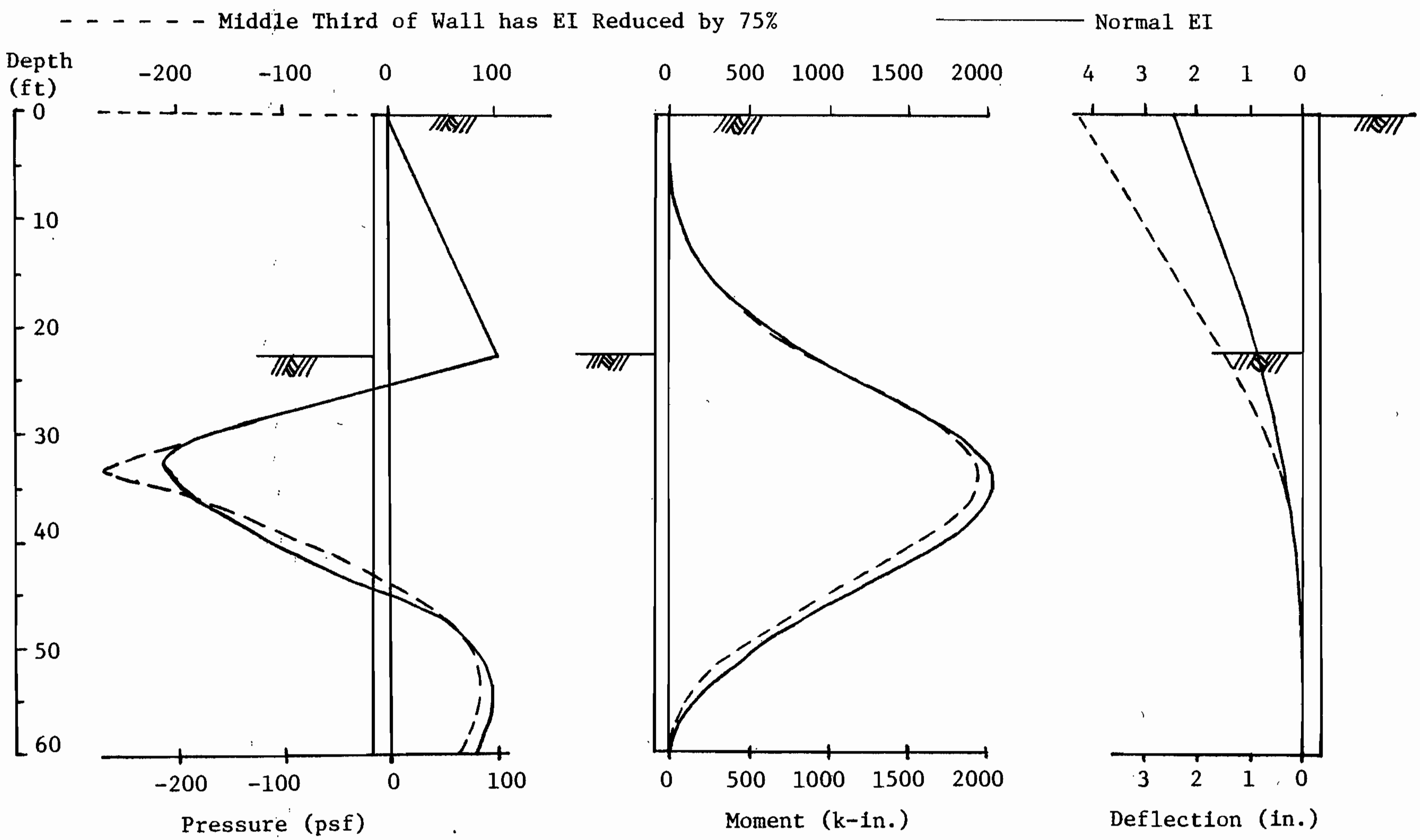
Influence of Pier Diameter on the Wall Response



