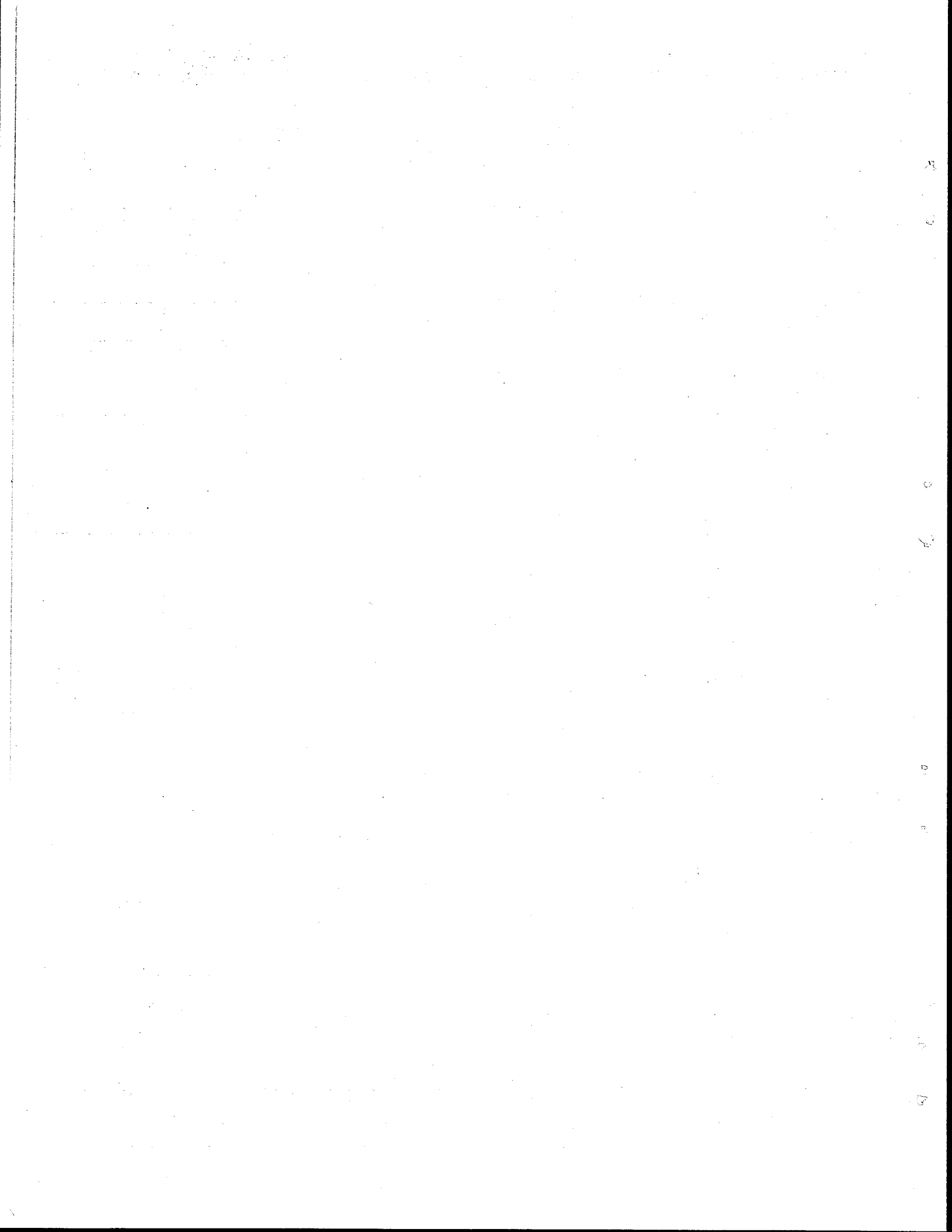


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16. Abstract In this report, a description is made of procedures available to design single piles subjected to static lateral loads on the basis of pressuremeter test results. Of the seven methods mentioned two are presented in detail: the Briaud-Smith-Meyer method in its detailed and simplified version and the Imai method. The methods are presented as step by step procedures. Then design examples are given and solved. An indication of the accuracy of the Briaud-Smith-Meyer method is given by comparing the predicted behavior and the measured behavior for actual case histories.			
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PRESSUREMETER DESIGN OF LATERALLY LOADED PILES

by

Jean-Louis Briaud, Larry Tucker, and Trevor Smith

Research Report 340-3

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

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SUMMARY

One of the most obvious applications of the pressuremeter test is the solution of the problem of laterally loaded piles. Indeed the cylindrical expansion of the pressuremeter probe is analogous to the lateral movement of the pile.

This report is divided into four parts. In a first part all known design methods of laterally loaded piles on the basis of pressuremeter tests are briefly reviewed. In a second part the Briaud-Smith-Meyer method, its simplified version and the Imai method are detailed in step by step procedures. In a third part a design example is presented to clarify the design steps of the three methods outlined in part 2. Finally in part 4, a comparison between the predicted and the measured response of four piles at four different sites is shown to give an idea of the accuracy of the Briaud-Smith-Meyer method.

When a pile is loaded laterally, the two main soil resistance components are the front resistance and the friction resistance to the pile lateral movement. The Briaud-Smith-Meyer method acknowledges this difference and uses a P-y curve made of two components: the Q-y curve and the F-y curve. This distinction between front resistance and friction resistance is as crucial for laterally loaded piles as the distinction between point resistance and friction resistance for vertically loaded piles. One reason is that the front resistance requires relatively large movements to be mobilized while the friction resistance requires comparatively small movements to be fully mobilized. As a result, at working loads the full friction resistance is generally

mobilized while the front resistance is only partially mobilized.

It must be emphasized that one of the critical elements in the accuracy of the predictions is the performance of quality pressuremeter tests and that such quality pressuremeter tests can only be performed by trained professionals.

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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 or 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.

IMPLEMENTATION STATEMENT

This report gives the details of existing pressuremeter methods for the design of laterally loaded piles. These methods require the use of a new piece of equipment: a preboring pressuremeter. These methods are 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 their implementation.

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GLOSSARY OF TERMS

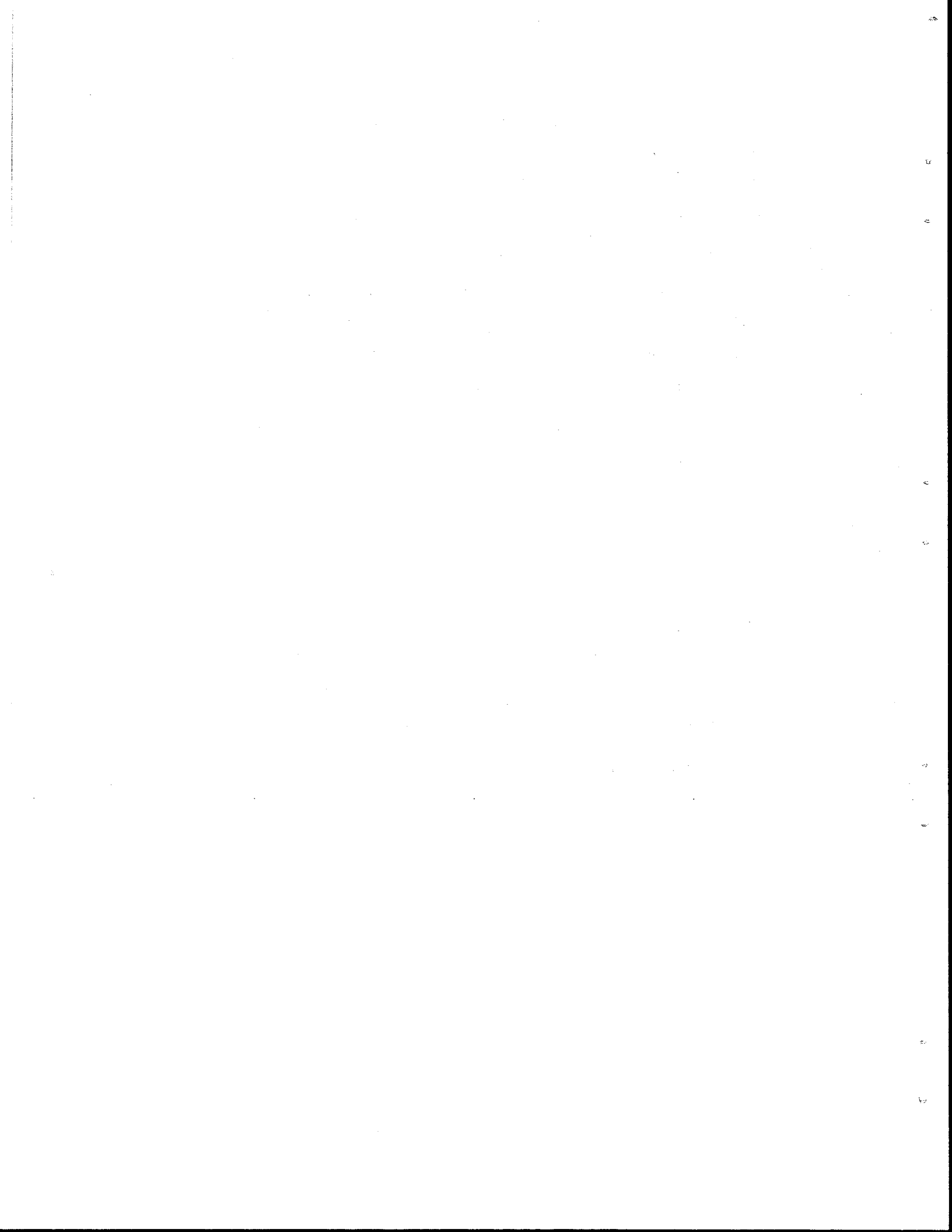
- A_p area of pile point
- B pile diameter or width
- C_u undrained soil strength
- E_m pressuremeter elastic modulus
- E_r pressuremeter elastic reload modulus
- F Friction resistance per unit length of pile
- F_b friction resistance on the base of the pile
- f pile head fixity parameter
- H horizontal load
- h pile length
- k_{OH} coefficient of at rest earth pressure
- k modulus of subgrade reaction
- k_m soil stiffness value
- k_o specific soil stiffness value
- \bar{k}_o equivalent soil stiffness value
- K_o basic soil stiffness value
- K design soil stiffness value
- ℓ_o pile transfer length
- L length of inflatable part of pressuremeter probe
- ℓ distance between ground surface and application of lateral load
- M bending moment in pile at any depth z
- M_{max} maximum bending moment in the pile
- M_o bending moment in the pile at the ground surface
- M_f bending moment corresponding to a completely fixed pile head connection

GLOSSARY OF TERMS (Con't)

- P total soil resistance per unit length of pile
- p pressure in probe during a pressuremeter test
- P_L limit pressure from a pressuremeter test
- P_{OH} total horizontal at rest pressure
- P_L^* net limit pressure from a pressuremeter test = $P_L - P_{OH}$
- p^* net pressure from a pressuremeter test
- P_{corr} corrected net pressure from a pressuremeter test
- p_o beginning pressure on linear portion of pressuremeter curve
- p_f final pressure on linear portion of pressuremeter curve
- Q front resistance per unit length of pile
- r_o initial radius of pressuremeter probe before inflation
- r increase in radius of pressuremeter probe
- R_1 pressuremeter radius corresponding to the increase in volume V_1
- ΔR_1 increase in pressuremeter radius corresponding to the increase in volume ΔV_1
- R_{pmt} pressuremeter radius before inflation
- RR relative rigidity of pile-soil system
- r_o beginning radius on linear portion of pressuremeter curve
- r_f final radius on linear portion of pressuremeter curve
- r_m radius at mid-point of linear portion of pressuremeter curve
- SF shape factor for friction resistance
- T shear in pile at any depth z
- T_o shear in pile at ground surface
- U_o pore water pressure at test depth
- V_o initial pressuremeter probe volume before inflation

GLOSSARY OF TERMS (Con't)

- ΔV increase in volume from V_0
- V_1 volume of probe when p_{OH} is reached = $V_0 + \Delta V_0$
- ΔV_1 net increase in volume after $p_{OH} = \Delta V - \Delta V_0$
- ΔV_{OR} increase in pressuremeter probe volume to reach p_{OH} on the reload modulus
- X_a $\frac{\Delta V}{V_1}$ at point "a" on pressuremeter curve
- ΔX change in X over a specified increment
- y horizontal displacement of the pile
- z depth
- z_c critical depth
- z_{max} depth to maximum bending moment in the pile
- α rheological factor
- β Relative pile-soil stiffness term
- δ finite difference increment length
- χ influence factor of critical depth on pressuremeter
- ψ influence factor of critical depth on pile
- σ normal stress
- τ shear stress



CHAPTER 1. INTRODUCTION

1.1 The Phenomenon

One of the most obvious applications of the pressuremeter test is the solution of the problem of laterally loaded piles (4). The cylindrical expansion of the pressuremeter probe is analogous to the lateral movement of the pile (Fig. 1).

When a pile is loaded laterally there are several components to the soil resistance (Fig. 2): The front resistance due to normal stresses, σ_r , the friction resistance due to shear stresses, $\tau_{r\theta}$, the friction resistance due to shear stresses, τ_{rz} , the base friction resistance due to shear stresses, $\tau_{z\theta}$ and τ_{zr} , and the base moment resistance due to normal stresses, σ_z . Except for very short stubby piles ($D/B \leq 3$) the major components of soil resistance are due to σ_r and $\tau_{r\theta}$. At working loads the contribution due to the $\tau_{r\theta}$ effect may be as much as 50% of the total resistance (5). At any depth, z , the resultant of the above soil resistances is the P-y curve where P is the resultant soil resistance in force per unit length, and y is the horizontal displacement.

1.2 Existing Methods

At least seven methods can be identified to predict the top load-top movement of a laterally loaded pile on the basis of pressuremeter tests results. Methods 1 to 3 below make use of preboring pressuremeter results, while Methods 4 and 5 make use of selfboring pressuremeter results. The last two methods are the Briaud-Smith-Meyer method,