Influence of Embedment Depth on Wall Response

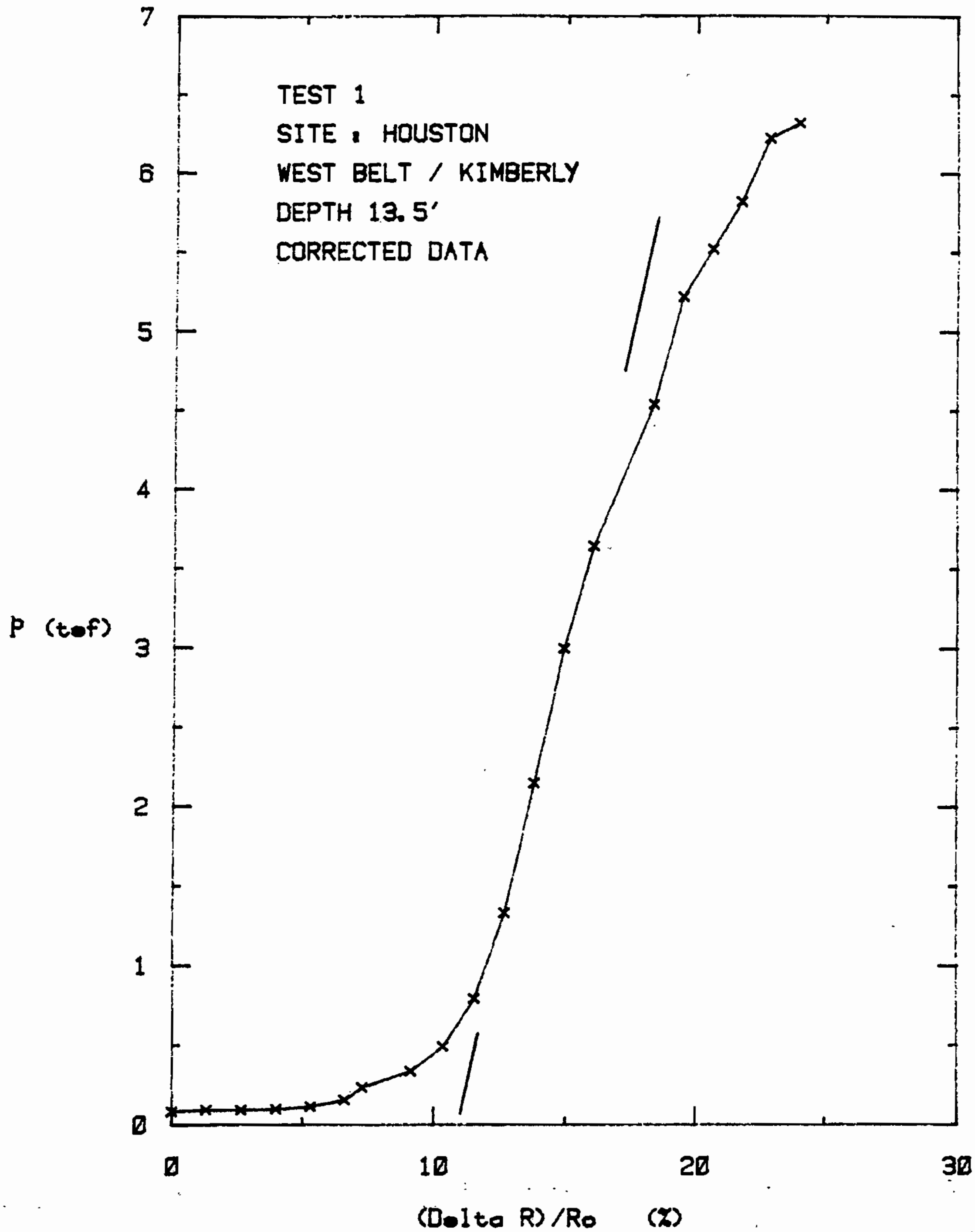


Influence of Variable EI on Wall Response



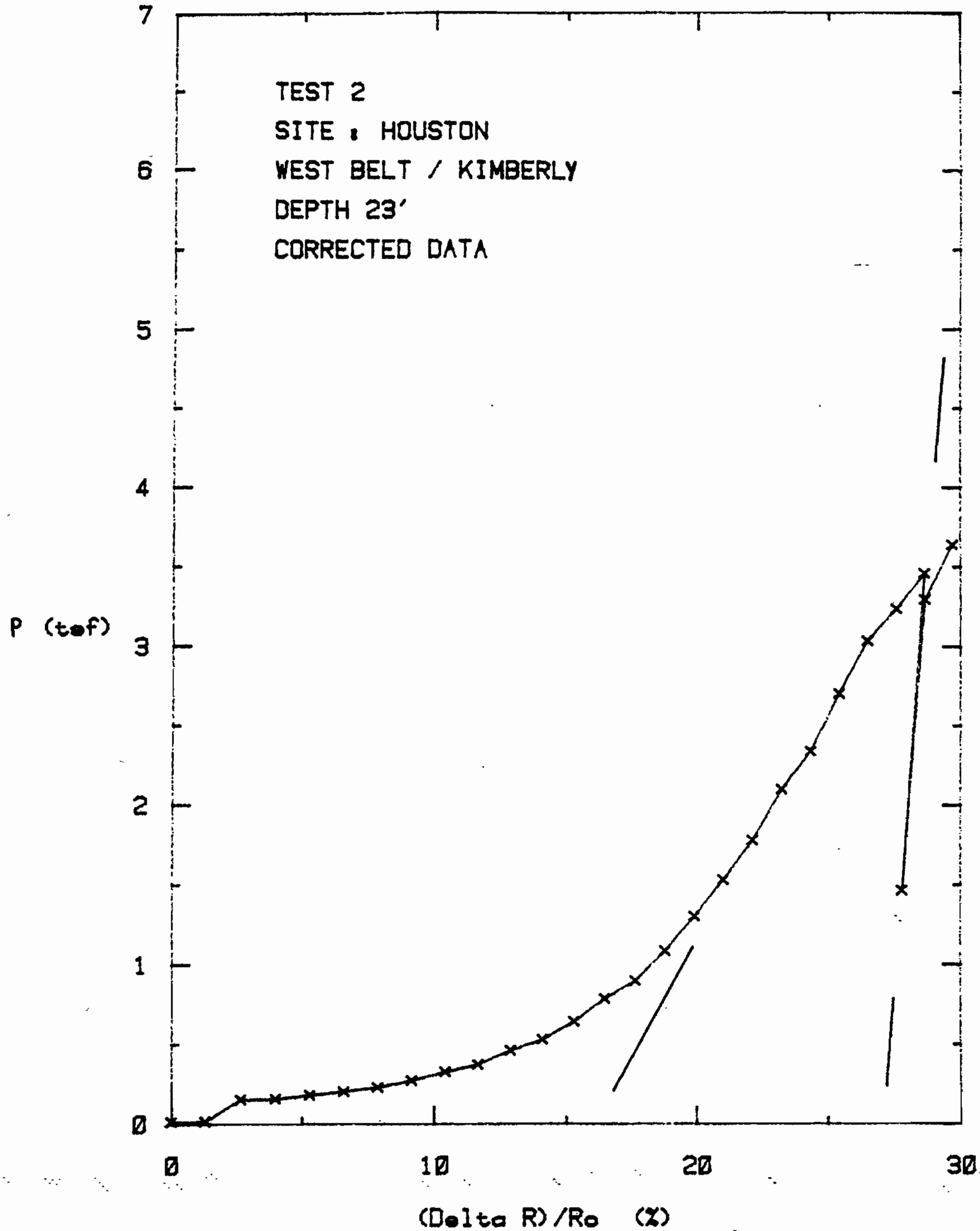
APPENDIX B

SOIL DATA: KIMBERLY AND WEST BELT



E_o = 144 TSF

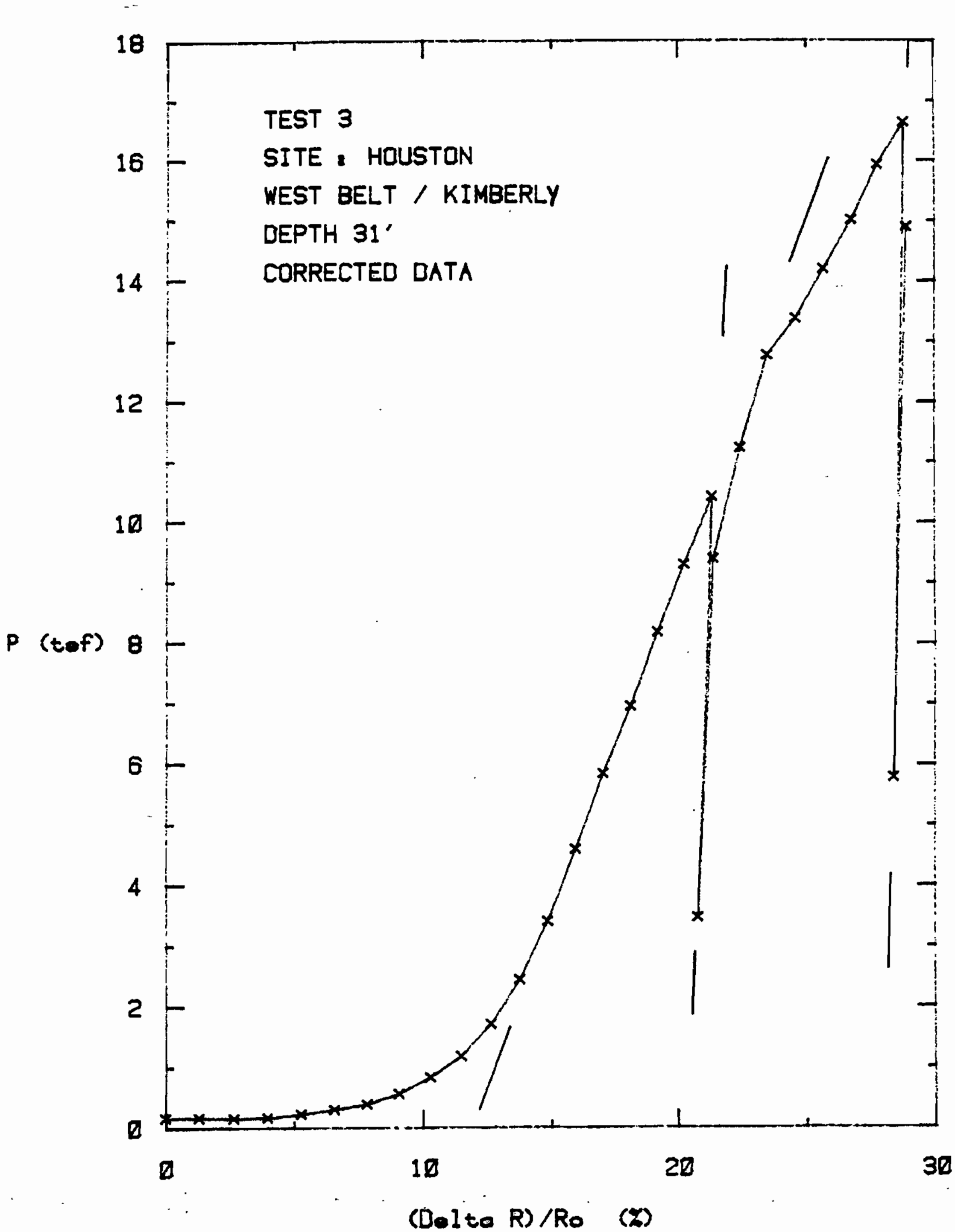
pl* = 6.5 TSF (ESTIMATED)



E_o = 47.7 TSF

E_R = 358 TSF

pl* = 6 TSF (ESTIMATED)

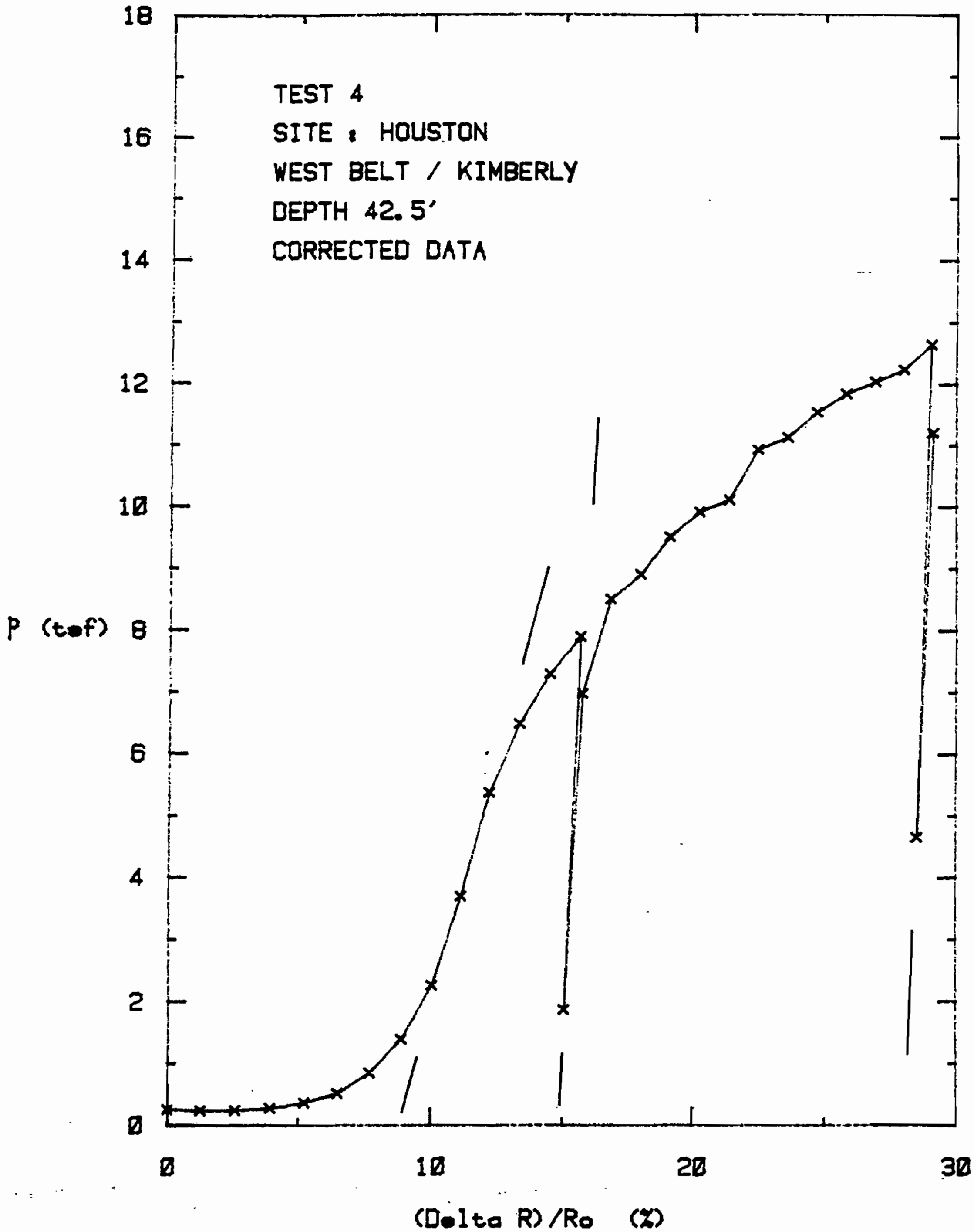


E_o = 179 TSF

E_{R1} = 1541 TSF

E_{R2} = 3181 TSF

pl* = 20 TSF (ESTIMATED)