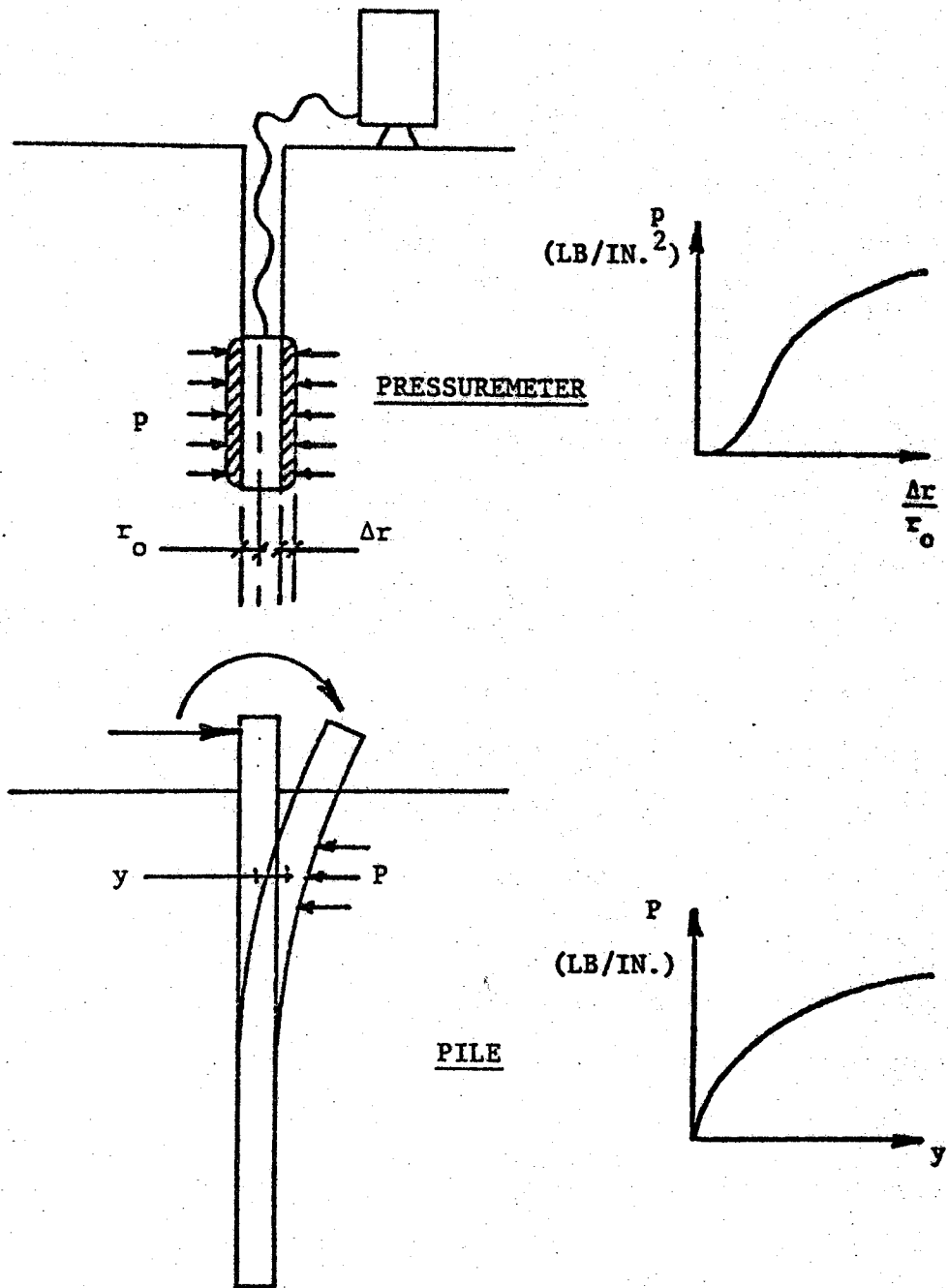


FIG. 1 - Pressuremeter-pile analogy

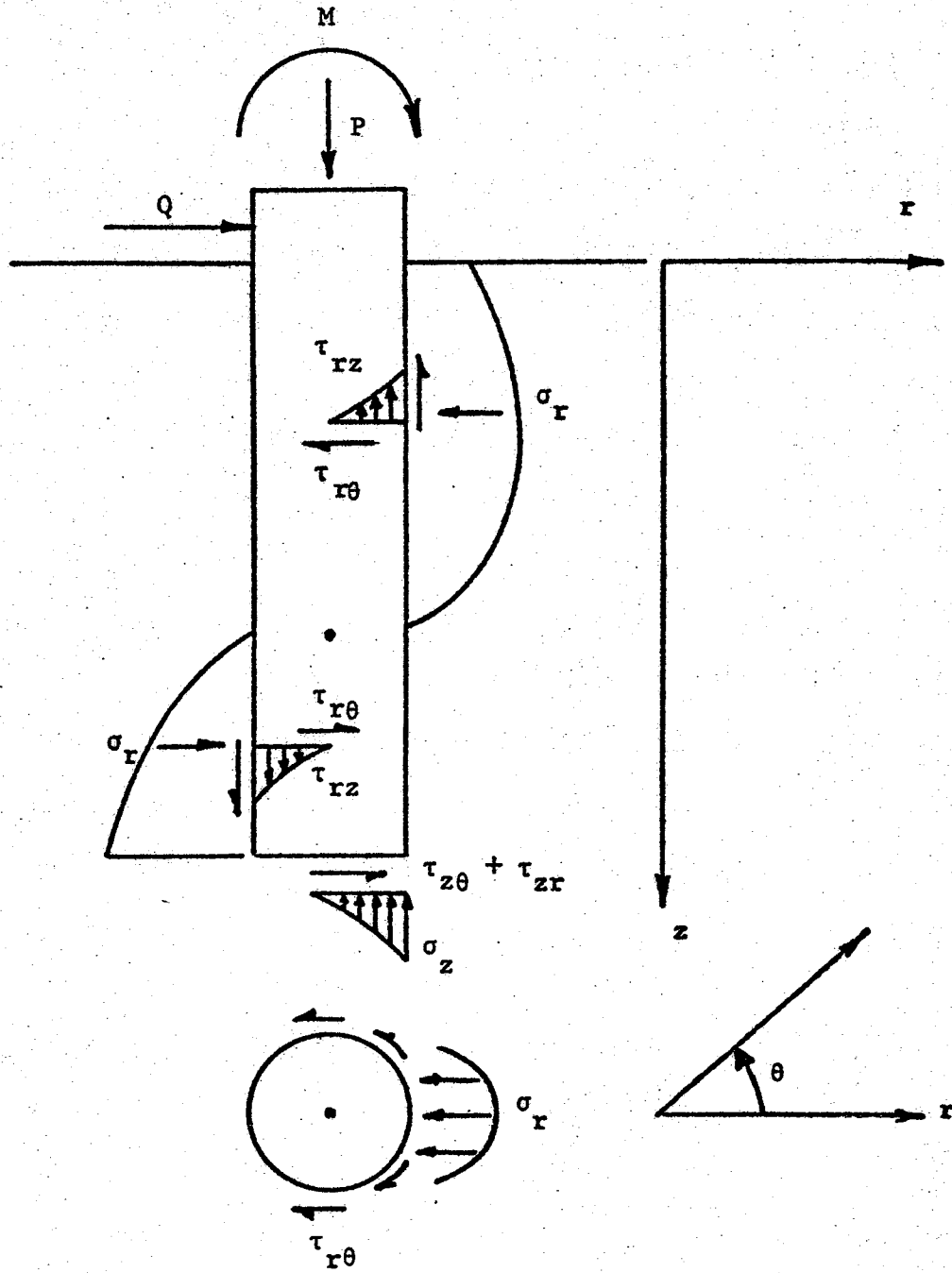


FIG. 2 - Components of soil resistance (after Reference 9)

and the Imai method. They require preboring pressuremeter results and are discussed in detail in Chapter 2.

Method 1 (15,2,10) considers the p-y curve to be bilinear elastic-plastic. The slope of the first linear portion of the curve is obtained from Menard's equation for the settlement of a strip footing (2). The second slope is half the first slope and the soil ultimate resistance, p_{ult} , is given by the pressuremeter limit pressure. The critical depth is handled as shown on Figure 3.

Method 2 (7,9) was developed for rigid drilled shafts and has the advantage of including all the components shown on Figure 2. The σ_r and $\tau_{r\theta}$ resistances are combined into one lateral resistance model which is a parabola cut off at p_{ult} obtained by Hansen's theory (11). The three other resistance models are elastic-plastic. The initial part of all models is correlated to the pressuremeter modulus. The ultimate values are obtained from the cohesion and friction angle of the soil. The critical depth effect is incorporated through Hansen's theory.

Method 3 (8) uses an elastic-plastic model for the frontal reaction. The slope of the elastic curve is obtained from the pressuremeter modulus and elasticity theory, while the ultimate value is considered to be the limit pressure from the pressuremeter. A friction model is also proposed, and the critical depth approach is the same as in Method 1.

Method 4 (1) uses the entire expansion curve from the self-boring pressuremeter as the p-y curve for the pile. Critical depth is handled as in Method 1.

$$S(R) = \frac{\text{Pressure at depth } z}{\text{Pressure at critical depth}}$$

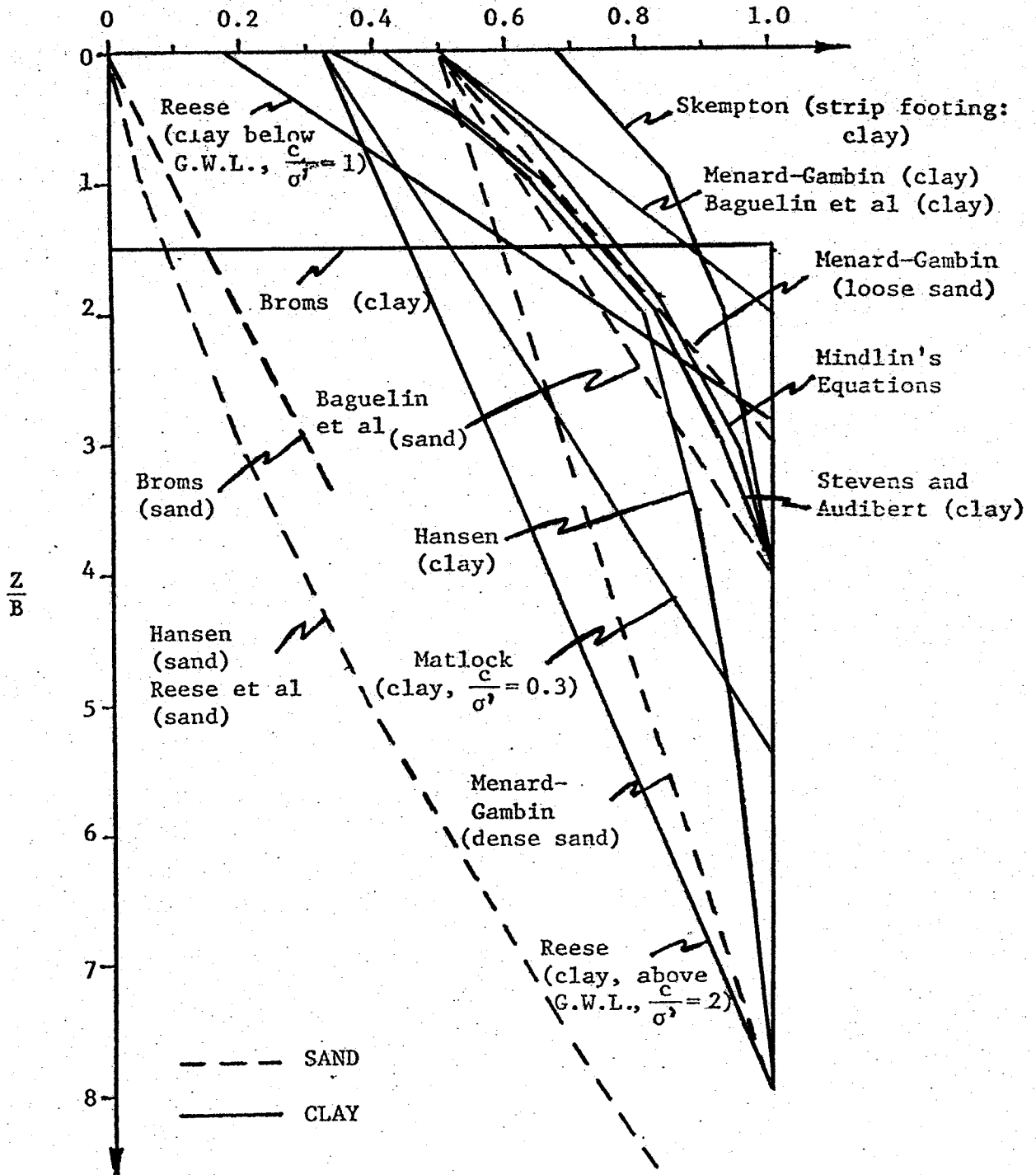


FIG. 3 - Critical depth - existing recommendations

Method 5 (12) uses the entire expansion curve from the selfboring pressuremeter, but multiplies all pressure ordinates by two before considering it as the pile p-y curve. No mention is made of the critical depth problem.

A comparison of these methods was made on one case history. The case history is a pile load test performed on the campus of Texas A&M University (14). The pile is a 3 ft (0.92 m) diameter reinforced concrete drilled shaft embedded 20 ft (6.10 m) in a stiff clay (Fig. 4). A horizontal load was applied at 2.5 ft (0.76 m) above the ground surface and was increased at the rate of approximately 5 tons (44.5 kN) per day. The load test result is shown on Figure 5.

The soil is a stiff clay with the following average characteristics: liquid limit 50%, plastic limit 20%, natural water content 25%, total unit weight 128 lb/ft³ (20.1 kN/m³). Unconfined compression tests values and miniature vane tests values were averaged to obtain the shear strength design profile shown on Figure 4. Pressuremeter tests were performed with a pavement pressuremeter (3) in a hand augered hole with no drilling mud. The net limit pressure, p_L^* , and the pressuremeter modulus, E_M , (2) are shown on Figure 4. Figure 5 shows the prediction according to Methods 1 through 3 and the Briaud-Smith-Meyer method.

$EI = 81 \times 10^6 \text{ ton. in}^2$
(Calculated)

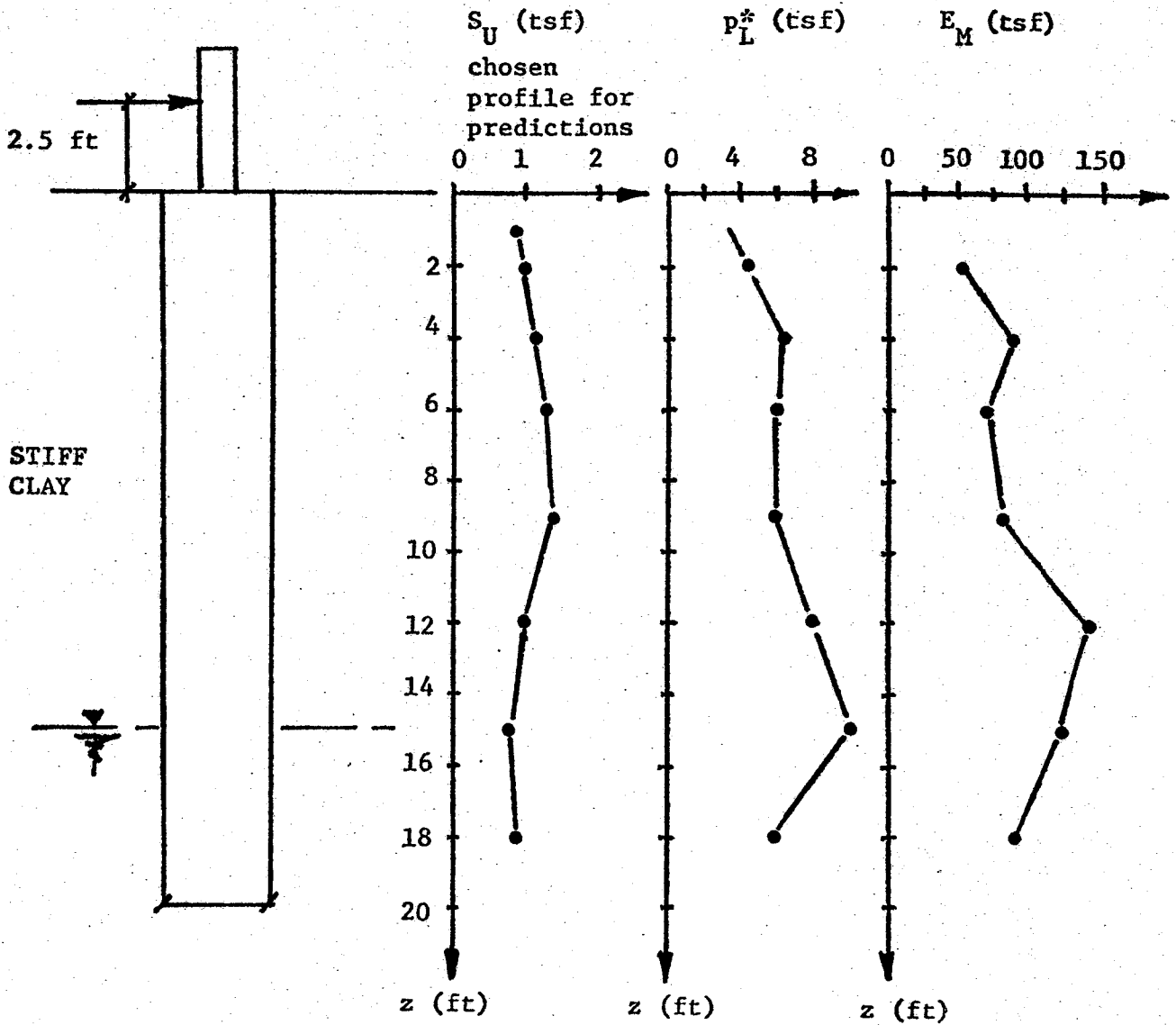


FIG. 4 - Soil properties: Texas A&M University Drilled Shaft

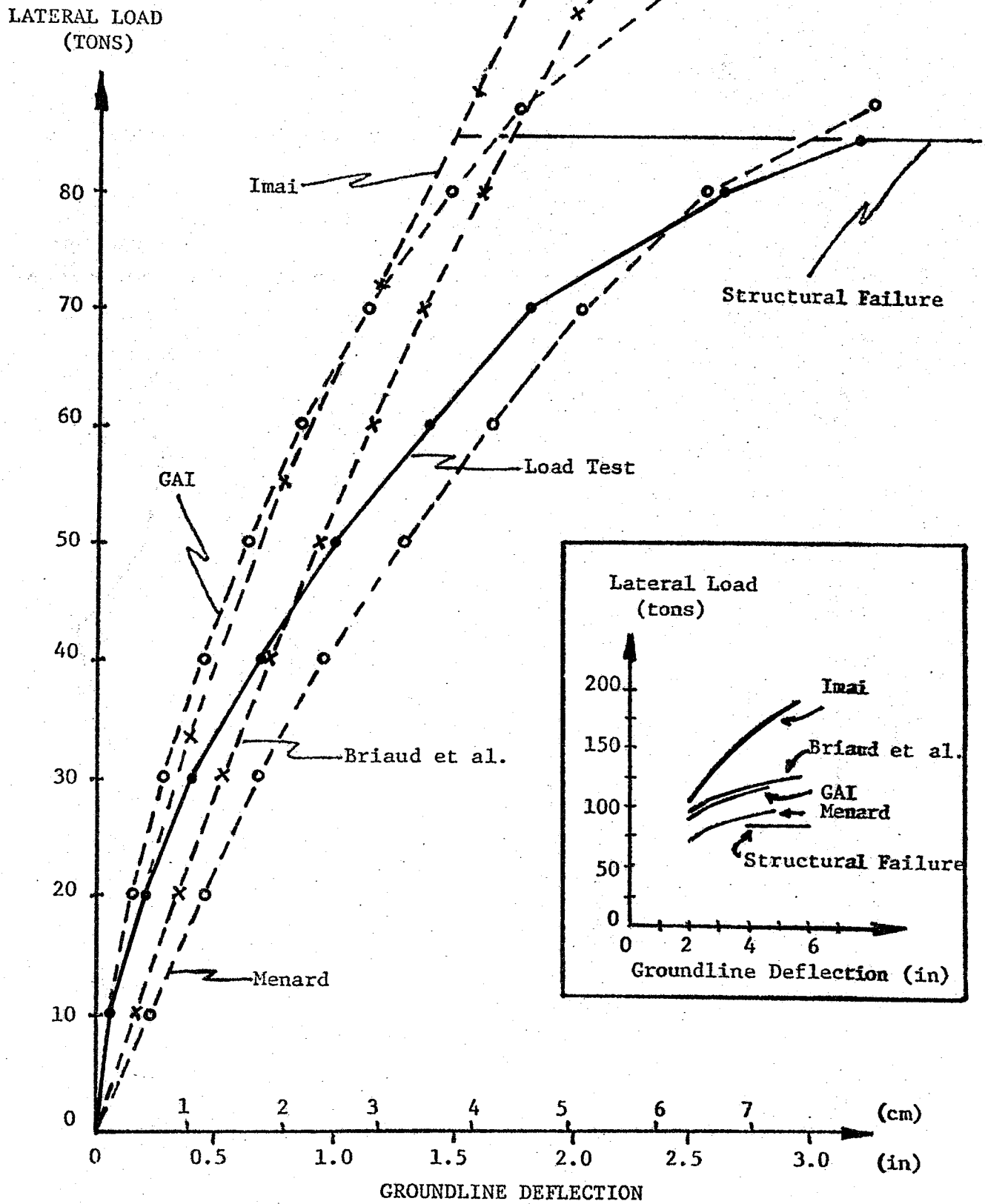


FIG. 5 - Comparison of predictions and measurement for the Texas A&M University Drilled Shaft

CHAPTER 2. DESIGN PROCEDURES

2.1 Briaud-Smith-Meyer Method

2.1.1 Introduction

In the analysis of a laterally loaded pile, the soil model is one of the key elements (6). The most commonly used method of analysis is the subgrade reaction method which is based on the solution of the governing differential equation by the finite difference method. The elementary soil model is the P-y curve which describes, at any depth z , how the soil resistance per unit length of pile P varies with the lateral displacement of the pile y .

2.1.2 The F-y/Q-y Mechanism

A vertically loaded pile derives its capacity from the point bearing capacity and from the friction along the pile shaft. The same two components, point bearing and friction, exist when a pile is loaded laterally. The point bearing will be called the front resistance Q and the friction resistance will be called F .