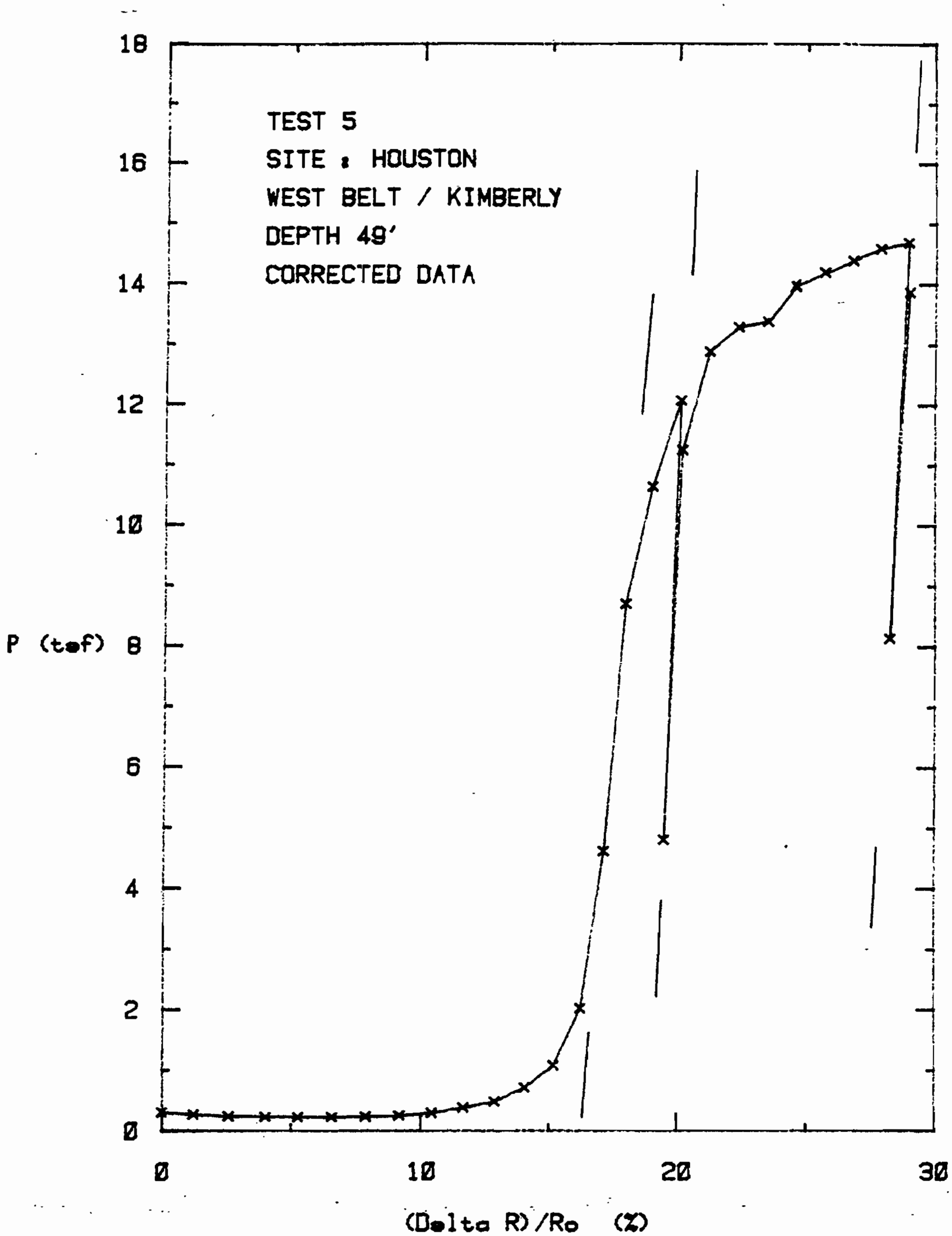


E_o = 235 TSF

E_{R1} = 1188 TSF

E_{R2} = 1870 TSF

pl* = 14 TSF (ESTIMATED)

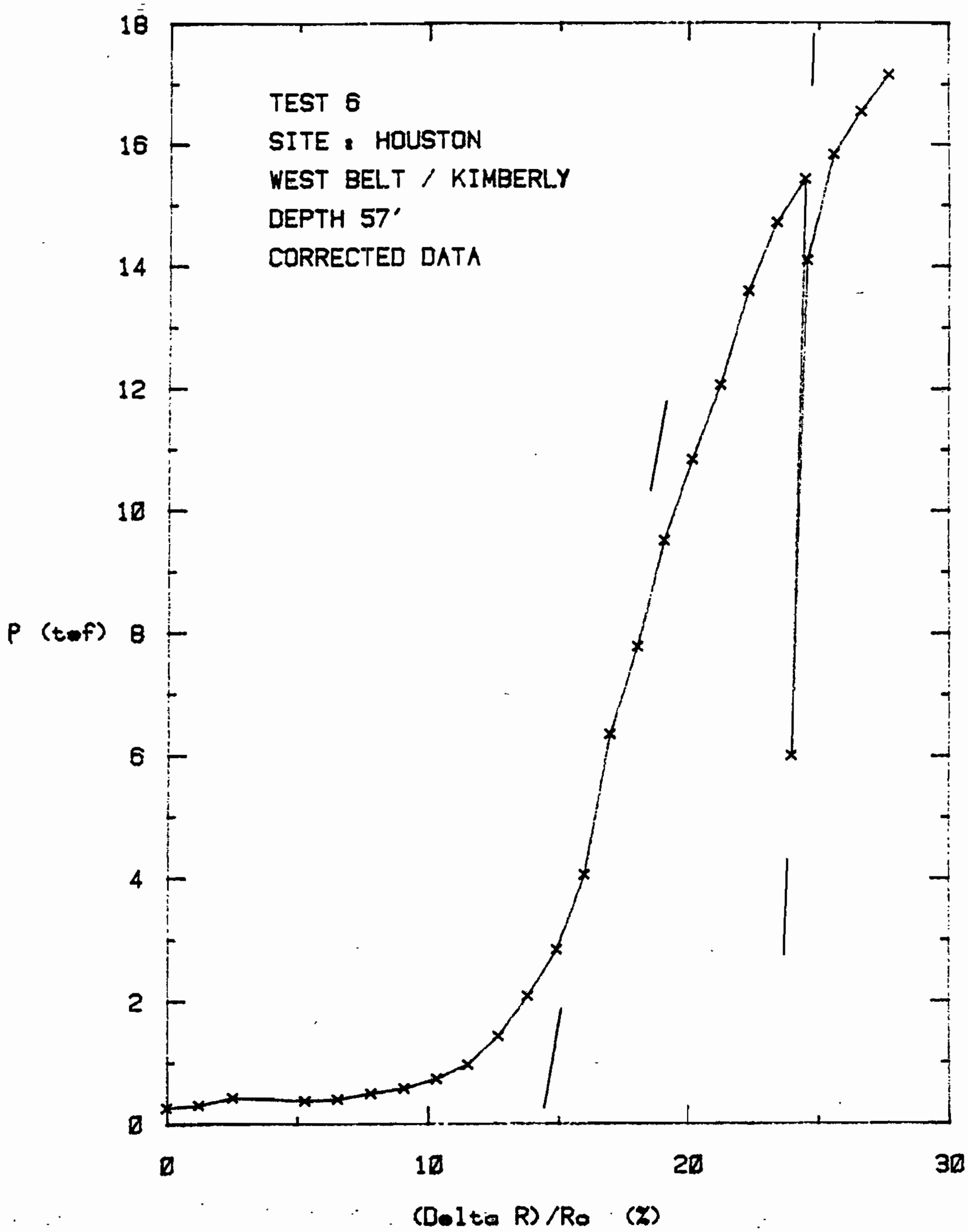


E_o = 770 TSF

E_{R1} = 1556 TSF

E_{R2} = 1343 TSF

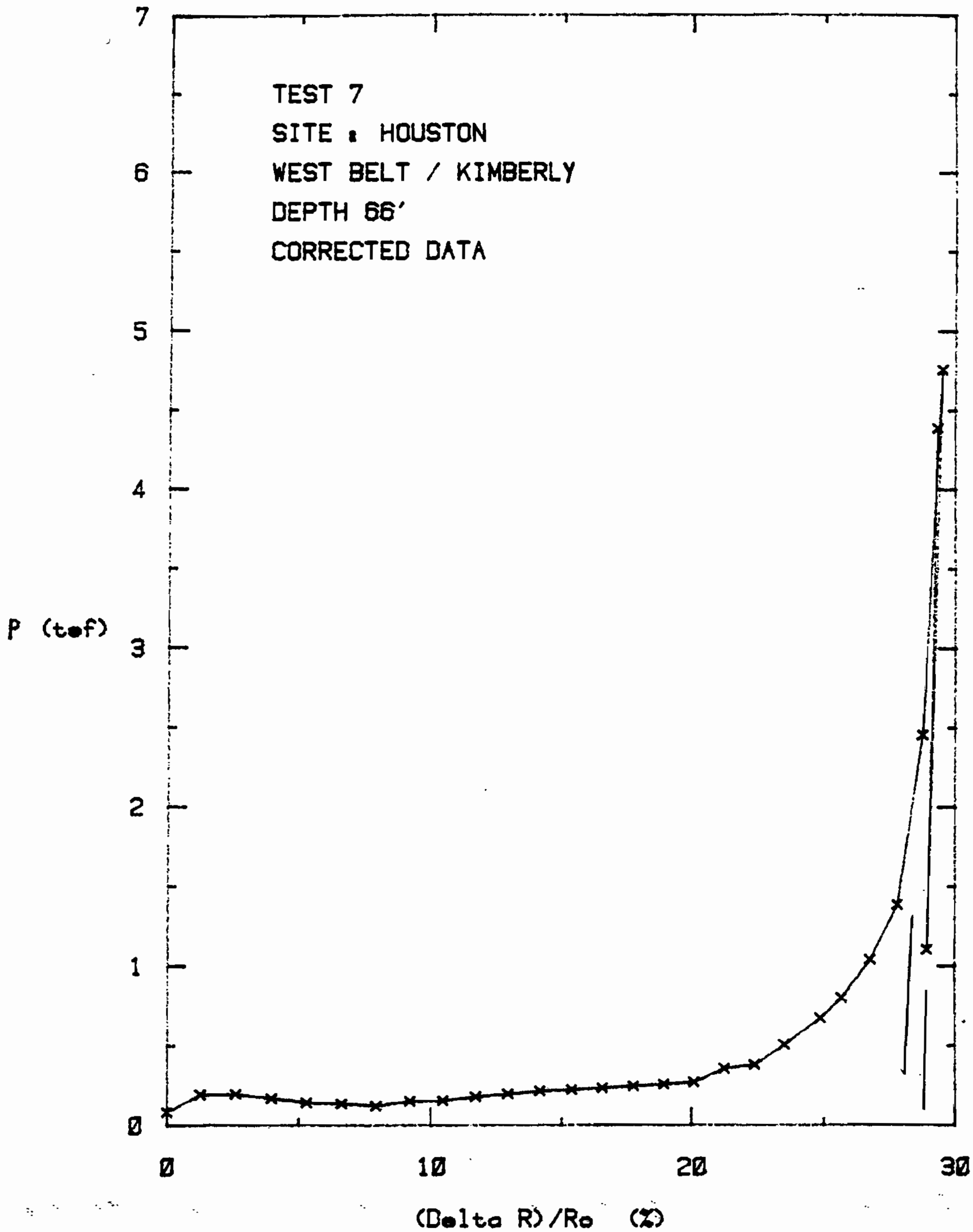
pl* = 14 TSF (ESTIMATED)



E_o = 367 TSF

E_R = 2344 TSF

pl* = 20.5 TSF (ESTIMATED)



E_o = 534 TSF

E_R = 1484 TSF

pl* = 14 TSF (ESTIMATED)

DRILLING REPORT

County Harris
Highway No. Beltway 8
Control 3256-1
Project No IPE 3-399C

Structure Retaining Wall at West Belt
Hole No. 5
Station 601 + 18.47
Loc. from Centerline Rt 78.39 Lt.