Fig. 6 gives an example which shows the distinct existence of the two components. A 3 foot diameter drilled shaft was loaded laterally in a stiff clay with an undrained shear strength from unconfined compression tests averaging 2000 psf. Pressure cells were installed along the shaft as shown on Fig. 6 in order to record the mobilization and distribution of the front pressure. The shaft was loaded and the resulting top load-top movement curve is shown on Fig. 6. At a horizontal load of 43 tons, the soil resistance due to front reaction was calculated from the pressure cell readings (5). Considering front

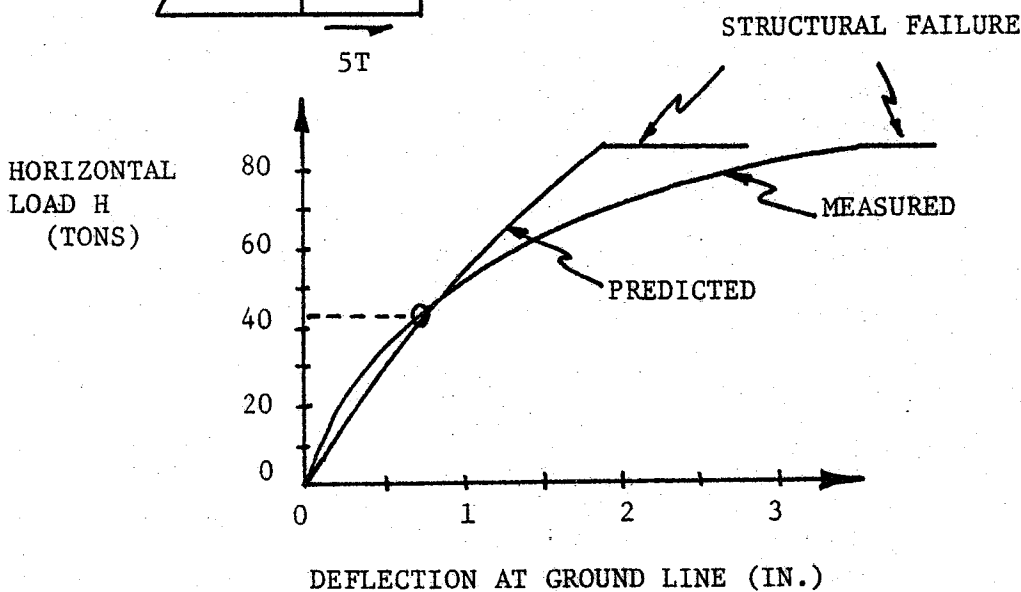
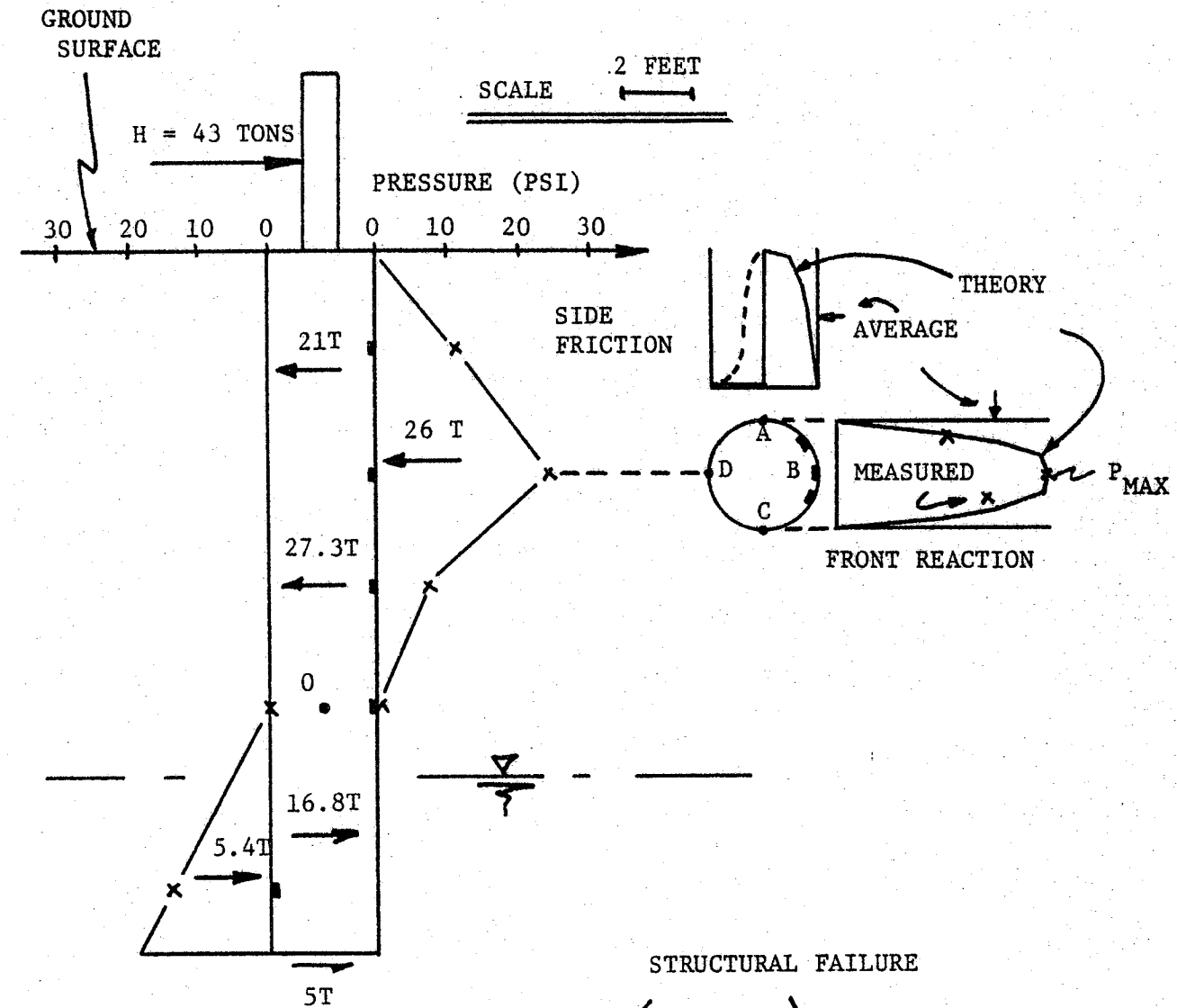


FIG. 6 - Example of Friction and Frontal Resistances

resistance only, horizontal and moment equilibrium cannot be obtained. After including the friction forces (Fig. 6) corresponding to the full shear strength of the stiff clay (5), both horizontal and moment equilibrium are approximately satisfied.

This example tends to indicate two points: 1. the friction resistance is an important part of the total resistance, 2. the friction resistance is fully mobilized before the front resistance. These two points verify that a fundamental soil model must distinguish between friction and front resistance and that at working loads the friction is all important.

2.1.3 The Proposed Method

The analogy of loading between the PMT and the pile is not complete and the pressuremeter curve is not identical to the P-y curve. It has been shown (5) that the pressuremeter curve gives the Q-y curve, and that the F-y curve can be obtained from the pressuremeter curve. The P-y curve is the addition of the F-y curve and the Q-y curve (Fig. 7). The following is a summary of the method which is proposed to obtain the Q-y and F-y model from the pressuremeter curve. More details and background on this method are presented in References 4, 5 and 6.

2.1.3.1 The pressuremeter curve

The pressuremeter curve is a plot of the pressure on the borehole wall on the vertical axis, and the increase in volume of the pressuremeter probe from the initial volume on the horizontal axis.

Figure 8 shows a typical pressuremeter curve with one unload-reload

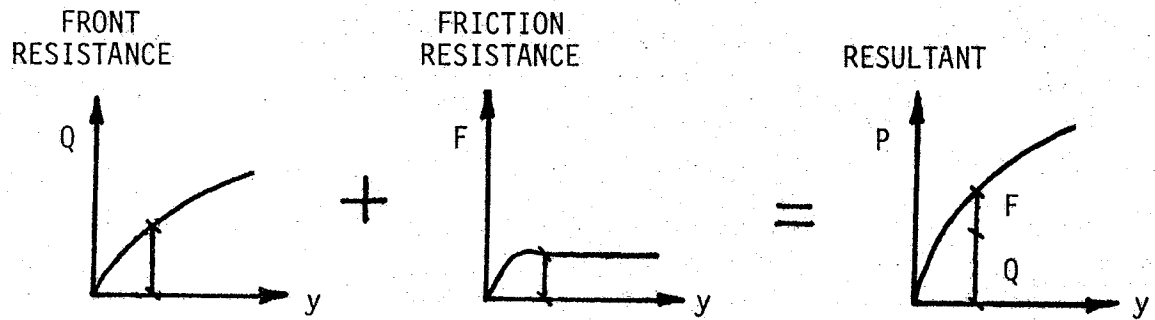
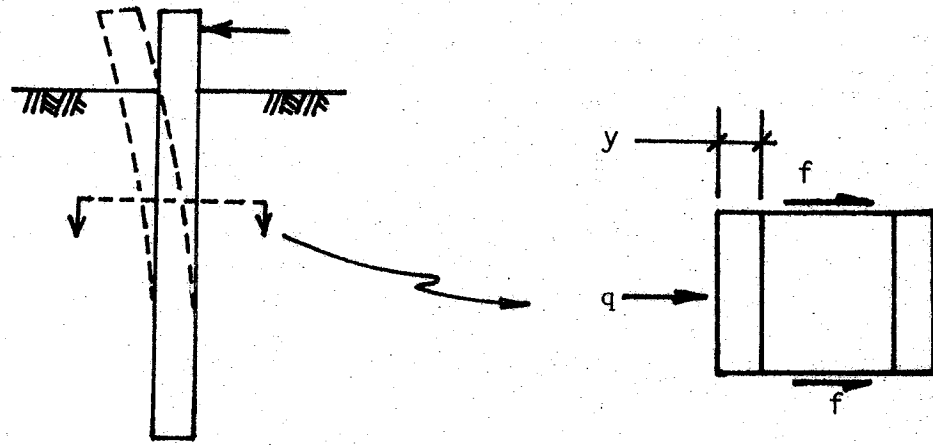


FIG. 7 - Lateral Load Mechanism

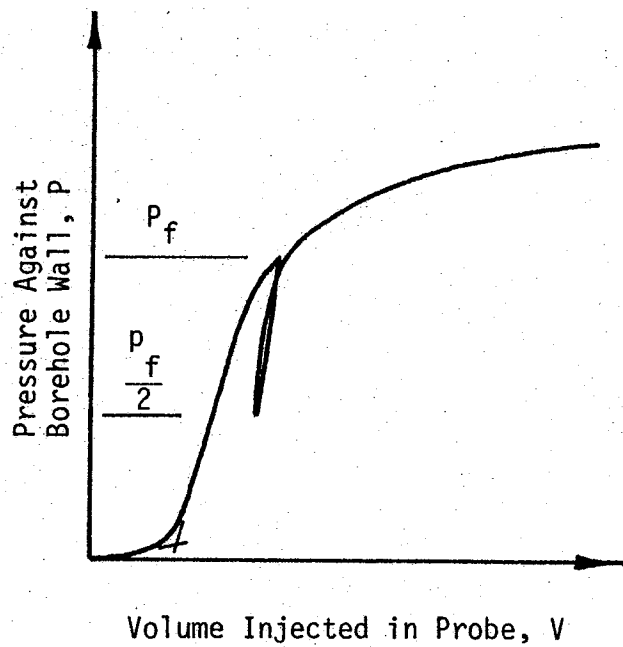


FIG. 8 - Typical Pressuremeter Test Curve with Unload-Reload Cycle

cycle. This cycle is necessary in the application of this method. The unloading should start at the end of the linear range of the pressure-meter test and continue until the pressure is reduced to one-half the pressure at the start of unloading (Fig. 8). At this point reloading is commenced and continues until the limit pressure is reached.

2.1.3.2 Total horizontal pressure at rest

The total horizontal pressure at rest, p_{OH} , may be calculated by considering the test depth, soil pressure, pore water pressure as:

$$p_{OH} = [(\sigma_{ov} - U_o)] \times K_{OH} + U_o \dots \dots \dots (1)$$

where, σ_{ov} = vertical total stress at test depth before test

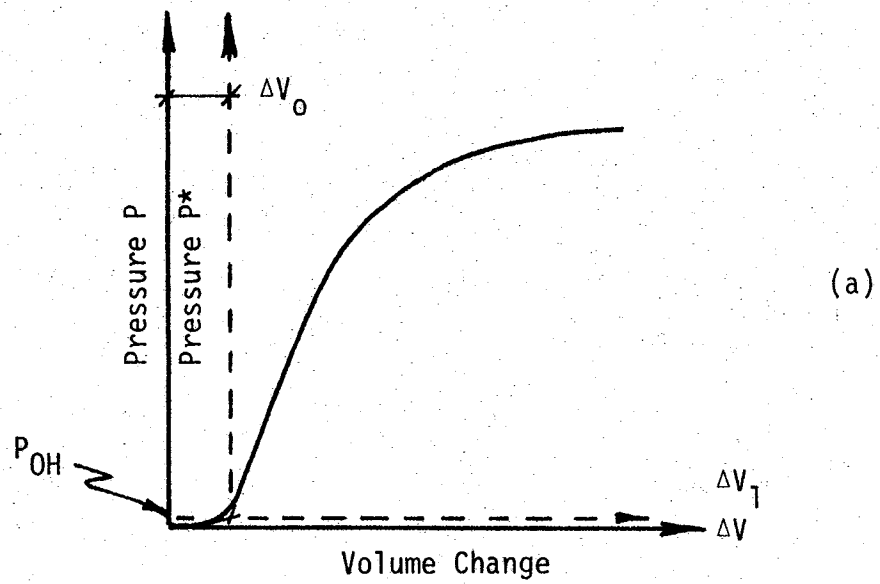
U_o = pore water pressure at test depth before test

K_{OH} = estimated coefficient of earth pressure at rest

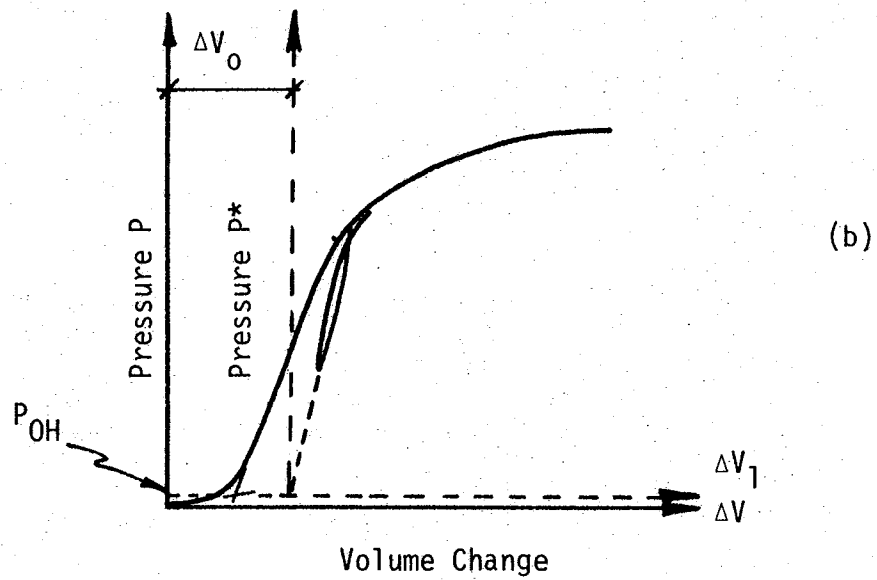
2.1.3.3 Translation of origin

To obtain a corrected curve the origin must be translated to correspond with p_{OH} . As shown in Fig. 9, the linear portion of the curve should be extrapolated back to p_{OH} , thus defining the new origin. If p_{OH} cannot be calculated by Eq. 1 it may be estimated graphically as shown in Fig. 9.

The reload cycle of a prebored test has been shown (16) to better approximate an undisturbed test and generate shear strength values in good agreement with laboratory values. The reload cycle should therefore be used to obtain the F-y curve for all piles, both driven and augered. For bored piles, or piles driven open ended which do not plug, the front reaction, Q-y curve is developed from the initial curve



(a)



(b)

FIG. 9 - Translation of Origin of Pressuremeter Curve

of a prebored test. For full displacement piles the reload cycle is used for the front resistance. This is summarized as follows:

Curve	Pile Type	
	Driven	Bored
F-y	Reload cycle	Reload cycle
Q-y	Reload cycle	Initial curve

When the reload cycle is used, the linear range is extrapolated back to p_{OH} to obtain the full curve (Fig. 9).

The notation used to define these curves is as follows:

p = pressuremeter pressure

$p^* = p - p_{OH}$

= net pressuremeter pressure

p_{OH} = horizontal earth pressure at rest

V_0 = initial probe volume before inflation

ΔV = increase in volume from V_0

ΔV_0 = increase in volume to reach p_{OH}

$V_1 = V_0 + \Delta V_0$ = volume of probe when p_{OH} is reached.

$\Delta V_1 = \Delta V - \Delta V_0$ = net increase in volume after p_{OH}

R_1 = pressuremeter radius corresponding to the probe volume

V_1

ΔR_1 = increase in radius corresponding to the increase in volume ΔV_1

2.1.3.4 Critical depth for the pressuremeter

The pressuremeter is subject to a reduction in the mobilized resistance at shallow depth. The reduction factor is shown in Fig. 10 as a function of the ratio of the test depth, z , to the critical depth,

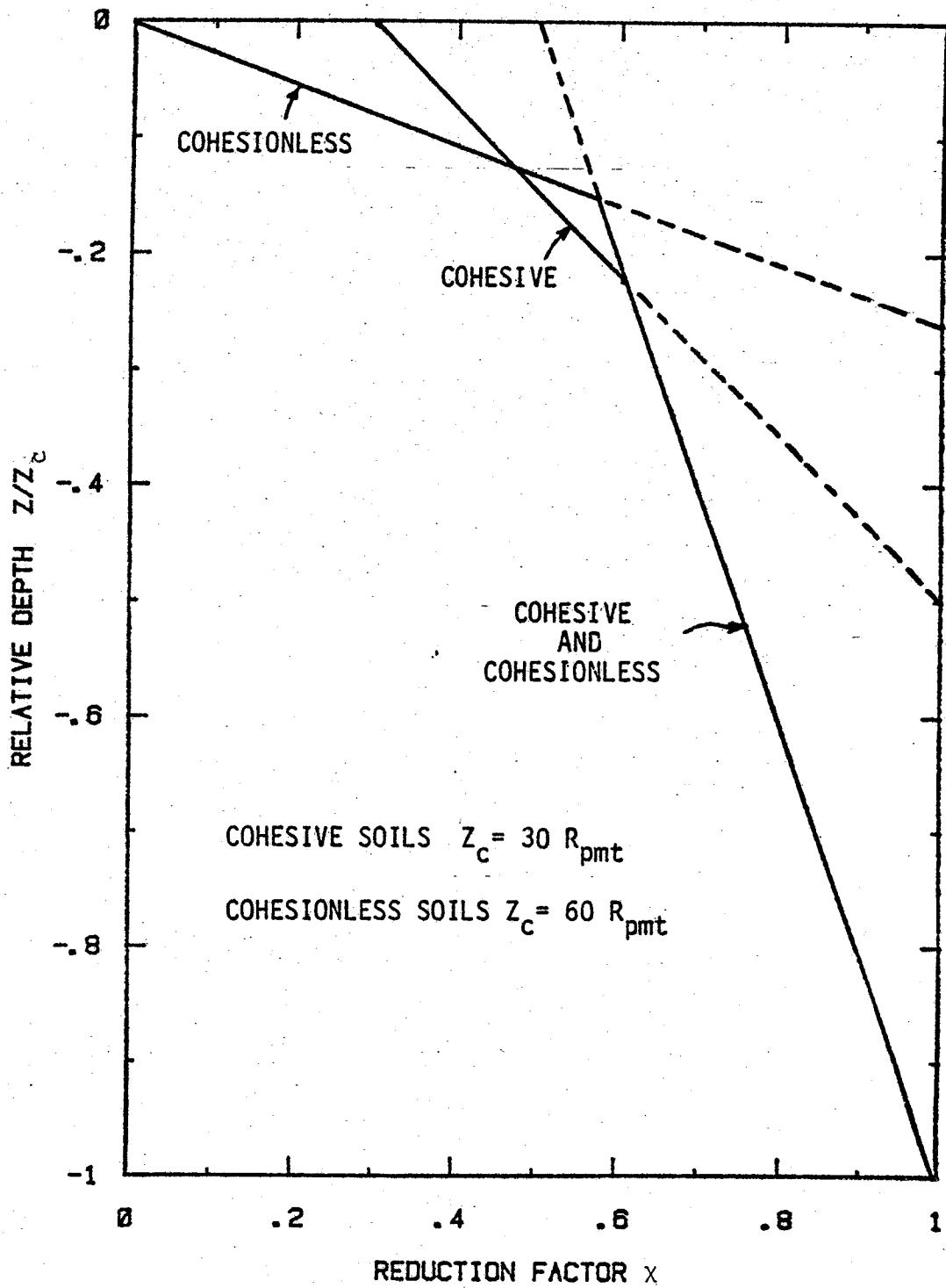


FIG. 10. - Proposed Reduction Factor for the Pressuremeter within the Critical Depth

z_c . The critical depth as recommended by Baguelin et al. (2) is

$$z_c = 30 R_{PMT} \text{ for cohesive soils}$$

$$z_c = 60 R_{PMT} \text{ for cohesionless soils}$$

where R_{PMT} = pressuremeter radius

The pressuremeter curve is corrected by taking

$$p_{corr} = \frac{p^*}{\chi}$$

where p_{corr} = corrected net pressure

χ = reduction in mobilized pressuremeter

pressure at all strains. $\beta = 1$ below the

pressuremeter critical depth.

This curve is then used to obtain the Q-y and F-y curves.

2.1.3.5 Front resistance

The front resistance of the pile, Q, is calculated by:

$$Q = \frac{p^*}{\chi} \times B \times SQ \dots \dots \dots (2)$$

SQ = pile shape factor = 1 for square piles loaded
parallel to their sides

SQ = pile shape factor = 0.80 for round piles and square piles not loaded parallel to their sides.

B = pile diameter, or width

2.1.3.6 Accounting for the critical depth for the pile

To account for the reduced soil reaction mobilized within the piles critical depth, the front reaction curve, Q, is multiplied by a reduction factor, ψ . Therefore Eq. 2 becomes

$$Q = \frac{P^*}{\chi} \times SQ \times B \times \psi \dots \dots \dots (3)$$

The reduction factor, ψ , is given on Fig. 11. The average critical depth for the pile, $Z_{c(av)}$ is a function of the relative pile to soil stiffness and is given by Eq. 4.

$$Z_{c(av)} = \frac{\pi}{4} (RR-5)(B) \dots \dots \dots (4)$$

or $Z_{c(av)} = B$, whichever is greater.

The relative rigidity factor, RR, is given by Eq. 5.

$$RR = \frac{1}{B} \sqrt[4]{\frac{EI}{P_L^*}} \dots \dots \dots (5)$$

where EI = pile flexural stiffness
 P_L^* = net pressuremeter limit pressure

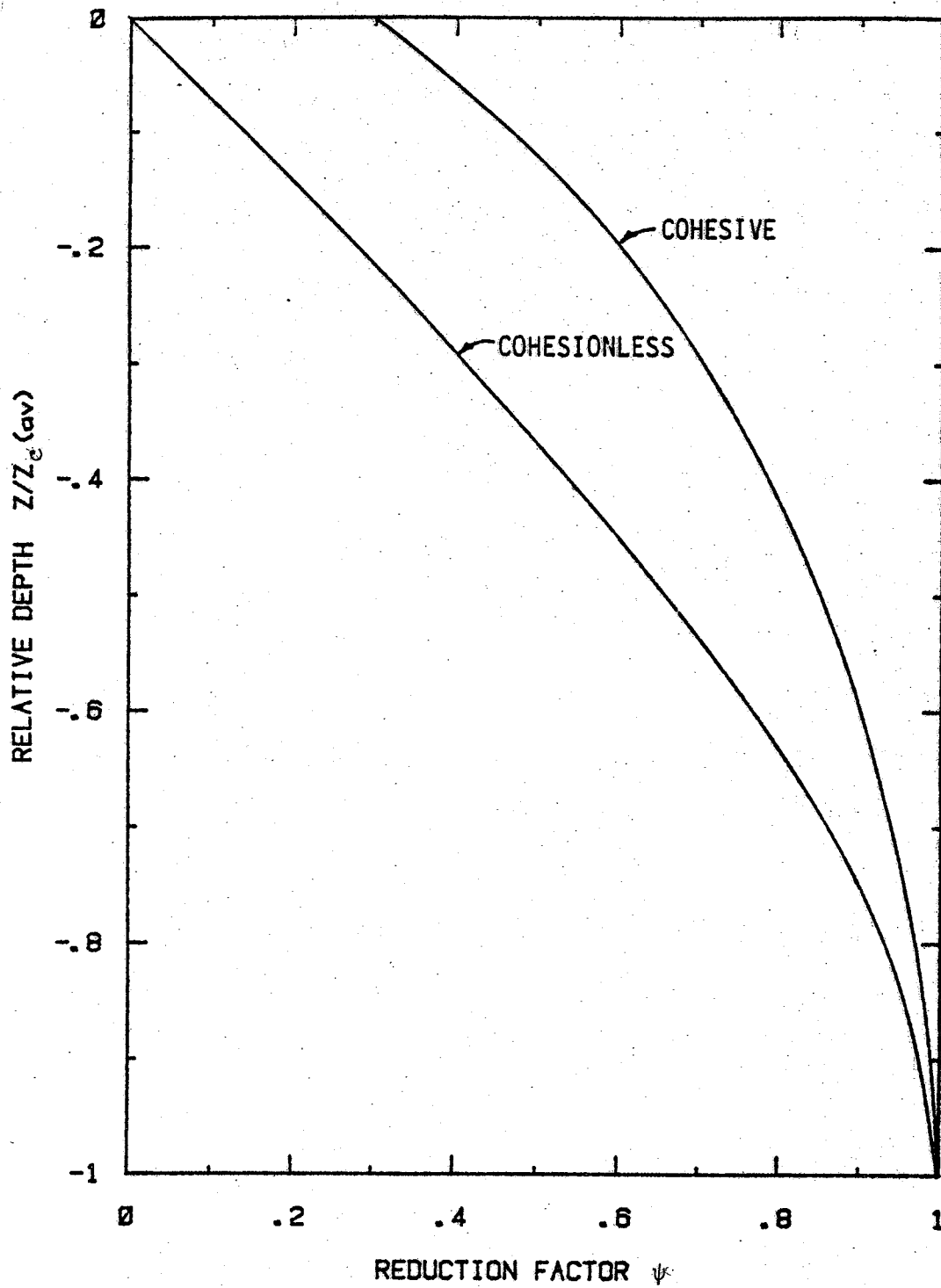


FIG. 11 - Proposed Reduction Factor for the Pile within the Critical Depth

The correlation between RR and $Z_{c(av)}/B$ is shown in Fig. 12 with measured data also plotted.

2.1.3.7 Pile displacement

Having translated the origin of the pressuremeter curve (Fig. 9), the change in volume must first be converted to a relative increase in pressuremeter radius. Assuming small strain conditions exist:

$$\frac{\Delta R_1}{R_1} = \frac{1}{2} \frac{\Delta V_1}{V_1} \dots \dots \dots (6)$$

The pile displacement, y, is then calculated by

$$\frac{y}{R_{pile}} = \frac{\Delta R_1}{R_1} \dots \dots \dots (7)$$

2.1.3.8 Friction resistance

Determine the friction resistance by the following procedure. The slope of the curve at a point is assumed to be the slope of the line joining the point before and the point after the point considered (Fig. 13).

Calculate the slope of the curve by:

$$\frac{\Delta P^*}{\Delta X} = \frac{P_a^* - P_b^*}{X_a - X_b} \times \frac{1}{\chi} \dots \dots \dots (8)$$

where $P_a = p^*$ for the point after the point considered
 $P_b = p^*$ for the point before the point considered

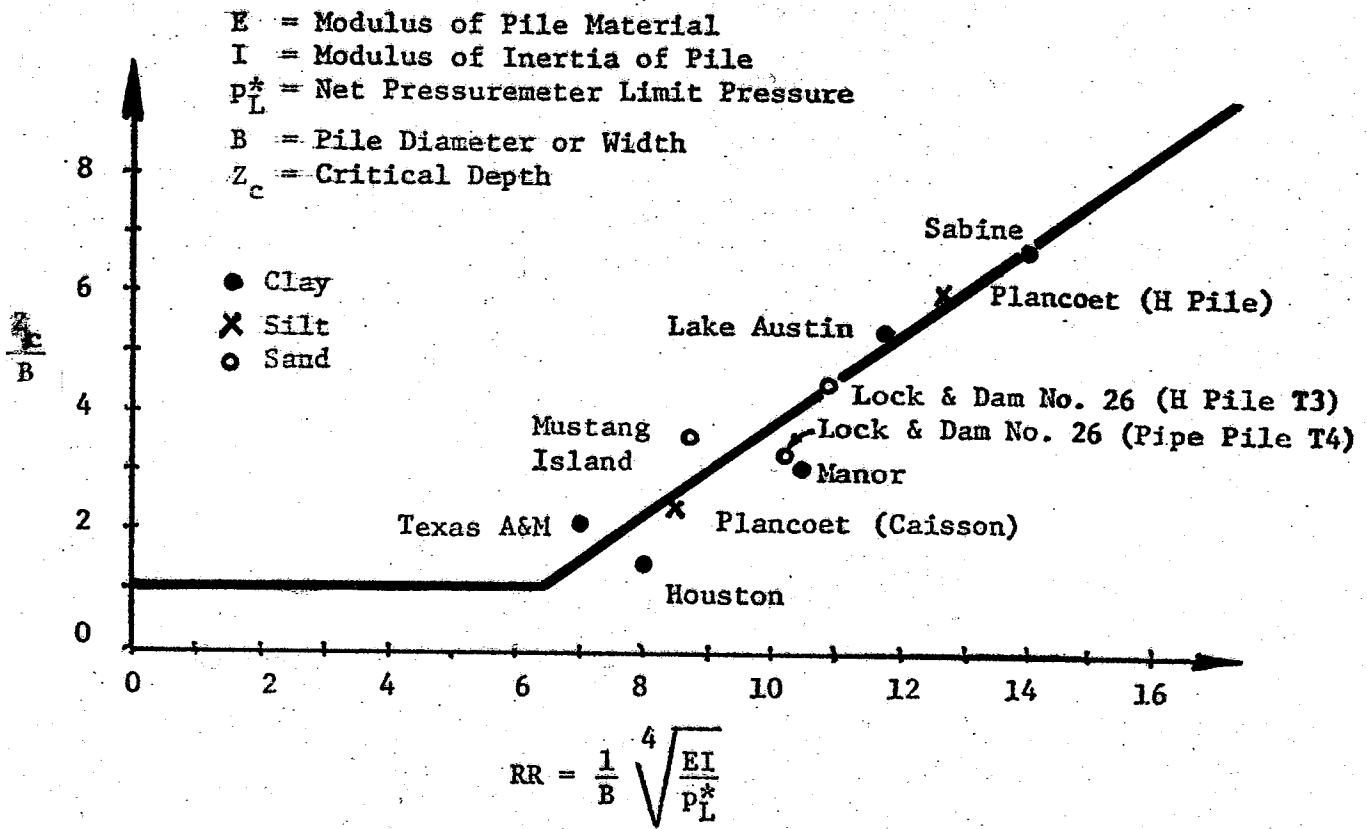


FIG. 12- Critical Depth as a Function of Relative Rigidity

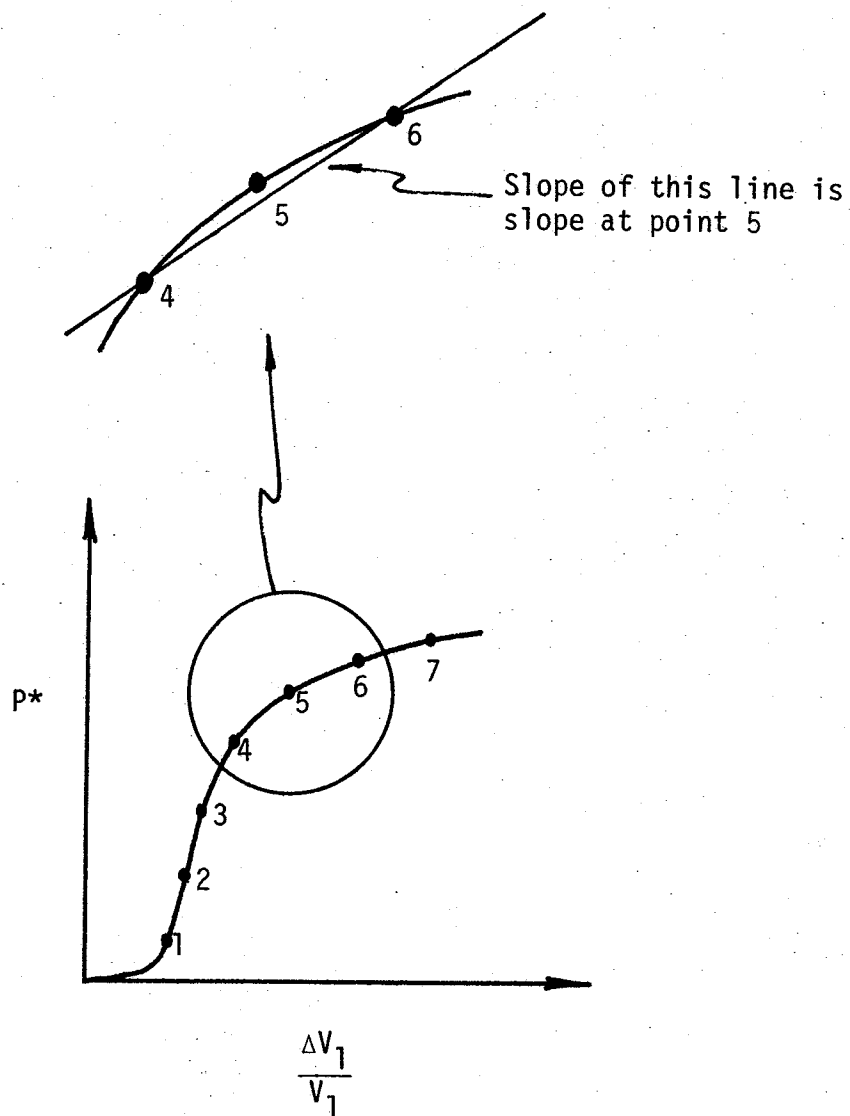


FIG. 13 - Determining the Slope

$$X_a = \frac{\Delta V_1}{V_1} \text{ for the point after the point considered}$$

$$X_b = \frac{\Delta V_1}{V_1} \text{ for the point before the point considered}$$

$$\frac{\Delta P^*}{\Delta X} = \text{slope of the curve at the point considered}$$

χ = reduction factor for pressuremeter critical depth.

The shear stress, τ , mobilized by the pile is calculated from the slope of the curve by:

$$\tau = X (1+X) \frac{\Delta P^*}{\Delta X} \times \frac{1}{\chi} \dots \dots \dots (9)$$

where $X = \Delta V_1/V_1$ for the point considered

The friction resistance, F , mobilized on the pile is then determined as:

$$F = \tau x B x SF \dots \dots \dots (10)$$

where $SF = \text{shape factor} = 2$ for square piles loaded parallel to their sides
 $= 1$ for round piles and square piles not loaded parallel to their sides.

Note that no pile critical depth reduction factor is applied to the friction component.

2.1.3.9 Total resistance

The total resistance of the pile is calculated by:

$$P = F + Q \dots \dots \dots (11)$$

where P = total resistance of pile

2.1.3.10 Base resistance on a rigid pile

The mobilization of shear resistance upon the base of a rigid rotating pile, may be significant. The shear stress is assumed to be mobilized linearly and to reach the shear strength at a translation of 0.1 in. (2.5 mm). If the program used is not equipped with a separate base friction model, the base friction curve can be added to the deepest p-y curve as follows:

$$F_b = C_u \frac{A_p}{\delta} \dots \dots \dots (12)$$

where F_b = base mobilized resistance

δ = finite difference increment length

A_p = base area

The units of B are therefore force/unit length, and consistent with those of Q and F . The base P-y curve only is then given by

$$P = Q + F + F_b \dots \dots \dots (13)$$

2.2 Briaud-Smith-Meyer Simplified Method: Subgrade Modulus Approach

For small strains, the problem of laterally loaded piles may be modelled by elasticity. The result is a method that has the advantage of being simple enough to be used without the help of a computer (10). The method uses a linear P-y curve.

2.2.1 Obtaining the Modulus of Subgrade Reaction k

The linear portion of the pressuremeter curve may be used to obtain a modulus of subgrade reaction. By performing the steps of the Briaud-Smith-Meyer method for one point on the linear portion of the pressuremeter curve, such as point A or B in Fig. 14, a linear elastic P-y curve is obtained. The slope of this linear P-y curve divided by the diameter B of the pile gives a modulus of subgrade reaction, k.