District No. 12
Date 2-2-81
Grd. Elev. 77.59
Grd. Water Elev. 58.0

Elev. (Ft.)	Depth (Ft.)	Samples	Log	THD Pen. Test No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure & Ult. Stress (psi)	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Description of Material and Remarks
												3' washed bore
						BW-5-1	0-11.1	132	15	39		clay,silty,gray,tan,soft,moist & calcite nodules
	5					2	0- 9.5	131	18			same + calcite nodules
		x		11	11							
						3	0-35.3	134	15			same + calcite nodules
						4	0-30.9	132	16	48		same + calcite nodules
						5	0-12.0		12			same +ferrous & calcite nodules
	10					6	0-10.3	132	25			same + ferrous & calcite nodules
		x		9	9							
						7	0- 9.0	132	15	38		clay,silty,gray,tan,moist & ferrous nodules
						8	0-24.6	134	15			same
						9	0-37.4	139	13			clay,silty,gray,brown,moist & ferrous nodules
	15					10	0-49.5	137	17			same
		x		19	22							
						11	0-52.0	137	15	42		clay,silty,gray,brown,moist
						12	0-51.4	135	15			same
						13	0-28.4	130	13			silty,sand,gray,soft,moist
	20					14	0-14.1	124	15			same
		x		42	18							
						15	MC		10	50		silty,sand,gray,soft,moist
						16	0-23.2	132	11			same
	25					17	MC		18			silty,sand,gray,soft, wet
		x		34	39							
		x		39	40							
	30											
		x		44	48							
		x		50/6"	50/45"							

County Harris
Highway No. Beltway 8
Control 3256-1
Project No IPE 3-399C

Structure Retaining Wall at West Belt
Hole No. 5
Station 601 + 18.47
Loc. from Centerline Rt. 78.39 Lt.

District No. 12
Date 2-2-81
Grd. Elev. 77.60'
Grd. Water Elev.

Elev. (Ft.)	Depth (Ft.)	Sample	Log	THD PEN. TEST No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure & Ult. Stress (psi)	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	DESCRIPTION OF MATERIAL AND REMARKS
	35											
		x		50/4¼"	50/3¼"							
		x		50/2½"	50/1½"							
	40											
		x		26	19							
						18	MC		26			silt, clayey, brown, very soft, moist
						19	MC	123	26	45		clay, silty, brown, soft, moist
						20	0- 3.2	123	26			same
	45					21	0-35.1	129	27			same
		x		25	23							
						22	MC		26			clay, brown, firm, moist
						23	0-17.4	125	30			same
						24	0-27.0	123	29			same
	50					25	0-48.3	126	25	66		same
		x		17	22							

Louis Gourley Jr. Gordon Williams Title

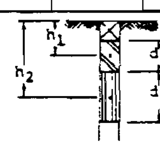
D-5
2/71

PILING OR DRILLED SHAFT FOUNDATION DESIGN

CALCULATED ALLOWABLE STATIC FRICTIONAL RESISTANCE
(Based on Coulomb's Theory, TAT)

Elev. (Ft.)	d (Ft.)	h (Ft.)	w (#/Ft³)	ø°	Tan ø	c (#/Ft²)	whTan ø (#/Ft²)	c+whTan ø = s (#/Ft²)	(T/Ft²)	Design Stress or ½ Shear Strength of Soil S (Tons/Ft²) Per Stratum	Allowable Static Frictional Resistance dS (Tons/Ft. of Perimeter) Per Stratum	Accumulative
77.5 to 74.5	3				DISREGARDED							
74.5 to 72.5	2	4	132	0	0	742	0	742	0.37	0.18	0.37	0.37
72.5 to 71.5	1				PEN	= 22				0.36	0.36	0.73
71.5 to 69.5	2	7	133	0	0	2382	0	2382	1.19	0.60	1.19	1.92
69.5 to 67.5	2	9	132	0	0	804	0	804	0.40	0.20	0.40	2.32
67.5 to 66.5	1				PEN	= 18				0.30	0.30	2.62
66.5 to 64.5	2	12	133	0	0	1210	0	1210	0.60	0.30	0.60	3.22
64.5 to 62.5	2	14	138	0	0	3130	0	3130	1.56	0.78	1.56	4.78
62.5 to 61.5	1				PEN	= 41				0.68	0.68	5.46
61.5 to 59.5	2	17	136	0	0	3722	0	3722	1.86	0.93	1.86	7.32
59.5 to 57.5	2	19	127	0	0	1530	0	1530	0.76	0.38	0.76	8.08
57.5 to 56.5	1				PEN	= 60				1.00	1.00	9.08
56.5 to 52.5	4	23	70	0	0	1670	0	1670	0.83	0.41	1.67	10.75
52.5 to 50.5	2				PEN	= 73				1.04	2.08	12.83
50.5 to 47.5	3				PEN	= 79				1.13	3.38	16.21
47.5 to 45.5	2				PEN	= 92				1.15	2.30	18.51
45.5 to 39.5	6				PEN	= 100				1.25	7.50	26.01
39.5 to 35.5	4				PEN	= 45				0.75	3.00	29.01
35.5 to 33.5	2	43	61	0	0	230	0	230	0.11	0.05	0.11	29.12
33.5 to 32.5	1	44.5	67	0	0	2527	0	2527	1.26	0.63	0.63	29.75
32.5 to 30.5	2				PEN	= 48				0.80	1.60	31.35
30.5 to 28.5	2	48	60	0	0	1600	0	1600	0.80	0.40	0.80	32.15

d (stratum thickness); h (depth of overburden to centroid of stratum); w (wet density of soil): For submerged conditions use wet density minus 62.4; ø (angle of internal friction); c (cohesion of soil) = c from TAT x 144; s (shear strength of soil) = c + whTan ø; S (½ shear strength of soil) = s/2; foundation perimeter (shortest measure around foundation).



Accumulative Allowable Static Frictional Resistance in Tons/Ft. of Pile Perimeter = ΣdS based on a safety factor of 2.0. **FORMULA:** p (Accumulative Allowable Static Frictional Resistance in Tons) = (ΣdS) (Pile Perimeter). To calculate $\Sigma(S_R dS)$ for drilled shafts, complete Form 1190.

Remarks:

County Harris
Highway No. Beltway 8
Control 3256-1
IPE 399C

Structure Ret. Wall at West Belt
Hole No. 5
Station 601 + 18.47
Loc. from Centerline Rt. 78.4 Lt.

District No. 12
Date 3-19-81
By Dong Q. Nguyen

Form 1091

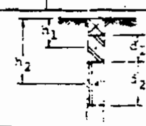
D-5
2/71

PILING OR DRILLED SHAFT FOUNDATION DESIGN

CALCULATED ALLOWABLE STATIC FRICTIONAL RESISTANCE
(Based on Coulomb's Theory, TAT)

Elev. (Ft.)	d (Ft.)	h (Ft.)	w (#/Ft^3)	ø°	Tan ø	c (#/Ft^2)	whTan ø (#/Ft^2)	c+whTan ø s (#/Ft^2)	s (T/Ft^2)	Design Stress or ½ Shear Strength of Soil S (Tons/Ft^2)	Allowable Static Frictional Resistance dS (Tons/Ft. of Perimeter)	
										Per Stratum	Per Stratum	Accumulative
28.5 to 27.5	1	49.5	62	0	0	3478	0	3478	1.74	0.87	0.87	33.02
27.5 to 26.5	1				PEN	= 41				0.82	0.82	33.84

d (stratum thickness); h (depth of overburden to centroid of stratum); w (wet density of soil); For submerged conditions use wet density minus 62.4; ø (angle of internal friction); c (cohesion of soil) = c from TAT x 144; s (shear strength of soil) = c + whTan ø; S (½ shear strength of soil) = s/2; foundation perimeter (shortest measure around foundation).



Accumulative Allowable Static Frictional Resistance in Tons/Ft. of Pile Perimeter = ΣdS based on a safety factor of 2.0. FORMULA: p (Accumulative Allowable Static Frictional Resistance in Tons) = (ΣdS) (Pile Perimeter). To calculate $\Sigma(S_R dS)$ for drilled shafts complete Form 1193

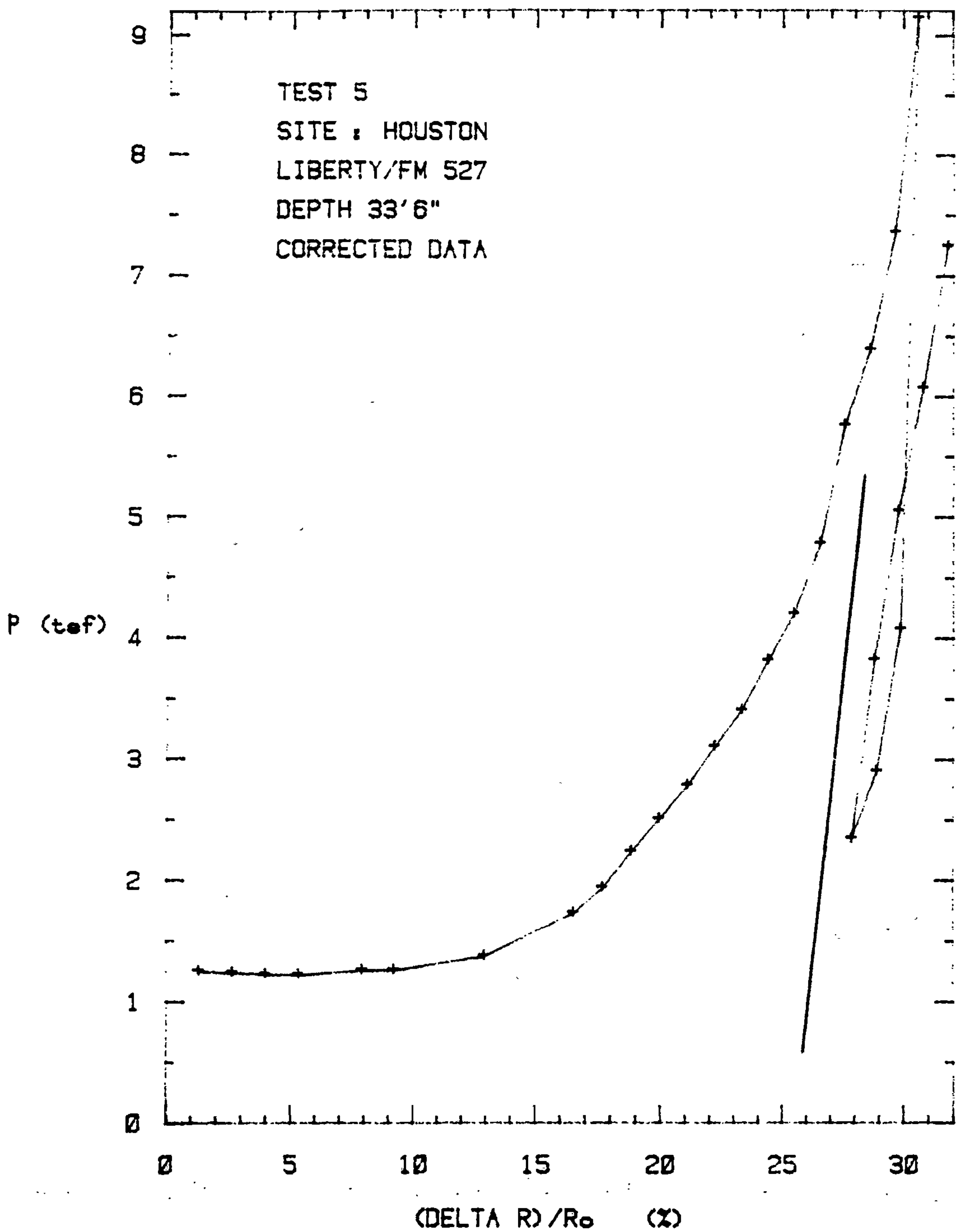
Remarks:

County Harris
Highway No. Beltway 8
Control 3256-1
IPE 399C
Structure Ret. Wall at West Belt
Hole No. 5
Station 601 + 18.47
Loc. from Centerline Rt. 78.4 Lt.
District No. 12
Date 3-13-81
By Dong Q. Nguyen

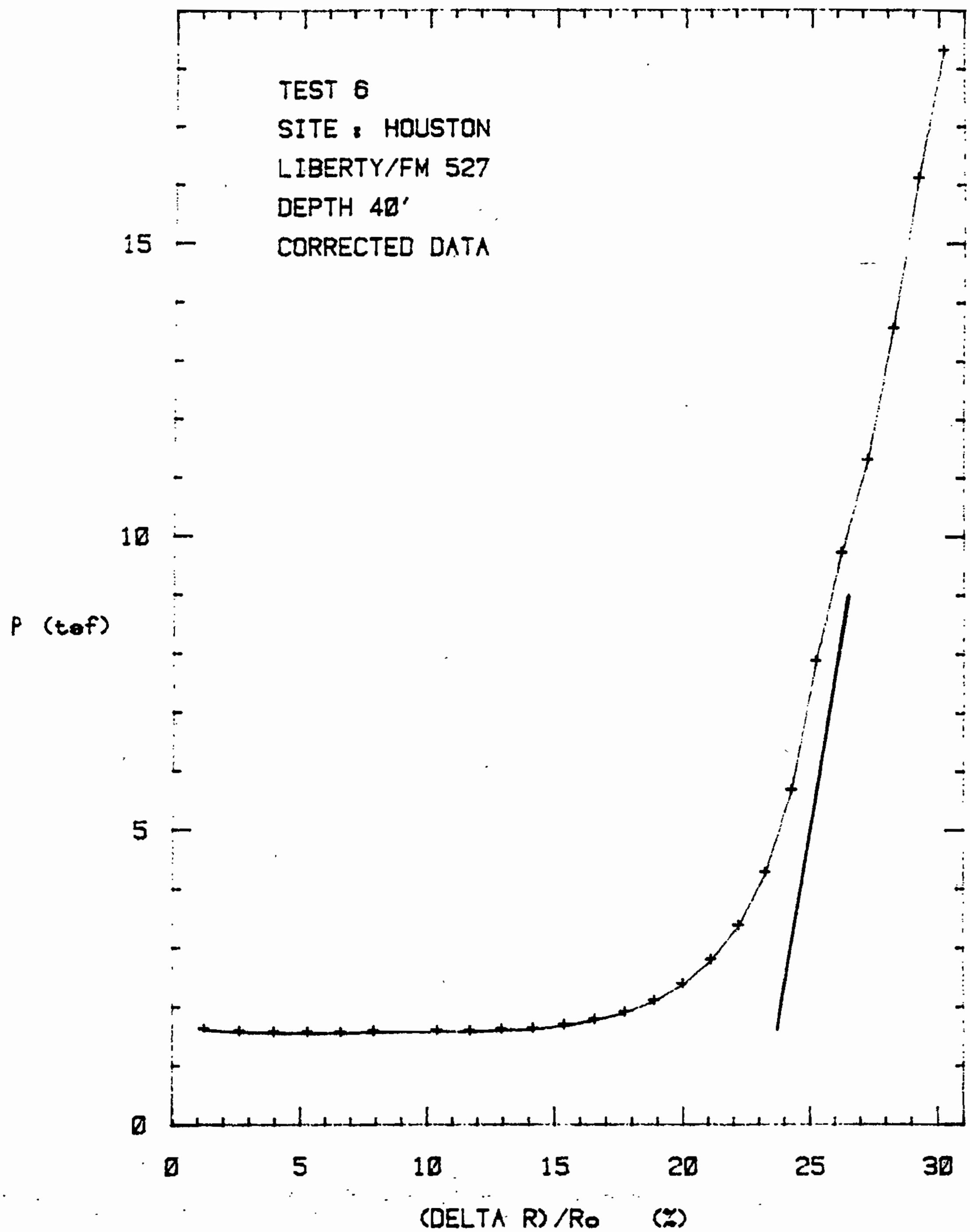
Form 1091

APPENDIX C

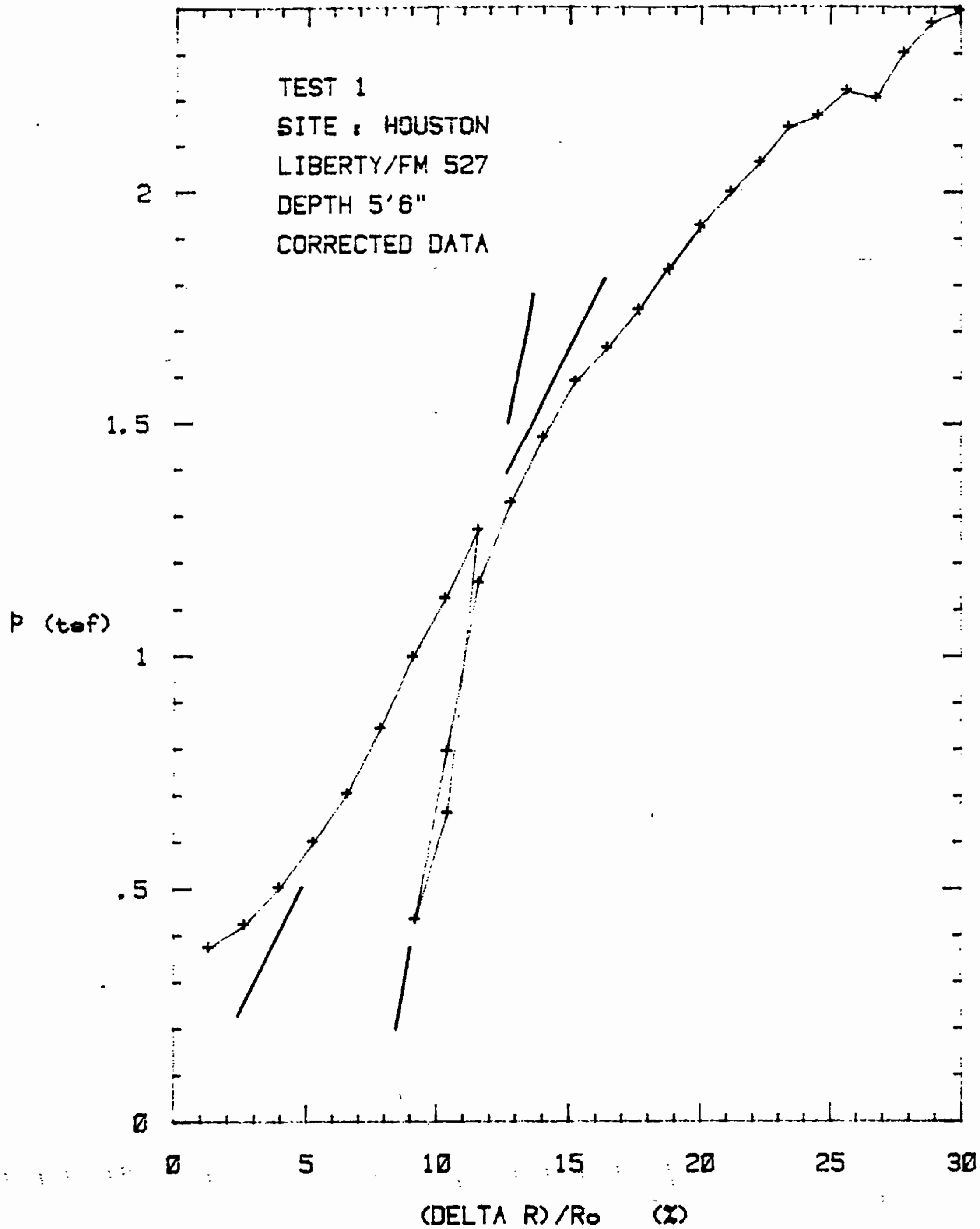
SOIL DATA: LIBERTY AND MESA


TEST 5
SITE : HOUSTON
LIBERTY/FM 527
DEPTH 33'6"
CORRECTED DATA
P (tsf)
9
8
7
6
5
4
3
2
1
0
0
5
10
15
20
25
30
(DELTA R)/Ro (%)