Thus,

$$k = \frac{P/B}{y} \dots \dots \dots (14)$$

where p and y are a coordinate pair on the P-y curve. Such a modulus may be obtained from either the initial or reload cycle of the pressuremeter curve depending on the criteria given in Section 2.1.3.3. Referring to Figure 14 the modulus of subgrade reaction k is given by:

$$k = \frac{P_A^*}{R_{pile}} \times \frac{2}{\chi} \left(\frac{\psi SQ(V_o + \Delta V_o)}{\Delta V_A - \Delta V_o} + \frac{SF(V_o + \Delta V_{OR})}{\Delta V_B - \Delta V_{OR}} \right) \dots \dots \dots (15)$$

for bored piles and unplugged driven piles

$$k = \frac{P_A^*}{R_{pile}} \times \frac{2}{\chi} \frac{(\psi SQ + SF)(V_o + \Delta V_{OR})}{\Delta V_B - \Delta V_{OR}} \dots \dots \dots (16)$$

for plugged driven piles

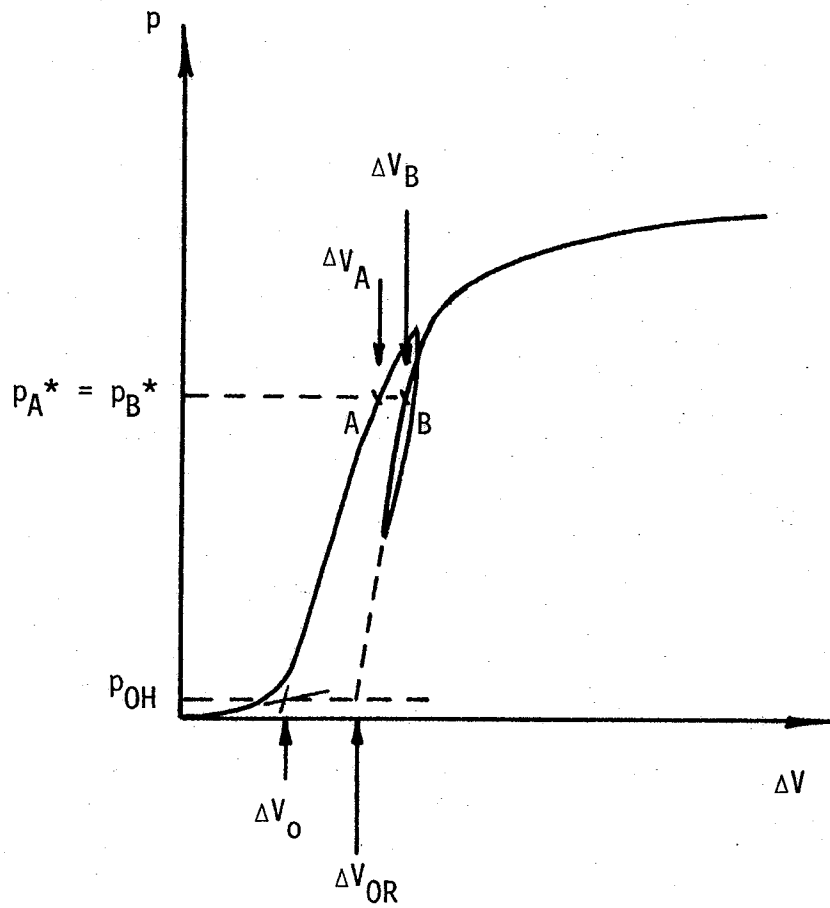


FIG. 14 - Parameter Definition for Calculation of the Subgrade Modulus

where p_A , V_O , ΔV_O , ΔV_{OR} , ΔV_A , ΔV_B are defined on the pressure-meter curve of Fig. 14.

R_{pile} is the pile radius

ψ , χ are the pile and pressuremeter critical depth correction factors (see Section 2.1.3.6 and 2.1.3.4)

SQ , SF are the front resistance and friction shape factors (see Section 2.1.3.5 and 2.1.3.8)

Eqs. 15 and 16 are a simplification of the more detailed method previously presented. The more detailed method has been checked for piles up to 36 in. in diameter. The modulus of subgrade reaction obtained by the above calculations corresponds to deflections of the order of 1 to 2% of the pile diameter. For larger deflections the value of k is obtained from the pressuremeter curve at correspondingly higher values of p_A^* , ΔV_A , ΔV_B .

2.2.2. Zone of Influence for the Modulus of Subgrade Reaction

This method is primarily for use in homogenous soils where k is fairly constant with depth. If k is not constant with depth an average value is taken over a depth of 3 times the transfer length, ℓ_o , (where transfer length is as defined in the next section). Since the transfer length is a function of k this is an iterative procedure but convergence is accomplished fairly quickly. A good starting point is to take an average k value over 5 pile diameters below the ground surface. Calculate the transfer length with this k value and compare $3\ell_o$ to the depth over which k was averaged. If they are not the same compute an average k over the depth $3\ell_o$ below the ground surface. Continue until $3\ell_o$ matches the depth over which k was averaged to calculate ℓ_o . In layered soils the minimum of the above k value and

the k value obtained within 5 pile diameters is to be used.

2.2.3 Calculating the Transfer Length l_0

The transfer length is defined as:

$$l_0 = \sqrt[4]{\frac{2EI}{kR}} \dots \dots \dots (17)$$

The transfer length is used in the solution to the governing differential equation. It is generally accepted that if

$$\frac{h}{l_0} \geq 3 \quad \text{pile is flexible}$$

$$\frac{h}{l_0} \leq 1 \quad \text{pile is rigid.}$$

where h is the pile length.

2.2.4 Calculating the Deflection y_0 at the Ground Surface

The solution to the governing differential equation is (1):

$$y = \frac{2T_0}{l_0 kB} e^{-z/l_0} \cos \frac{z}{l_0} + \frac{2M_0}{l_0^2 kB} e^{-z/l_0} \left(\cos \frac{z}{l_0} - \sin \frac{z}{l_0} \right) \dots (18)$$

where y is the lateral deflection of the pile at a depth z

T_0 , M_0 are the shear and bending moment at the
ground surface

B is the pile diameter or width.

2.2.5 Calculating the depth z_{\max} to maximum bending moment, M_{\max} .

The shear T in the pile at any depth z is given by:

$$T = T_0 e^{-z/\ell_0} \left(\cos \frac{z}{\ell_0} - \sin \frac{z}{\ell_0} \right) - \frac{2M_0}{\ell_0} e^{-z/\ell_0} \sin \frac{z}{\ell_0} \dots (19)$$

The solution of the equation $T(z) = 0$ gives M_{\max} .

2.2.6 Calculating the maximum bending moment M_{\max}

$$M = T_0 \ell_0 e^{-z/\ell_0} \sin \frac{z}{\ell_0} + M_0 e^{-z/\ell_0} \left(\cos \frac{z}{\ell_0} + \sin \frac{z}{\ell_0} \right) \dots (20)$$

M_{\max} is obtained for $z = z_{\max}$.

2.2.7 What if the pile had been rigid?

In this case (1): $p = ky = Rz + S$ and $y' = R/k$

The shear at any depth is: $T = T_0 - \int_0^z pBdz$

The bending moment at any depth is: $M = M_0 + T_0z - \int_0^z pB(x-z)dz$

Then $T = T_0 - RB \frac{z^2}{2} - SBz \dots (21)$

$$M = M_0 + T_0z - RB \frac{z^3}{6} - SB \frac{z^2}{2} \dots (22)$$

If h is the embedded length of the pile,

for $z = h$, $T = 0$ and $M = 0$; this gives R and S

$$R = - \frac{6(hT_o + 2M_o)}{h^3 B} \quad S = \frac{2(2hT_o + 3M_o)}{h^2 B}$$

M_{\max} is obtained for $T = 0$:

$$T_o - RB \frac{z^2}{2} - SBz = 0$$

but $T_o = RB \frac{h^2}{2} + SB$

so $RB \frac{h^2}{2} + SBh - RB \frac{z^2}{2} - SBz = 0$

which gives $z_{\max} = - \frac{2S}{R} - h \dots \dots \dots (23)$

and then M_{\max} is obtained from the moment equation.

2.2.8 Alternate method of obtaining k - Menard-Gambin Method

Another method of obtaining k has been proposed (10). A profile of elastic moduli, E, is obtained from the pressuremeter test curves.

The value of k is then obtained from:

for $R > 0.30^m$, $\frac{1}{k} = \frac{1.33}{3E} R_o \left(\frac{R}{R_o} \times 2.65\right)^\alpha + \frac{\alpha}{3E} R \dots \dots \dots (24)$

for $R \leq 0.30^m$, $\frac{1}{k} = \frac{1.33}{3E} (2.65)^\alpha + \frac{\alpha}{3E} R \dots \dots \dots (25)$

where E is the average pressuremeter modulus

$$R_0 = 0.30^m$$

R is the pile radius

α is a rheological factor (see Figs. 15 and 16).

The depth over which k is averaged is found as detailed in Section 2.2.2.

2.3 Imai's Method

**Note: Imai's method (13) makes use of some empirical equations which were derived using SI units; there, English units are not applicable and must be converted. The final result, however, a load-deflection curve, will be converted into kips and inches.

2.3.1 The pressuremeter curve

The pressuremeter curve is a plot of the pressure on the borehole wall on the vertical axis, and the volume injected into the probe on the horizontal axis (Fig. 8). For this method, the initial volume and length of the inflatable part of the probe must be known to calculate the pressuremeter radius at anytime during inflation:

$$r = \sqrt{\frac{V_0 + \Delta V}{\pi L}} \dots \dots \dots (26)$$

where V_0 = initial deflated volume of pressuremeter probe

ΔV = volume injected into pressuremeter probe

L = length of inflatable part of pressuremeter probe

r = corresponding radius value.

Soil Type	Peat		Clay		Silt		Sand		Sand and Gravel	
	E_m/p_l^*	α	E_m/p_l^*	α	E_m/p_l^*	α	E_m/p_l^*	α	E_m/p_l^*	α
Over-consolidated			>16	1	>14	2.3	>12	1/2	>10	1/2
Normally Consolidated		1	9-16	2/3	8-14	1/2	7-12	1/3	6-10	1/4
Weathered and/or remoulded			7-9	1/2		1/2		1/3		1/4

Fig. 15 - Values of the Parameter α for Soil

Rock	Extremely fractured	Other	Slightly fractured or extremely weathered
	$\alpha = 1/3$	$\alpha = 1/2$	$\alpha = 2.3$

Fig. 16 - Values of the Parameter α for Rock

2.3.2 Soil stiffness value k_m

To calculate the soil stiffness value, k_m , use the linear portion of the pressure-radius curve (Fig. 17). The k_m value is calculated by (Fig. 17):

$$k_m = \frac{p_f - p_o}{r_f - r_o} \dots \dots \dots (27)$$

- where p_o = beginning pressure on linear portion (units of kg/cm^2)
- p_f = final pressure on linear portion (units of kg/cm^2)
- r_o = beginning radius of probe on linear portion (units of cm)
- r_f = final radius of probe on linear portion (units of cm)
- k_m = soil stiffness value (units of $\text{kg}/\text{cm}^2/\text{cm}$).

2.3.3 Specific stiffness value k_o

At each pressuremeter test depth, the specific k-value, k_o , must be determined. The radius at the mid-point of the range used to determine k_m is calculated by:

$$r_m = \frac{r_o + r_f}{2} \dots \dots \dots (28)$$

where r_m = radius of the probe at mid-point of linear portion. The k_o value is then calculated by:

$$k_o = \frac{\pi}{2} \sqrt[4]{2 r_o (r_m - r_o)^2 \cdot k_m} \dots \dots \dots (29)$$

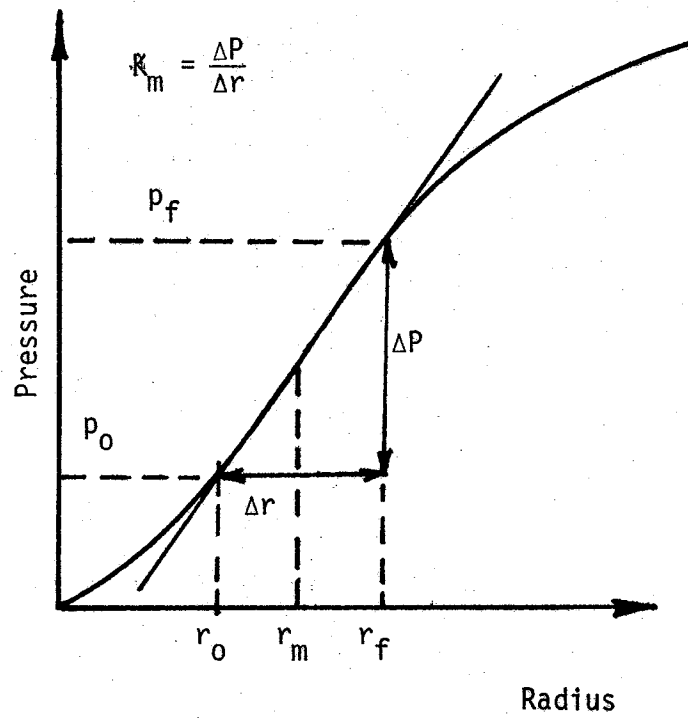


FIG. 17 - Pressure vs Radius Curve to Determine K_m

2.3.4 Soil layer thickness

Determine the thickness of each soil layer by considering that a layer boundary exists at the mid-point between two consecutive pressuremeter tests depths.

2.3.5 Equivalent stiffness value \bar{k}_o

Calculate the equivalent k_o -value, \bar{k}_o , as the arithmetic average of the k_o values weighted according to the corresponding soil layer thicknesses:

$$\bar{k}_o = \frac{\Sigma[(\text{soil layer thickness}) \times (k_o)]}{\Sigma(\text{soil layer thickness})} \dots \dots \dots (30)$$

2.3.6 Pile parameters

Determine the parameters E, I, B, ℓ , f for the pile, where:

E = Young's Modulus of pile material (kg/cm²)

I = geometrical moment of inertia (cm⁴)

B = pile diameter, or width (cm)

ℓ = distance between ground surface and point of application of lateral load (cm)

f = parameter for quantifying the fixity of the pile top.

The degree of fixity f is defined as the ratio of the actual bending moment, M, to the bending moment corresponding to a completely

fixed connection, M_f .

$$f = \frac{M}{M_f} \dots \dots \dots (31)$$

Thus, when the pile top is completely free, $f = 0$, and when it is absolutely fixed, $f = 1$.

	FIXED	FREE
f-value	1	0

2.3.7 Basic stiffness value K_o

Calculate the basic K-value, K_o , by:

$$K_o = \frac{\bar{k}_o}{\sqrt[4]{B}} \dots \dots \dots (32)$$

2.3.8 Design stiffness value, K

To obtain the horizontal load which corresponds to an arbitrary displacement y , calculate the design K-value, K , by:

$$K = \frac{K_o}{\sqrt{y}} \dots \dots \dots (33)$$

where y = arbitrary displacement (cm)

2.3.9 Horizontal load calculations

Determine the horizontal load for a chosen displacement y by the following calculations:

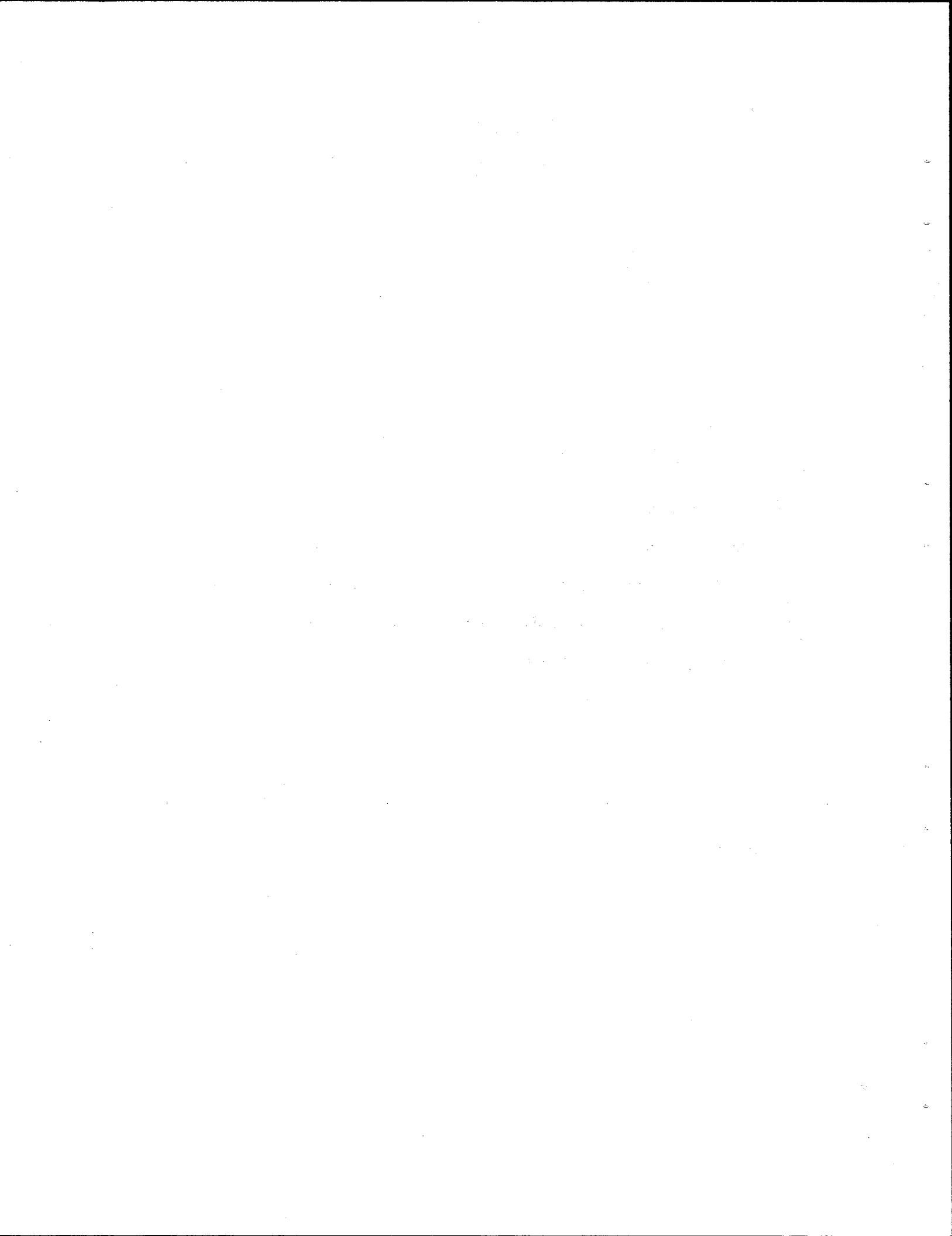
$$(a) \beta = \sqrt[4]{\frac{KB}{EI}} \dots \dots \dots (34)$$

(b) Horizontal load, H:

$$H = \frac{12EI\beta^3}{[(4-3f)(1+\beta)^3+2]} y \dots \dots \dots (35)$$

2.3.10 Plot H-vs-y curve

Repeating the calculations of Eqs. 33, 34, and 35 for various values of displacement, y , leads to a load-displacement curve. The values of horizontal load, H, from Eq. 35 are plotted versus the chosen values of horizontal displacement, y .



CHAPTER 3. DESIGN EXAMPLE: MUSTANG ISLAND

3.1 Soil and pile test information

Two test piles were loaded laterally in sand at a site on Mustang Island near Corpus Christi, Texas (17). The sand at the test site varied from clean fine sand to silty fine sand, both of high relative densities. The angle of shearing friction, ϕ , was 39 degrees from SPT correlations and the submerged unit weight, γ' , was found to be 66 lbs/ft³ (10.5 kN/m³).

The lateral load tests were performed within a 5.5 ft (1.9 m) deep pit with the water table maintained at, or slightly above, the test pit bottom. The steel test piles were 24 in. (610 mm) in diameter with a wall thickness of 3/8 in. (10 mm), and instrumented with strain gages. The piles were driven to a total embedded depth of 69 ft (21 m) with 9 ft (2.7 m) projecting above the test mudline.

The loading sequence comprised both cyclic and static lateral load tests in a free head condition. Deflection and inclination of the pile at the mudline were recorded, together with bending strains. The flexural stiffness of the pile was determined to be 5.867×10^{10} lb·in² (1.69×10^5 kN·m²).

The variation of pressuremeter limit pressures with depth from an investigation conducted in May 1982 is presented in Fig. 18.

3.2 Briaud-Smith-Meyer Method

The pile was driven, thus the reload cycle of the pressuremeter test is used for both the Q-y and F-y curves. The results of the pressuremeter test at a depth of 4 ft (1.22 m) using a pavement

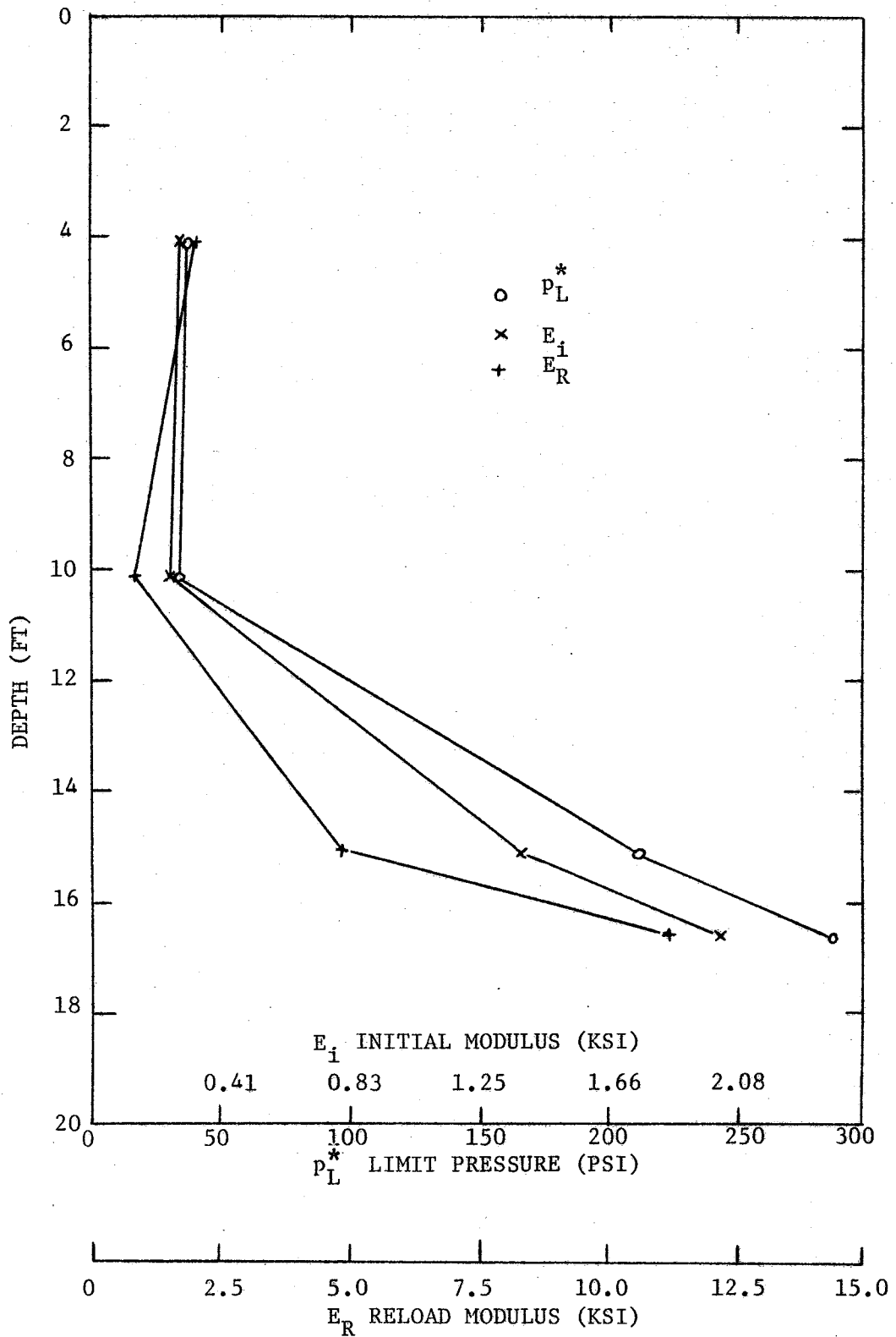


FIG. 18 - Pressuremeter Test Results for Mustang Island Site

pressuremeter (3) are shown in Fig. 19.

3.2.1 Translation of the axis

From a graphical construction (Fig. 19) p_{OH} is obtained as 24 kPa. The reload cycle of the pressuremeter test is shown on Fig. 19 with the axis translated to p_{OH} . An increase in volume of 65 cm^3 was needed to reach p_{OH} on the reload cycle. A table of pressure and volume values for the indicated points is given below.

Point	ΔV cm^3	ΔV_1 cm^3	$\frac{\Delta V_1}{V_1}$	$P - P_{OH}$ lb/in.^2
0	65	0	0	0
1	70.5	5.5	0.0208	15.5
2	75	10.0	0.0378	21.9
3	79.4	14.4	0.0544	24.1
4	84.4	19.4	0.0733	24.7

3.2.2 Check pressuremeter critical depth

The radius of the pressuremeter used is 0.69 in. This yields a critical depth of

$$z_c = 60 \times 0.69 \text{ in.}$$

$$= 41.4 \text{ in.}$$

Since the test depth was 48 in. there is no depth effect to consider on this test ($\chi = 1$).

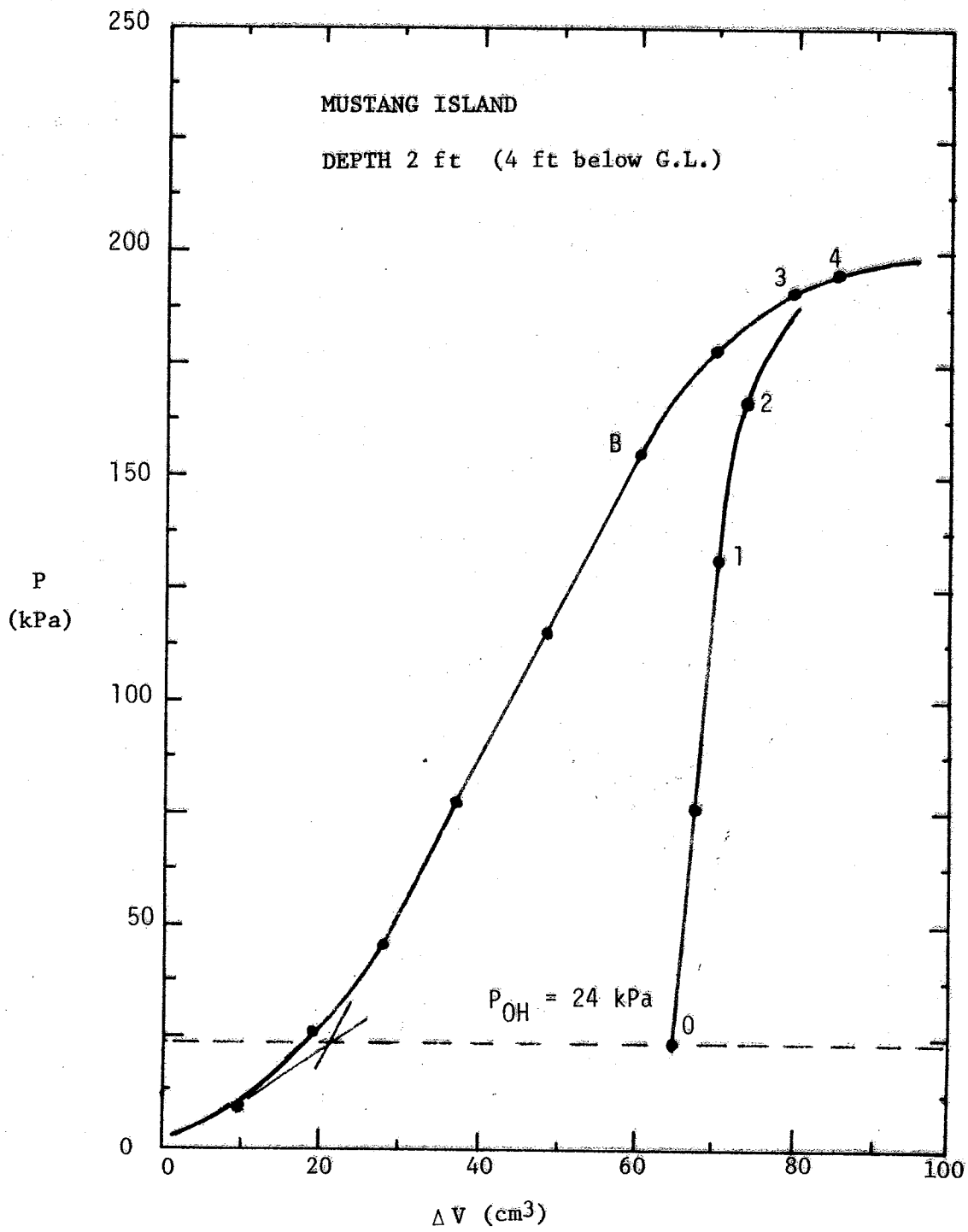


FIG. 19- Mustang Island Site: 4 ft Pressuremeter Test

3.2.3 Front resistance, Q

The front resistance is found by

$$Q = p \times SQ \times B$$

The pile is circular so SQ is 0.8. For point 1 on the reload cycle this gives

$$\begin{aligned} Q &= 15.5 \text{ lb/in.}^2 \times 0.8 \times 24 \text{ in.} \\ &= 297.6 \text{ lb/in.} \end{aligned}$$

$$\text{Point 2: } Q = 21.9 \times 0.8 \times 24 = 420.5 \text{ lb/in.}$$

$$\text{Point 3: } Q = 24.1 \times 0.8 \times 24 = 462.7 \text{ lb/in.}$$

$$\text{Point 4: } Q = 24.7 \times 0.8 \times 24 = 474.2 \text{ lb/in.}$$

3.2.4 Accounting for the critical depth for the pile

The relative pile to soil stiffness is given by

$$\begin{aligned} RR &= \frac{1}{B} \sqrt[4]{\frac{EI}{pL}} \\ &= \frac{1}{24 \text{ in.}} \sqrt[4]{\frac{5.867 \times 10^{10} \text{ lb}\cdot\text{in.}^2}{35 \frac{\text{lb}}{\text{in.}^2}}} \\ &= 8.43 \end{aligned}$$

The critical depth is then found as

$$\begin{aligned}
 z_c &= \frac{\pi B}{4} \text{ (RR-5)} \\
 &= \frac{\pi \times 2 \text{ ft}}{4} \text{ (8.43-5)} \\
 &= 5.4 \text{ ft}
 \end{aligned}$$

The pressuremeter test considered is at a depth of 4 ft so

$$\frac{z}{z_c} = \frac{4}{5.4} = 0.74$$

From Fig. 11, the reduction factor ψ is 0.90. Thus the corrected front resistance at point 1 is

$$\begin{aligned}
 Q &= 297.6 \times 0.90 \\
 &= 267.8 \text{ lb/in.}
 \end{aligned}$$

$$\text{Point 2: } Q = 420.5 \times 0.90 = 378.4 \text{ lb/in.}$$

$$\text{Point 3: } Q = 462.7 \times 0.90 = 416.4 \text{ lb/in.}$$

$$\text{Point 4: } Q = 474.2 \times 0.90 = 426.8 \text{ lb/in.}$$

3.2.5 Pile displacement, y

The displacement of the pile at point 1 is found by

$$\begin{aligned}
 y &= \frac{1}{2} \frac{\Delta V_1}{V_1} R_{\text{pile}} \\
 &= \frac{1}{2} 0.0208 \times 12 \text{ in.} \\
 &= 0.125 \text{ in.}
 \end{aligned}$$

Point 2: $y = 1/2 \times 0.0378 \times 12 = 0.227 \text{ in.}$

Point 3: $y = 1/2 \times 0.0544 \times 12 = 0.326 \text{ in.}$

Point 4: $y = 1/2 \times 0.0733 \times 12 = 0.440 \text{ in.}$

3.2.6 Lateral friction, F

The shear stress at point 1 is calculated as

$$\begin{aligned}\tau &= \frac{V_1}{V_1} \left(1 + \frac{V_1}{V_1}\right) \frac{\Delta P}{\Delta \left(\frac{\Delta V_1}{V_1}\right)} \\ &= 0.0208 (1 + 0.0208) \frac{21.9-0}{0.0378-0} \\ &= 12.3 \frac{\text{lb}}{\text{in.}^2}\end{aligned}$$

The friction is then found by

$$F = \tau \times SF \times B$$

For a circular pile $SF = 1.0$

Thus the friction at point 1 is

$$\begin{aligned}F &= 12.3 \frac{\text{lb}}{\text{in.}^2} \times 1.0 \times 24 \text{ in.} \\ &= 295.2 \frac{\text{lb}}{\text{in.}}\end{aligned}$$

$$\text{Point 2: } F = \left[0.0378 (1 + 0.0378) \frac{24.1 - 15.5}{0.0544 - 0.0208} \right] \times 1 \times 24$$

$$= 241.0 \frac{\text{lb}}{\text{in.}}$$

$$\text{Point 3: } F = \left[0.0544 (1 + 0.0544) \frac{24.7 - 21.9}{0.0733 - 0.0378} \right] \times 1 \times 24$$

$$= 108.6 \frac{\text{lb}}{\text{in.}}$$

$$\text{Point 4: } F = \left[0.0733 (1 + 0.0733) \frac{24.7 - 24.1}{0.0733 - 0.0544} \right] \times 1 \times 24$$

$$= 59.9 \frac{\text{lb}}{\text{in.}}$$

3.2.7 Total resistance, P

The total resistance is the sum of the front and the friction resistance. Thus for point 1:

$$\begin{aligned} P &= Q + F \\ &= 267.8 \frac{\text{lb}}{\text{in.}} + 295.2 \frac{\text{lb}}{\text{in.}} \\ &= 563.0 \frac{\text{lb}}{\text{in.}} \end{aligned}$$

$$\text{Point 2: } P = 378.4 + 241.0 = 619.4 \text{ lb/in.}$$

$$\text{Point 3: } P = 416.4 + 108.6 = 525.0 \text{ lb/in.}$$

$$\text{Point 4: } P = 426.8 + 59.9 = 486.7 \text{ lb/in.}$$

The resulting P-y curve is shown on Fig. 20.

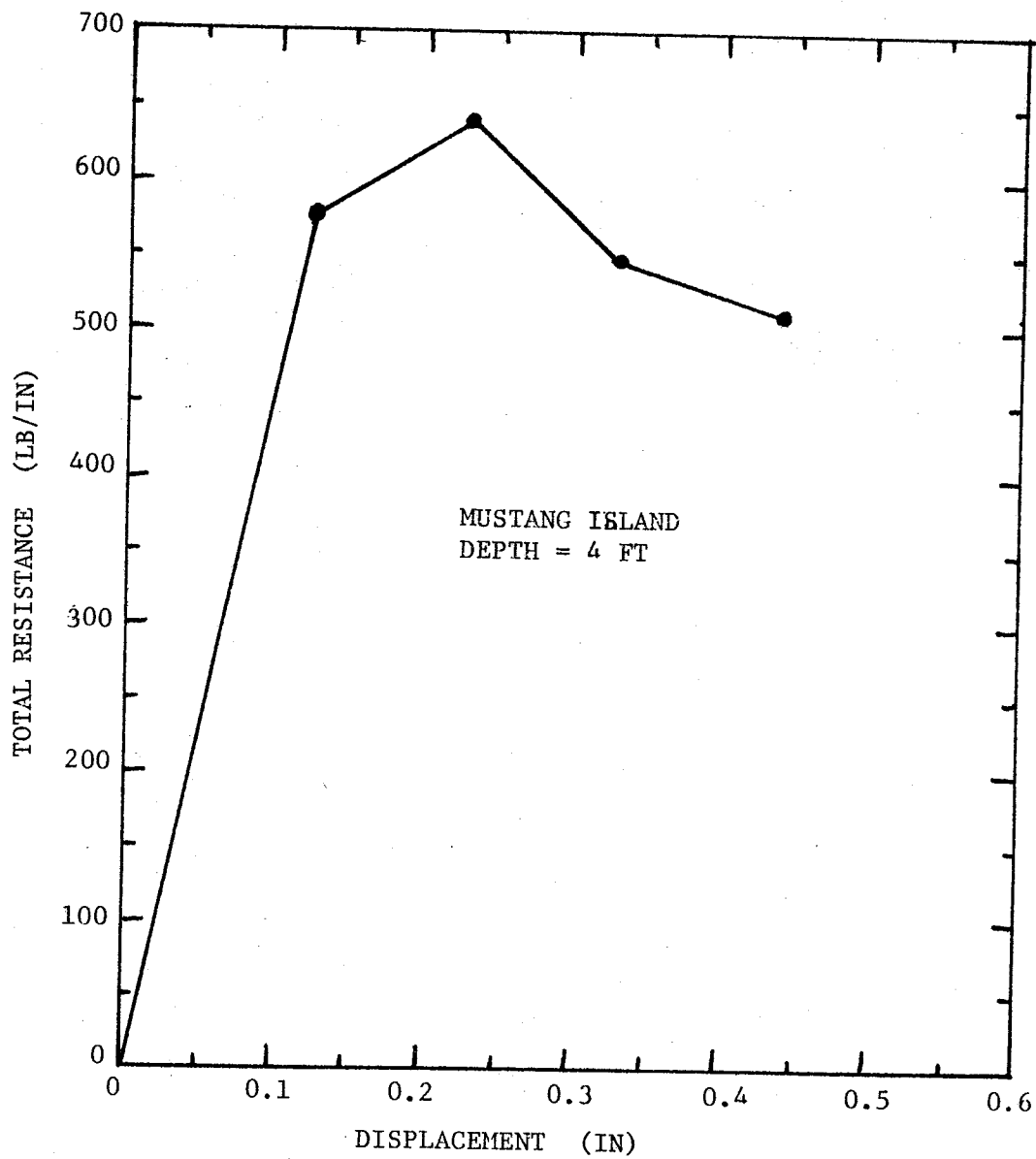


FIG. 20 - Predicted P-y Curve Using Briaud-Smith-Meyer Method

3.2.8 Obtaining the top load-top movement curve

The P-y curves obtained at the depth of each pressuremeter test are input into a finite difference beam-column computer program which will calculate the pile deflection under a given loading condition. The predicted ground line deflection versus the lateral applied load for the Mustang Island pile is shown with the measured results on Fig. 21. The predicted maximum bending moment versus the lateral applied load is shown with the measured results on Fig. 22.