$E_0 = 319$ tsf


TEST 6
SITE : HOUSTON
LIBERTY/FM 527
DEPTH 40'
CORRECTED DATA
P (tsf)
(DELTA R)/Ro (%)
0
5
10
15
20
25
30

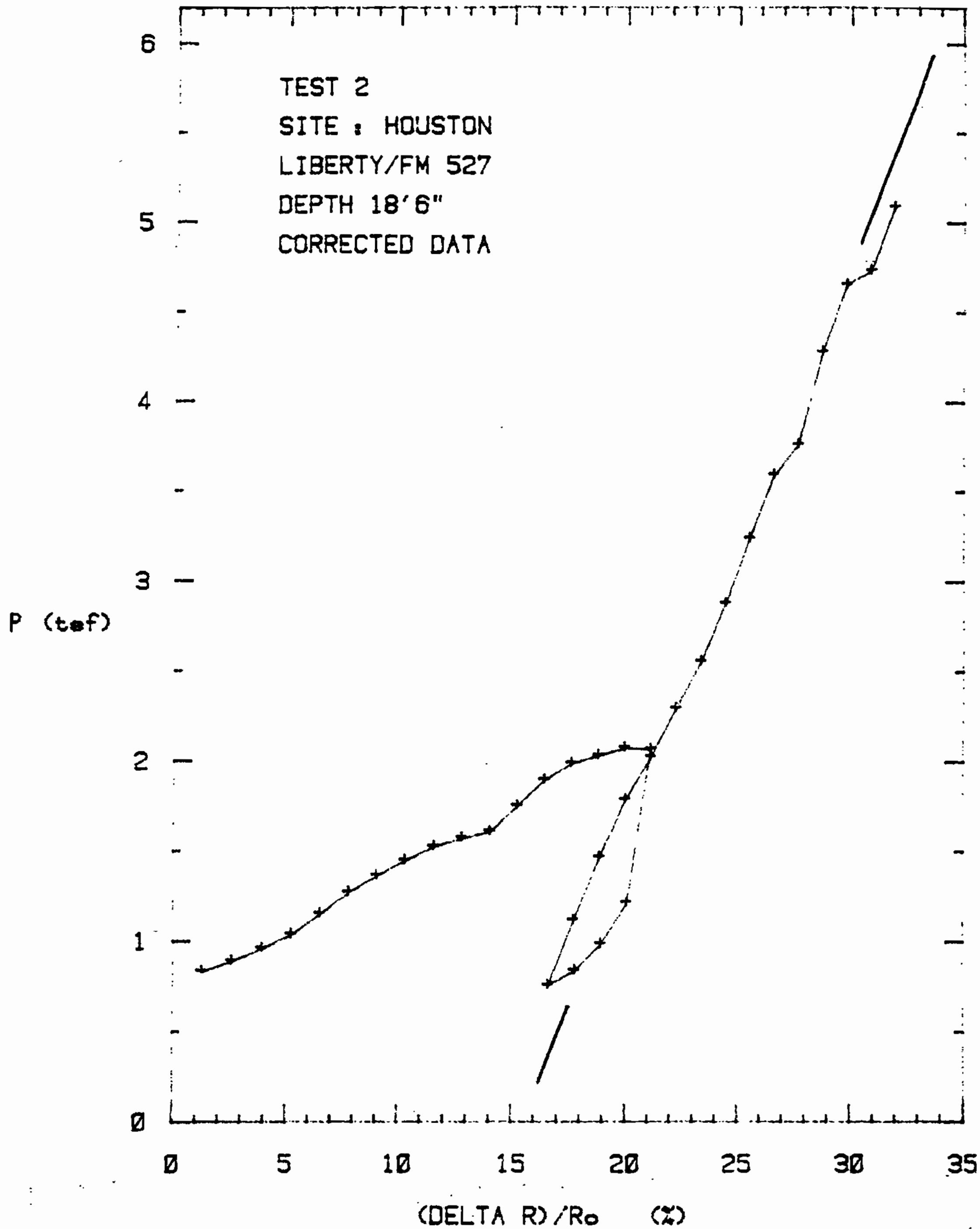
E_0 = 418 tsf



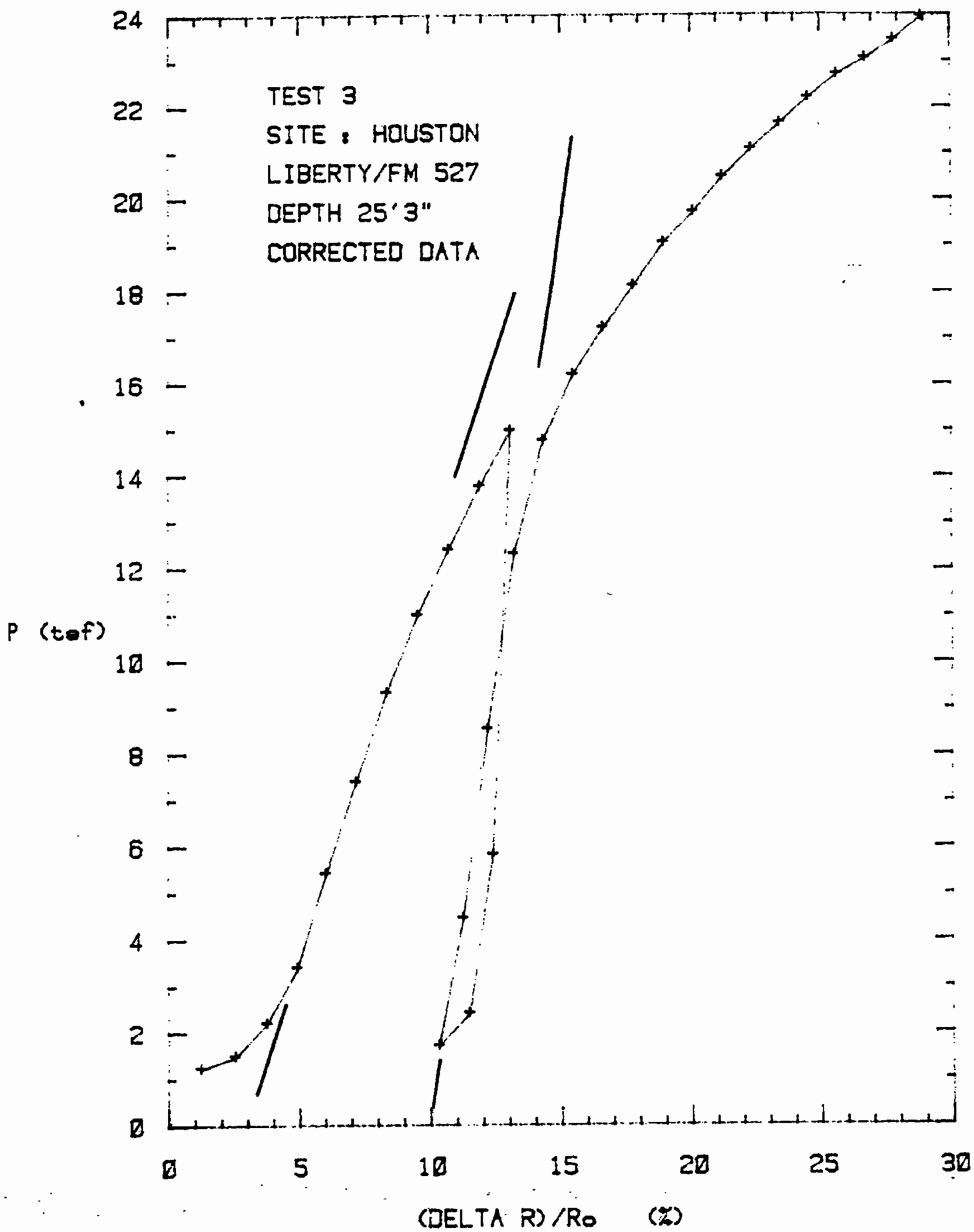
$E_o = 16.4$ tsf

$E_R = 44.7$ tsf

pl* = 2.35 tsf


TEST 2
SITE : HOUSTON
LIBERTY/FM 527
DEPTH 18'6"
CORRECTED DATA
P (tsf)
(DELTA R)/Ro (%)
0
5
10
15
20
25
30
35
1
2
3
4
5
6