3.3 Briaud-Smith-Meyer Simplified Method: Subgrade Modulus Approach

3.3.1 Calculating the modulus of subgrade reaction

From section 2.2.1 and for the 4 ft pressuremeter test (Fig. 19)

$$k = \frac{p_A^*}{R_{\text{pile}}} \times \frac{2}{\chi} \frac{(\psi \text{SQ} + \text{SF})(V_o + \Delta V_{\text{OR}})}{(\Delta V_B - \Delta V_{\text{OR}})}$$

$$p_A^* = 154 - 24 = 130 \text{ kPa} = 18.85 \text{ lb/in.}^2$$

$$R_{\text{pile}} = 12 \text{ in.}$$

$$\chi = 1.0$$

$$\psi = 0.90$$

$$\text{SQ} = 0.8$$

$$\text{SF} = 1.0$$

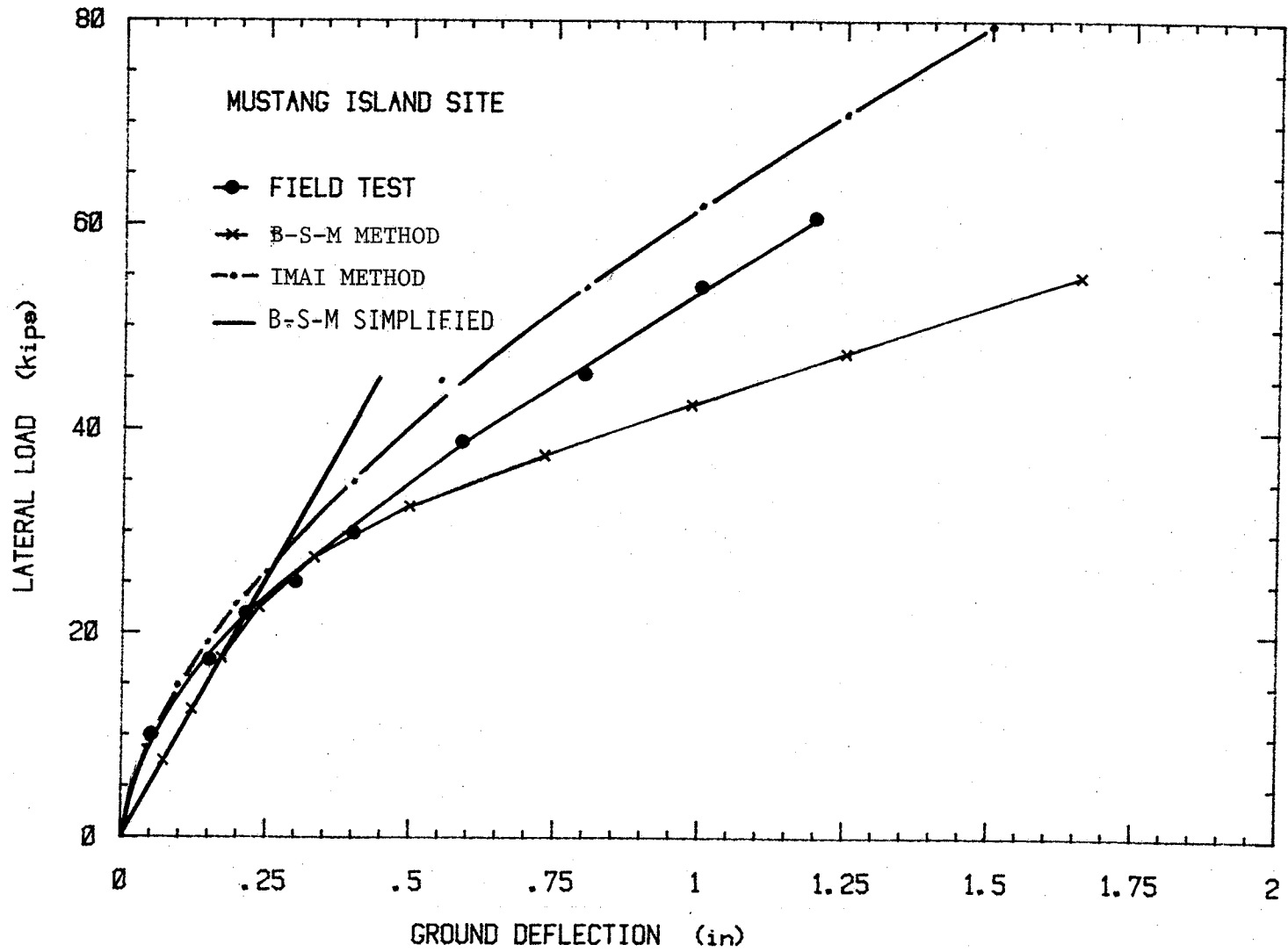


FIG. 21- Mustang Island Site : Comparison of Groundline Deflection

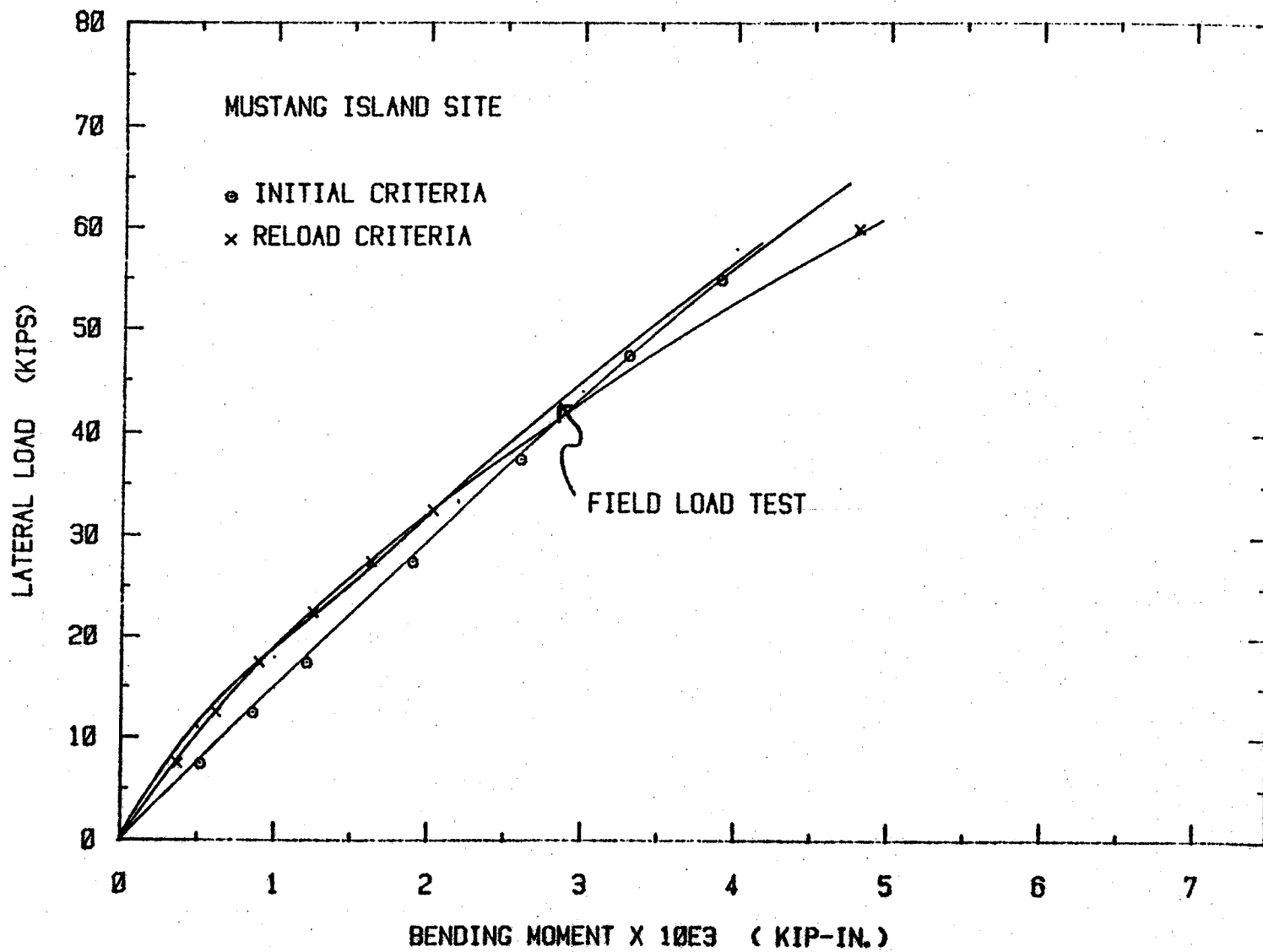


FIG. 22 - Mustang Island Site: Comparison of Maximum Bending Moment

$$V_o = 200 \text{ cm}^3$$

$$\Delta V_{OR} = 65 \text{ cm}^3$$

$$\Delta V_B = 71.5 \text{ cm}^3$$

$$k = \frac{18.85}{12} \times \frac{2}{1} \left(\frac{(0.90 \times 0.8 + 1)(200 + 65)}{71.5 - 65} \right) = 220.3 \text{ lb/in}^3$$

A profile of k with depth is shown on Fig. 23.

3.3.2 Transfer length, l_o

The average k value over 5 pile diameters is 126/lb/in.³. Using this k value the transfer length is

$$l_o = \sqrt{\frac{2 \times 5.867 \times 10^{10}}{277 \times 12}}$$

$$= 93.86 \text{ in.} = 7.82 \text{ ft}$$

Therefore 3 times l_o is 23.5 ft. Calculate l_o again using an average k at the midpoint between 10 ft and 23.5 ft, at 16.8 ft.

$$\text{at 16.8 ft: } l_o = \sqrt{\frac{2 \times 5.867 \times 10^{10}}{277 \times 12}}$$

$$= 77 \text{ in.} = 6.42 \text{ ft}$$

$$3 \times l_o = 19.3 \text{ ft}$$

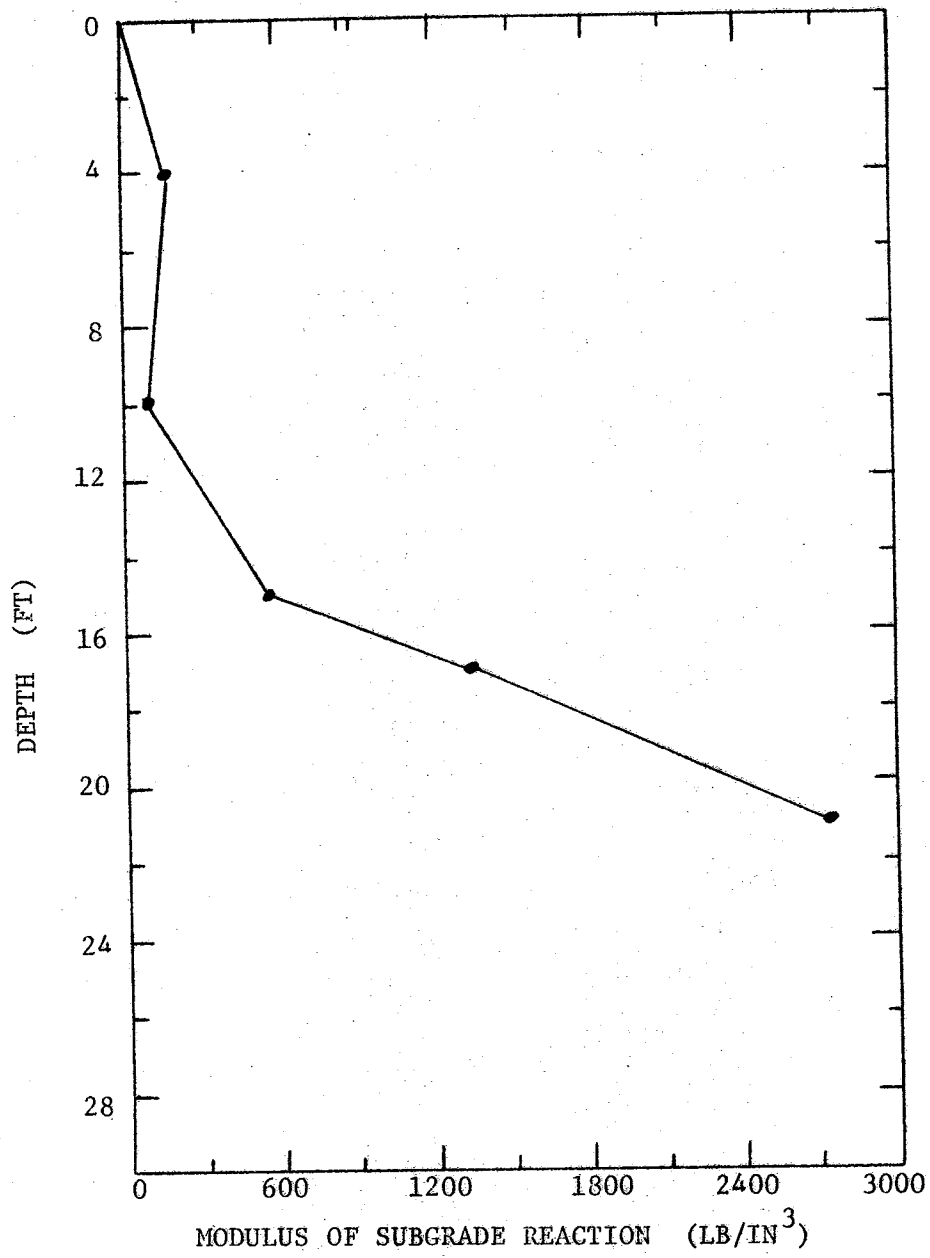


FIG. 23 - Modulus of Subgrade Reaction from Pressuremeter Tests at Mustang Island Site

Calculate ℓ_o at midpoint again.

$$\begin{aligned} \text{at 18 ft: } \ell_o &= \frac{2 \times 5.867 \times 10^{10}}{370 \times 12} \\ &= 71.7 \text{ in} = 5.97 \text{ ft} \end{aligned}$$

$$3 \times \ell_o = 17.9 \text{ ft.}$$

This is approximately the same depth that k was averaged over. Thus,

$$k = 370 \text{ lb/in.}^3, \text{ and } \ell_o = 71.7 \text{ in.}$$

Since

$$h = 69 \text{ ft} < 3\ell_o$$

the pile is flexible.

The minimum of k within 5 pile diameters and k within $3\ell_o$ is 126 lb/in.³ and is used for further calculations.

3.3.3 Calculating the deflection at the ground surface

At the ground surface $z = 0$, the solution simplifies to

$$y = \frac{2}{\ell_o kb} \left[T_o + \frac{M_o}{\ell_o} \right]$$

For the Mustang Island pile there was no moment at the ground surface so the deflection in inches is given by

$$y = \frac{2T_o}{\ell_o kb} = \frac{2T_o}{71.7 \times 126 \times 24} = \frac{T_o (\text{lbs})}{108410}$$

This is the equation of the straight line shown on Fig. 21.

3.3.4 Calculating z_{\max}

Taking top horizontal load of 20 kips and top moment of 0, the depth to the maximum bending moment, z_{\max} is found by

$$\begin{aligned} 0 &= T_0 e^{-z/l_0} \left(\cos \frac{z}{l_0} - \sin \frac{z}{l_0} \right) \\ &= 20 e^{-z/1.7} \left(\cos \frac{z}{1.7} - \sin \frac{z}{1.7} \right) \end{aligned}$$

$$z_{\max} = 56.3 \text{ in.} = 4.69 \text{ ft.}$$

3.3.5 Calculating M_{\max}

The corresponding maximum bending moment, M_{\max} , is

$$\begin{aligned} M_{\max} &= 20 \times 4.69 \times e^{-\frac{56.3}{71.7}} \sin \frac{56.3}{71.7} \\ &= 145.4 \text{ k}\cdot\text{ft} \end{aligned}$$

3.4 Imai's method

3.4.1 Calculating soil stiffness value k_m

Using the initial cycle of the pressuremeter tests calculate the probe radius at two points on the linear portion of the pressuremeter curve. Using points A and B from Fig. 17

$$\text{Point A: } r_A = \sqrt{\frac{199.79 + 21}{\pi \times 22.8}} = 1.7557 \text{ cm}$$

$$\text{point B: } r_B = \sqrt{\frac{199.79 + 59.5}{\pi \times 22.8}} = 1.9026 \text{ cm}$$

Then

$$k_m = \frac{1.585 - 0.2446}{1.9026 - 1.7557} = 0.125 \text{ kg/cm}^3$$

3.4.2 Calculating specific stiffness value k_o

First calculate the radius at the midpoint between points A and B.

$$r_m = \frac{1.7557 + 1.9026}{2} = 1.8292 \text{ cm}$$

Then

$$k_o = \frac{\pi}{2} \sqrt[4]{2 (1.7557)(1.8292 - 1.7557)^2} \times 9.125$$

$$= 5.32$$

In a similar manner the k_o values of the other pressuremeter test depths are calculated. The k_o values with the corresponding layer thicknesses are shown in the table below.

k_o	Layer Thickness (cm)
5.32	213.4
3.16	167.6
17.78	99.1
30.32	91.4
30.86	114.3
8.67	228.6

3.4.3 Calculating \bar{k}_o

The equivalent k_o value, \bar{k}_o , is the average k_o value over a depth 1.5 times the depth to a bending moment of zero. This gives

$$\bar{k}_o = \frac{(5.32 \times 23.4) + (3.16 \times 167.6) + (17.78 \times 99.1)(30.32 \times 91.4) +$$

$$\frac{(30.86 \times 114.3) + (8.69 \times 228.6)}{914.4}$$

$$= 12.81$$

3.4.4 Pile parameters

The pile parameters are given as

$$EI = 1.723 \times 10^8 \text{ kg.cm}^2$$

$$B = 60.96 \text{ cm}$$

$$h = 0 \text{ cm}$$

$$f = 0$$

3.4.5 Calculating basic stiffness value K_o

$$K_o = \frac{\bar{k}_o}{\sqrt[4]{B}} = \frac{12.81}{\sqrt[4]{60.96}}$$

$$= 4.584$$

3.4.6 Calculating the horizontal load

The design stiffness value K is found by

$$K = \frac{K_o}{\sqrt{y}}$$

For a deflection, y, of 0.127 cm

$$K = \frac{4.584}{\sqrt{0.127}} = 12.86$$

From this value β is determined as

$$\beta = \sqrt[4]{\frac{KB}{EI}}$$

$$= \sqrt[4]{\frac{12.86 \times 60.96}{1.723 \times 10^8}}$$

$$= 4.6185 \times 10^{-2}$$

The horizontal load, H, is then found as

$$H = \frac{12 EI \beta^3}{[(4-3f)(1+\beta h)^3 + 2]} y$$
$$= \frac{(12)(1.723 \times 10^8)(4.6185 \times 10^{-2})^3}{[(4-3(0))(1+4.6185 \times 10^{-2})(0) + 2]} \times 0.127$$
$$= 4312 \text{ kg}$$

= 9.5 kips at deflection of 0.05 in.