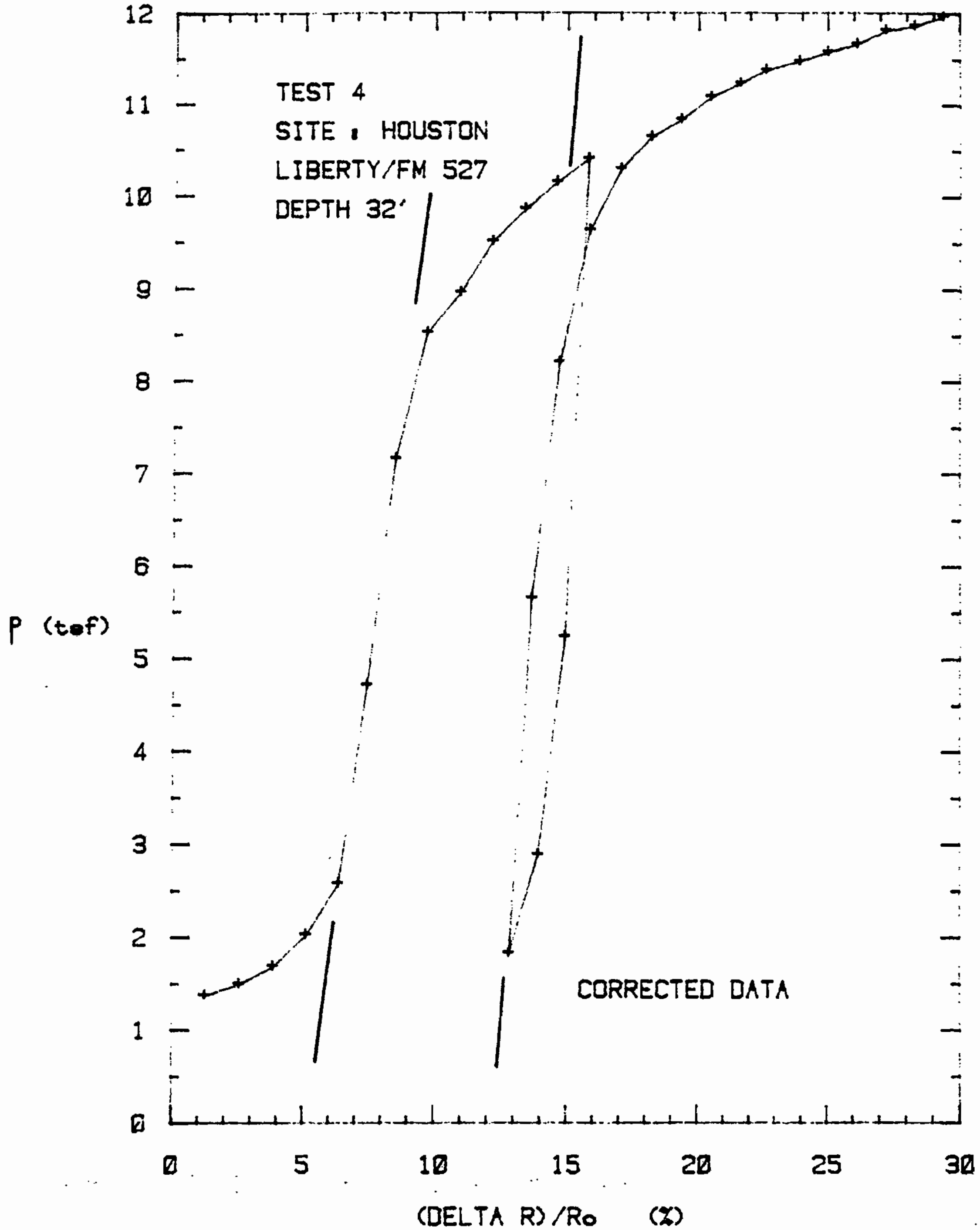
$E_0 = 55.3$ tsf



E_o = 245 tsf

E_R = 504 tsf

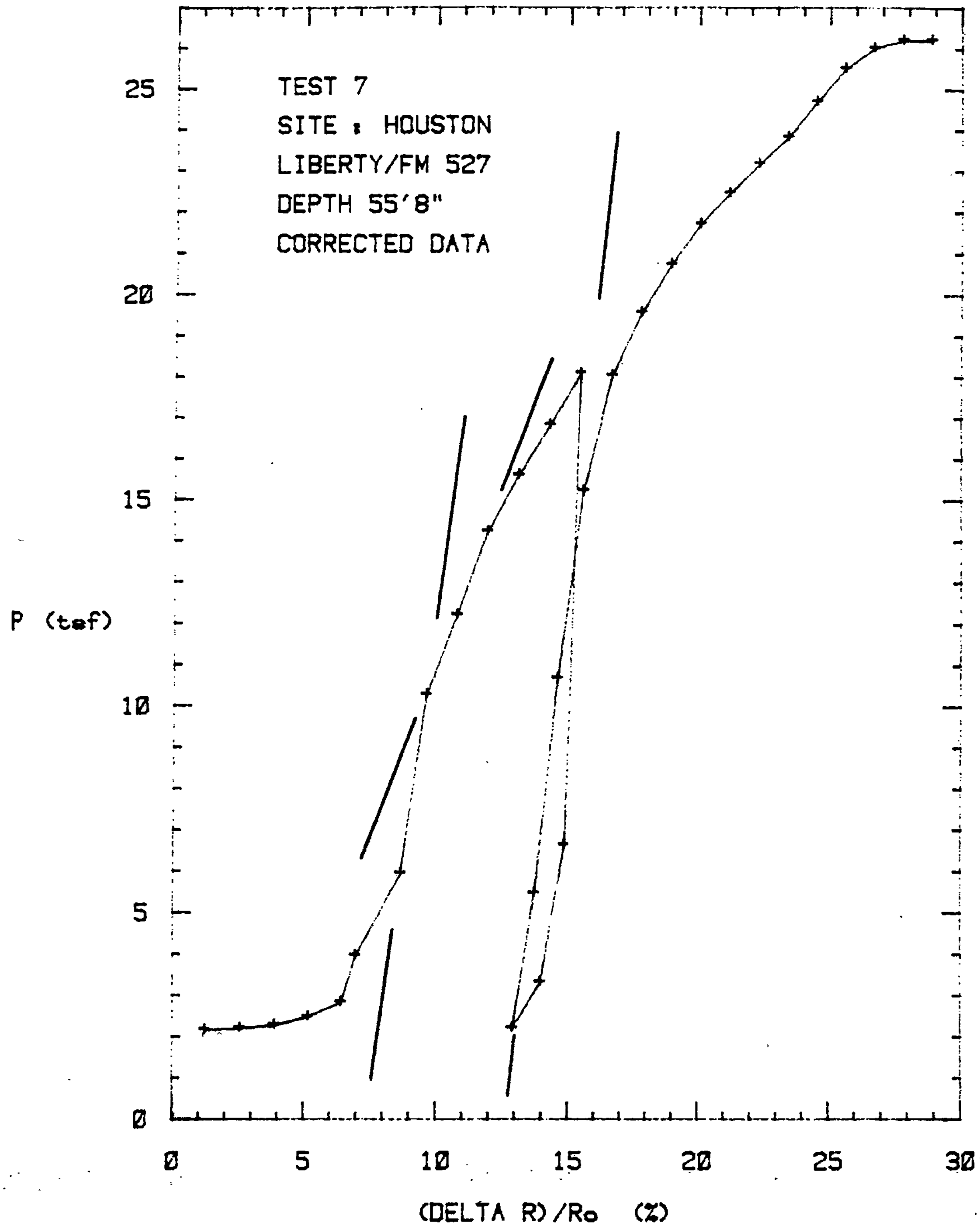
pl* = 28.5 tsf



E_o = 306 tsf

E_R = 712 tsf

pl* = 10.4 tsf



E_o = 662 tsf

E_R = 942 tsf

pl* = 26.5 tsf

Texas Highway Department
Form 564

DRILLING REPORT

(For use with Undisturbed Sampling & Testing)

County: Harris
Highway No.: FM 527
Control: 980-1
Project No.: IPE 3-539

Structure: Railroad Underpass
Hole No.: 4
Station: 423 + 08
Loc. from Centerline: Rt. ______ Lt. 53

District No.: 12
Date: 4-11-79
Grd. Elev.: 46.0
Grd. Water Elev.: ______

Elev. (Ft.)	Depth (Ft.)	Sample	Log	THD Pen. Test No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure (psi)	Ult. Stress (psi)	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Description of Material and Remarks
													Fish tailed fill
													to
													5'
	5												
						527-4-1	0	13	127	25			Clay, slight sand, silt, dark gray, soft, moist
						2	5	16	130	21			Same
						3	10	23	131	19			Clay, sandy, gray, tan, soft, moist
						4	15	32	132	19			Same
						5	0	7		21	32		Same (sc)
	10			5	5								
						6	5	11	132	20			Fine sand, slight clay, lt. gray, tan, soft, wet
						7	10	32	134	21	25		Fine sandm lt. gray, tan, soft, water bearing
													Same as above
						8				25	21		Same
	15			12	15								Using washer pen
													Sand, no recovery- washed out
													Same
				23	20								(s)
	20												Washed out
				23	24								
													Same
				23	24								
													Same
				42	40								
	25												Same
				9	10								
						9	15	32	125	29			Clay, brown, gray, stiff, moist, w/calcareous
						10	0	22	128	23			Clay, slight silt, brown, gray, stiff, moist, w/calcs
						11	5	39	130	22	58		Same (ch)
	30			14	15								
						12	10	40	128	24			Clay, brown, stiff, moist w/calcs
						13	15	81	133	20			Clay, sandy, brown, gray, moist, w/calcs
						14				23			Fine sand, silty, brown, very soft, water bearing
						15	0	17	135	19	54		Clay, silty, brown, stiff, moist
	35												

Driller: Bob Springer
Logger: R.K. Curson
Title: ______

(Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken.)

HW29 1096 F-1201 3-68 2M

Texas Highway Department
Form 584

DRILLING REPORT

(For use with Undisturbed Sampling & Testing)

County Harris
Highway No. 527
Control
Project No. IPE-3-630

Structure
Hole No. 4
Station 423 + 08
Loc. from Centerline Rt. Lt.

District No. 12
Date 5-14-79
Grd. Elev.
Grd. Water Elev.

Elev. (Ft.)	Depth (Ft.)	Samples	Log	THD Pen. Test No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure & Ult. Stress (psi)		Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Description of Material and Remarks
				50/3.50	50/4.75								Using tooth barrel
						16				14			Silty sand, brown, loose, water bearing
					50	17				24	28		Same (s)
													Using washer pen washed out
	40			50/6	50/2								
													Wash out
				50/2	50/1								Pack sau
						18	5	36	124	32			Silty, clay, brown, stiff, moist
						19	10	48	125	31			Same
						20	15	58	123	30	79		Same (ch)
	45			12	14								
						21	0	24	123	31			
						22	5	45	125	25			Same
						23	10	21	134	19			Clay, sandy, silty, brown, stiff, moist
						24	15	35	136	19	31		Same
	50			11	13								
						25	0	13	134	18			Sandy, clay, brown, gray, soft, moist
						26	5	24	136	17			Same
						27	10	47	138	17			Same
						28	15	74	138	17	30		Sand, slight, clay, brown, gray, soft, moist (cl)
	55			27	33								
						29	0	12	138	16			Same
						30	5	36	136	17			Same
						31	10	44	135	17			Same w/calcs
						32	15	55	135	17	31		Same
	60			36	41								
						33	0	56	131	20			Clay, brown, stiff, moist, w/calcs
						34	5	69	131	19			Same
						35	10	88	132	22			Clay, silt, brown, stiff, moist
						36	15	73	127	23	46		Same
	65			41	37								
						37				27			Dist core clay, brown, stiff, moist
						38				27			Same
						39	0	20	124	25			Clay, sandy, brown, stiff, moist
						40	5	49	122	29	70		Same (ch)
	70												

Driller Logger Title

(Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken)

HW29-1095 F-12011 3-68 2M

Texas Highway Department
Form 554

DRILLING REPORT
(For use with Undisturbed Sampling & Testing)

County: Harris | Structure: | District No. 12
Highway No. 527 | Hole No. 4 | Date 5-14-79
Control: | Station 423 + 08 | Grd. Elev.
Project No. IPE-3-630 | Loc. from Centerline: Rt. Lt. | Grd. Water Elev.

Elev. (Ft.)	Depth (Ft.)	Sample	Log	THD Pen. Test No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure & Ult. Stress (psi)		Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Description of Material and Remarks
				26	37								
						41	10	57	124	30			Clay, silty, brown, stiff, moist
						42	15	59	124	29			Same
						43	0	32	125	25			Clay, silt, laminate, brown, stiff, moist
						44	5	58	121	28	71		Same (ch)
	75			10	9								
						45	10	46	116	37			Same
						46	15	44	117	34			Same
						47	0	15	118	30			Same
						48	5	42	116	39	57		Same
	80			9	9								
						49	10	42	125	25			Clay, slight silt, gray, stiff, moist
						50	15	51	125	25			Same
						51	0	32	124	25			Same
						52	5	46	126	23	51		Same
	85			14	15								
						53	10	54	128	28			Clay, brown, blue, stiff, moist, w/calcs
						54	15	56	128	22			Same
						55	0	49	130	21			Clay, sandy, silty, brown, stiff, moist
						56	5	47	130	20	35		Same w/calcs
	90			50/3.75	50/1.50					s			
						57	10	49	130	25			Clay, silty, brown, stiff, moist
						58	15	47	130	21			Same w/calcs nodules
						59				19			Silt, slight clay, brown, stiff, moit w/ silt stone
						60				18	41		Same (cl)
[illegible]	100			36	33								
						61	0	50	132	19			Same
						62	5	63	131	21			Same
						63	10	46	128	23			Same
						64	15	74	130	23	51		Same
[illegible]	105			30	32								
						65	0	50	131	21			Silty, clay, brown, stiff, moist
						66	5	57	130	23			Same
						67				21	36		Clay, sand, silt, brown, stiff, moist
				50/3.25	50/2								Using washer pen (s)

Driller: Bob Springer | Logger: R.K. Curson | Title:

Texas Highway Department
Form 556

DRILLING REPORT
(For use with Undisturbed Sampling & Testing)

County Harris | Structure | District No. 12
Highway No. 527 | Hole No. 4 | Date
Control | Station | Grd. Elev
Project No. IPE 3-630 | Loc. from Centerline Rt. Lt. | Grd. Water Elev.

Elev. (Ft.)	Depth (Ft.)	Sample	Log	THD PEN. TEST No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure (psi)	& Ult. Stress (psi)	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	DESCRIPTION OF MATERIAL AND REMARKS
													Washed out very hard pack sand
				50/4.25	50/1.50								(s)
													washed out layers of day, very hard
				50/50	50/75								
	110												No Recovery
						68				24			Silt, clay, slight sand, laminate, gray, stiff, moist
						69					29		Silt, clay, slight sand, laminate, gray, stiff, moist
													same as below
						70	10	49	122	25	35		Fine sand, very slight clay, gray, firm, moist (cl)
						71				22			Same
	115			50/2.75	50/1.75								
						72	15	97	129	42			Same
						73	0	4.3	125				Same
						74				26			Same
	120					75	5	22	124	26	36		Same
				25	28								
						76				27			Disturbed core, sand, clay, gray, firm, moist
						77				37	30		Disturbed core, clay, slight sand, gray, stiff, moist

Driller | Logger | Title

(Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken.

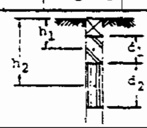
HW29-1095 F-1201 3-64 2M

PILING OR DRILLED SHAFT FOUNDATION DESIGN

CALCULATED ALLOWABLE STATIC FRICTIONAL RESISTANCE
(Based on Coulomb's Theory, TAT)

Elev. (Ft.)	d (Ft.)	h (Ft.)	w (#/Ft³)	ø°	Tan ø	c (#/Ft²)	whTan ø (#/Ft²)	c+whTan ø s (#/Ft²)	(T/Ft²)	Design Stress or ½ Shear Strength of Soil S (Tons/Ft²) Per Stratum	Allowable Static Frictional Resistance ds (Tons/Ft. of Perimeter) Per Stratum	Accumulative
46 to 41	5			DISREGARDED								
41 to 37	4	7	130	18.4	.333	274	303	577	.29	.14	.56	.56
37 to 36	1	9.5	133	12.5	.222	432	280	712	.35	.18	.18	.74
36 to 35	1			PEN =	10					.14	.14	.88
35 to 32	3	12.5	133	12.5	.222	432	369	801	.40	.20	.60	1.48
32 to 28	4			PEN =	27					.33	1.32	2.80
28 to 36	2			PEN =	43					.54	1.08	3.88
26 to 22	4			PEN =	47					.59	2.36	6.24
22 to 21	1			PEN =	82					1.03	1.03	7.27
21 to 19	2			PEN =	19					.23	.46	7.73
19 to 17	2	27	64	0	0	1440	0	1440	.72	.36	.72	8.45
17 to 16	1	29.5	68	32.5	.636	677	1276	1953	.98	.49	.49	8.94
16 to 15	1			PEN =	29					.59	.59	9.53
15 to 11	4	33	68	32.5	.636	677	1427	2104	1.05	.53	2.10	11.63
11 to 10	1			PEN =	100					1.25	1.25	12.88
10 to 8	2			PEN =	50					.63	1.26	14.14
8 to 4	4			PEN =	100					1.25	5.00	19.14
4 to 1	3	43.5	62	28.3	.538	1051	1451	2502		1.25	3.75	22.89
1 to 0	1			PEN =	26					.52	.52	23.41
0 to -2	2	47	62	28.3	.538	1051	1568	2619	1.31	.65	1.30	24.71
-2 to -4	2	49	73	23.2	.429	43	1534	1577	.79	.39	.78	25.49
-4 to -5	1			PEN =	24					.41	.41	25.90
-5 to -9	4	53	74	36.9	.750	288	2941	3229	1.61	.81	3.23	29.13

d (stratum thickness); h (depth of overburden to centroid of stratum); w (wet density of soil): For submerged conditions use wet density minus 62.4; ø (angle of internal friction); c (cohesion of soil) = c from TAT x 144; s (shear strength of soil) = c + whTan ø; S (½ shear strength of soil) = s/2; foundation perimeter (shortest measure around foundation).



Accumulative Allowable Static Frictional Resistance in Tons/Ft. of Pile Perimeter = ΣdS based on a safety factor of 2.0. **FORMULA:** P (Accumulative Allowable Static Frictional Resistance in Tons) = (ΣdS) (Pile Perimeter). To calculate $\Sigma(S_R dS)$ for drilled shafts, complete Form 1190.

Remarks:

County Harris — Structure Railroad Underpass — District No. 12
Highway No. FM 527 — Hole No. 4 — Date 6-5-79
Control 980 - 2 — Station 423 + 08 — By Gordon Williams
IPE 630 — Loc. from Centerline Rt. ____ Lt. 55

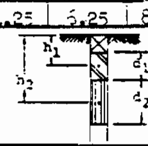
Form 1091

PILING OR DRILLED SHAFT FOUNDATION DESIGN

CALCULATED ALLOWABLE STATIC FRICTIONAL RESISTANCE
(Based on Coulomb's Theory, TAT)

Elev. (Ft.)	d (Ft.)	h (Ft.)	w (#/Ft³)	ø°	Tan ø	c (#/Ft²)	whTan ø (#/Ft²)	c+whTan ø s (#/Ft²)	c+whTan ø s (T/Ft²)	Design Stress or ½ Shear Strength of Soil S (Tons/Ft²) Per Stratum	Allowable Static Frictional Resistance dS (Tons/Ft. of Perimeter) Per Stratum	Allowable Static Frictional Resistance dS (Tons/Ft. of Perimeter) Accumulative
-9 to -10	1			PEN =	60					1.01	1.01	30.14
-10 to -14	4	58	74	32.0	.625	504	2682	3186	1.59	.79	3.18	33.32
-14 to -15	1			PEN =	77					1.29	1.29	34.61
-15 to -19	4	63	68	31.0	.600	1829	2570	4399	2.20	1.10	4.40	39.01
-19 to -22	3			PEN =	78					1.31	3.93	42.94
-22 to -24	2	69	61	45.0	1.00	590	4209	4799	2.40	1.20	2.40	45.34
-25 to -24	1			PEN =	53					1.27	1.27	46.61
-25 to -29	4	73	61	25.0	.467	1440	2079	3519	1.76	.88	3.52	50.13
-29 to -30	1			PEN =	19					.39	.39	50.52
-30 to -34	4	78	54	29.7	.571	691	2405	3096	1.55	.78	3.10	53.62
-34 to -35	1			PEN =	18					.36	.36	53.98
-35 to -37	2	82	63	18.4	.333	1109	1720	1829	.91	.46	.91	54.89
-37 to -39	2	84	63	21.8	.400	1526	2117	3643	1.82	.91	1.82	56.71
-39 to -40	1			PEN =	29					.59	.59	57.30
-40 to -41	1	86.5	63	21.8	.400	1526	2180	3706	1.85	.93	.93	58.23
-41 to -44	3	88.5	68	0	0	2880	0	2880	1.44	.72	2.16	6039
-44 to -45	1			PEN =	100					1.25	1.25	61.64
-45 to -49	4	93	68	0	0	2880	0	2880	1.44	.72	2.88	6452
-49 to -50	1			PEN =	69					1.16	1.16	65.68
-50 to -54	4	98	68	13.0	.231	2332	1539	3871	1.93	.97	3.87	6955
-54 to -55	1			PEN =	62					1.09	1.09	7064
-55 to -58	3	102	68	10.3	.182	2966	1262	4228	2.11	1.06	3.19	73.82
-58 to -63	5			PEN =	100					1.25	6.25	80.07

d (stratum thickness); h (depth of overburden to centroid of stratum); w (wet density of soil): For submerged conditions use wet density minus 62.4; ø (angle of internal friction); c (cohesion of soil) = c from TAT x 144; s (shear strength of soil) = c + whTan ø; S (½ shear strength of soil) = s/2; foundation perimeter (shortest measure around foundation).



Accumulative Allowable Static Frictional Resistance in Tons/Ft. of Pile Perimeter = ΣdS based on a safety factor of 2.0. **FORMULA:** p (Accumulative Allowable Static Frictional Resistance in Tons) = (ΣdS) (Pile Perimeter). To calculate $\Sigma(S_R dS)$ for drilled shafts, complete Form 119C.

Remarks:

County Harris
Highway No. FM 527
Control 980-1
IPE 630

Structure Railroad Underpass
Hole No. 4
Station 423 + 05
Loc. from Centerline Rt. ______ Lt. 23

District No. 12
Date 0-5-79
By Gordon Williams

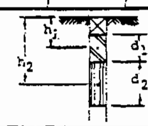
Form 1091

PILING OR DRILLED SHAFT FOUNDATION DESIGN

CALCULATED ALLOWABLE STATIC FRICTIONAL RESISTANCE
(Based on Coulomb's Theory, TAT)

Elev. (Ft.)	d (Ft.)	h (Ft.)	w (#/Ft³)	ø°	Tan ø	c (#/Ft²)	whTan ø (#/Ft²)	c+whTan ø s (#/Ft²)	c+whTan ø s (T/Ft²)	Design Stress or ½ Shear Strength of Soil S (Tons/Ft²) Per Stratum	Allowable Static Frictional Resistance dS (Tons/Ft. of Perimeter) Per Stratum	Allowable Static Frictional Resistance dS (Tons/Ft. of Perimeter) Accumulative
-63 to -69	6	112	61	35.5	.714	144	4878	5022	2.51	1.25	7.50	87.57
-69 to -70	1			PEN =	100					1.25	1.25	88.82
-70 to -74	4	118	61	35.5	.714	144	5139	5283	2.64	1.32	5.28	94.10
-74 to -77	3			PEN =	53					.89	2.67	96=77

d (stratum thickness); h (depth of overburden to centroid of stratum); w (wet density of soil): For submerged conditions use wet density minus 62.4; ø (angle of internal friction); c (cohesion of soil) = c from TAT x 144; s (shear strength of soil) = c + whTan ø; S (½ shear strength of soil) = s/2; foundation perimeter (shortest measure around foundation).



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Remarks:

County Harris
Highway No. FM 527
Control 980-1
IPE 630

Structure Railroad Underpass
Hole No. 4
Station 423 + 08
Loc. from Centerline Rt. ____ Lt. 53

District No. 12
Date 6-5-70
By Gordon Williams

Form 1091