The table below summarizes the calculations for other deflections.

y cm (in.)	K_0	β x 10	H kg (kips)
0.127 (0.05)	12.86	0.04619	4312 (9.5)
0.254 (0.10)	9.10	0.04235	6650 (14.7)
0.381 (0.15)	7.43	0.04026	8568 (18.9)
0.508 (0.20)	6.43	0.03884	10256 (22.6)
0.635 (0.25)	5.75	0.03777	11791 (26.0)
1.016 (0.40)	4.55	0.02562	15817 (34.9)
1.524 (0.60)	3.71	0.03386	20379 (44.9)
2.032 (0.80)	3.22	0.03266	24393 (53.8)
2.540 (1.00)	2.88	0.03176	28044 (61.8)
3.810 (1.50)	2.35	0.03019	36132 (79.7)

These results are plotted versus the measured deflections on Fig. 21.

CHAPTER 4. CASE HISTORIES

4.1 Houston Site

Reported by Reese and Welch (18) to develop criteria for stiff clay above the water table, the Houston Site was located at the intersection of State Highway 225 and Old South Loop East.

The soil consisted of 28.0 ft (8.5 m) of stiff to very stiff red clay, known locally as Beaumont clay, underlain by 2.0 ft (0.6 m) of interspersed silt and clay layers and very stiff tan silty clay to a depth of 42 ft (13 m). Undrained shear strength is reported as 2,000 lb/ft² (100 kPa) and the water table was located at a depth of 18.0 ft (5.5 m).

The pile consisted of a drilled reinforced concrete shaft, 30 in. (760 mm) in diameter, augered to a depth of 42 ft (13 m) and extended 2.0 ft (0.6 m) above the ground surface. The shaft was instrumented to measure bending strains with gages spaced at 15 in. (380 mm) intervals for the top two-thirds of the shaft and at 30 in. (760 mm) intervals for the bottom one-third.

The loading test consisted of applying a lateral load at the ground surface in a free head condition, and measuring top slope, top deflection and bending strains along the length of the shaft. Flexural stiffness of the shaft was determined by site loading to be approximately 2.8×10^{11} lb·in² (8.09×10^5 kN·m²).

Pressuremeter tests were conducted in November 1981 and the variation of limit pressure with depth is given in Fig. 24. The measured and predicted groundline deflection versus applied lateral load are

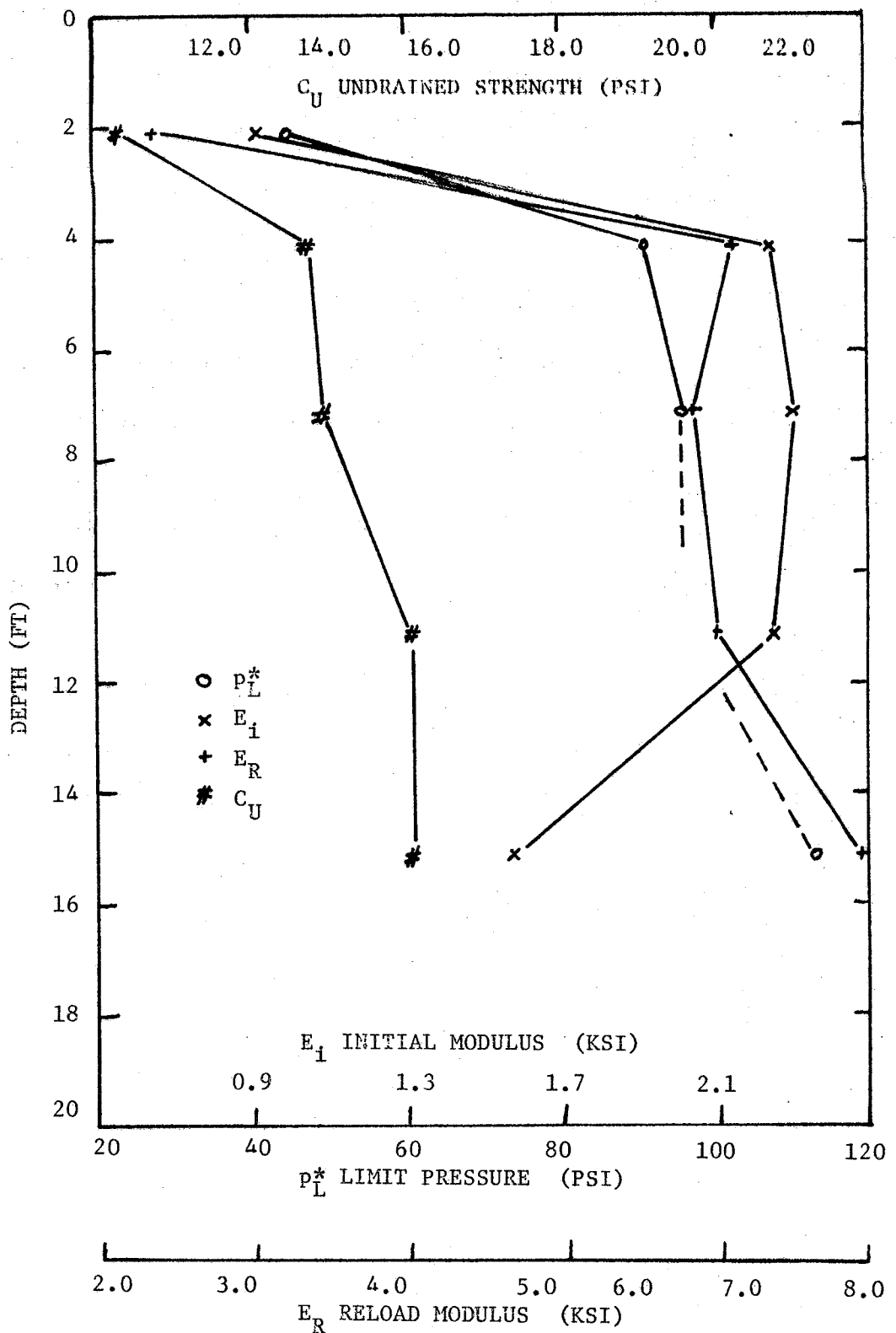


FIG. 24 - Pressuremeter Test Results and Undrained Shear Strength for the Houston Site

shown on Fig. 25. The measured and predicted maximum bending moments versus applied lateral load are shown on Fig. 26. The predictions are based on the Briaud-Smith-Meyer method.

4.2 Sabine Site

At a site near the mouth of the Sabine River a series of lateral load tests in free and fixed head conditions were performed and reported by Matlock (19).

The soil consisted of slightly overconsolidated inorganic clay of high plasticity with a single sand layer between 16.0 ft (5.0 m) and 20.0 ft (6.1 m). Thin sand partings and a few sand seams varying in thickness from 1 in. to 4 in. (25 mm to 100 mm) are scattered through the clay. Unconfined compression test shear strengths ranged from 100 lb/ft² (5 kPa) near the mudline to 500 lb/ft² (24 kPa) at a depth of 30.0 ft (9.1 m). The water table is reported at, or near, the ground surface.

The tests were performed in a pit 4.0 ft (1.2 m) deep flooded to a depth of 6 in. (150 mm). The pile was 12.75 in. (310 mm) in diameter and instrumented with 35 pairs of electric resistance strain gages to determine bending moment. The pile was driven, open ended, to an embedded depth of 36.0 ft. (10.9 m) with 6.0 ft (1.8 m) projecting above the test mudline.

The loading sequence comprised both cyclic and static lateral load tests with both free head and fixed head restraint conditions. At each load step surface deflection and slope were measured, together with continuous recording of bending strains. Flexural stiffness of the pipe pile was specified as 11.3×10^9 lb·in² (3.26×10^4 kN·m²).

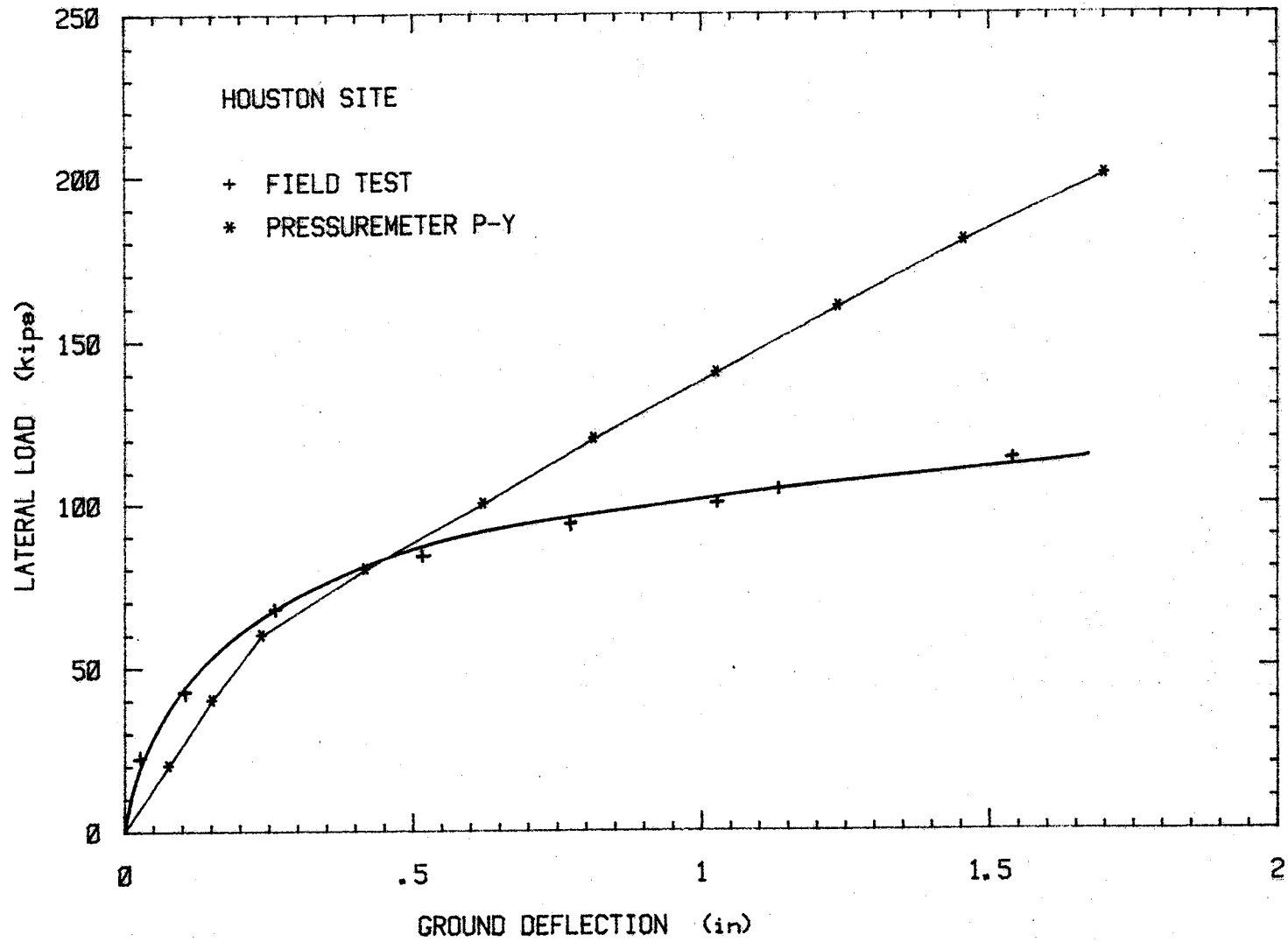


FIG. 25- Houston Site : Comparison of Groundline Deflection

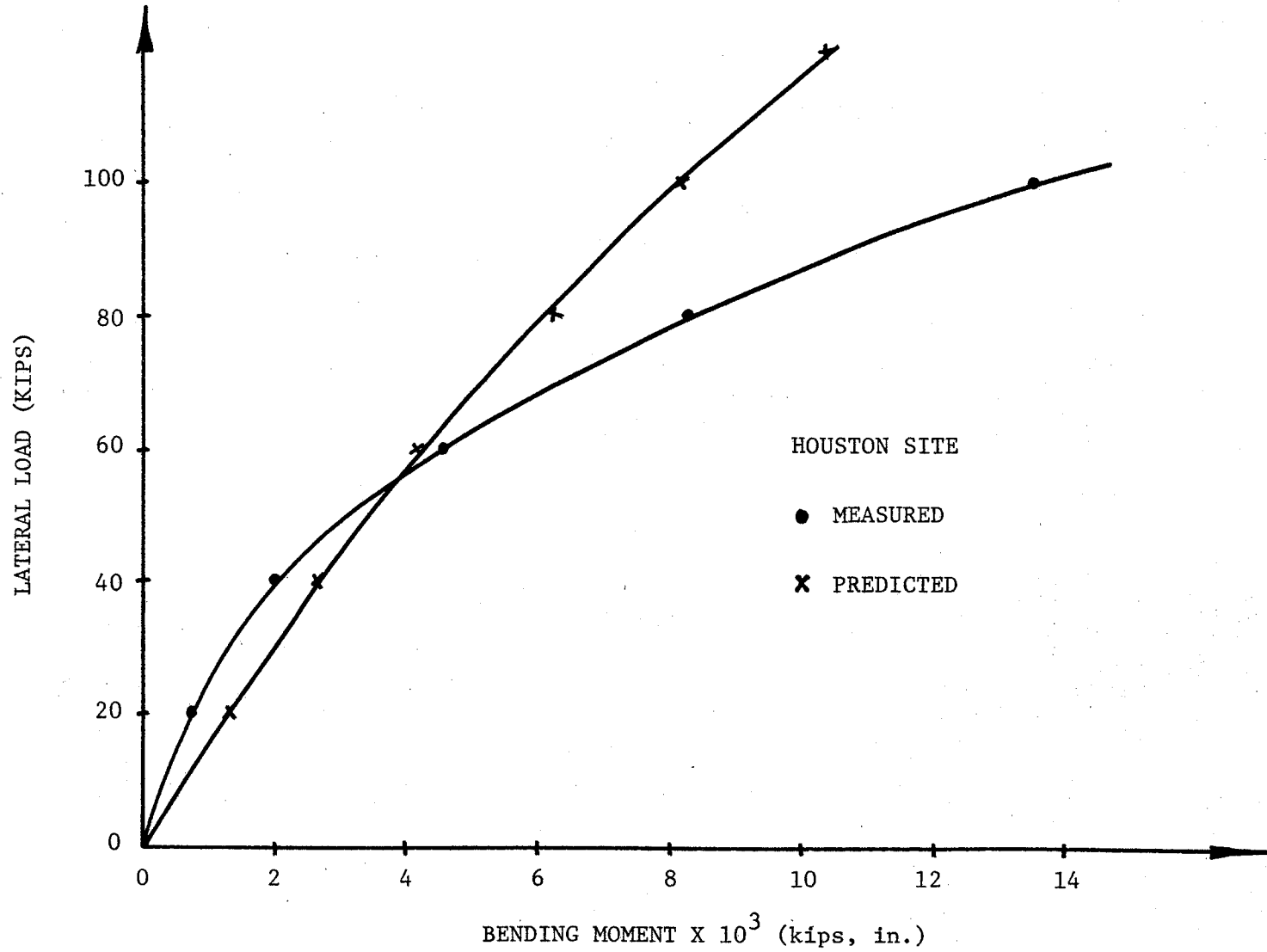


FIG. 26 - Houston Drilled Shaft. Lateral Load Versus Bending Moment Curves.

Pressuremeter tests were conducted during June 1982, and the variation of limit pressure with depth is given in Fig. 27. The measured and predicted groundline deflection versus applied lateral load is shown in Fig. 28. Figure 29 is a plot of measured and predicted maximum bending moment versus applied lateral load. The predictions are based on the Briaud-Smith-Meyer method.

4.3 Lake Austin Site

Free head lateral load tests were conducted on the shore of Lake Austin and are reported by Matlock (19).

The soil conditions consisted of inorganic clays and silts of high plasticity deposited during this century behind Lake Austin dam. The upper deposits have been subjected to desiccation during periods of prolonged drawdown leaving joints and fissures. Vane shear strengths averaged 800 lb/ft^2 (38 kPa) with little variation with depth whereas unconfined compression tests gave 500 lb/ft^2 (24 kPa). The lateral load tests were performed in a 2 ft (610 mm) deep pit, which remained flooded.

The tubular steel test pile was 12.75 in. (324 mm) in diameter and instrumented with 35 pairs of electric strain gages to determine bending moment. The pile was driven, closed end, through an 18 ft (5.5 m) deep, 8 in. (203 mm) diameter pilot hole to an embedment depth of 40.0 ft. (12.2 m) below the test mudline.

A series of free head static lateral load tests are reported with a single preliminary cyclic test. During each load step head deflection and inclination were measured together with bending strains. Flexural stiffness of the pile was determined by experiment before

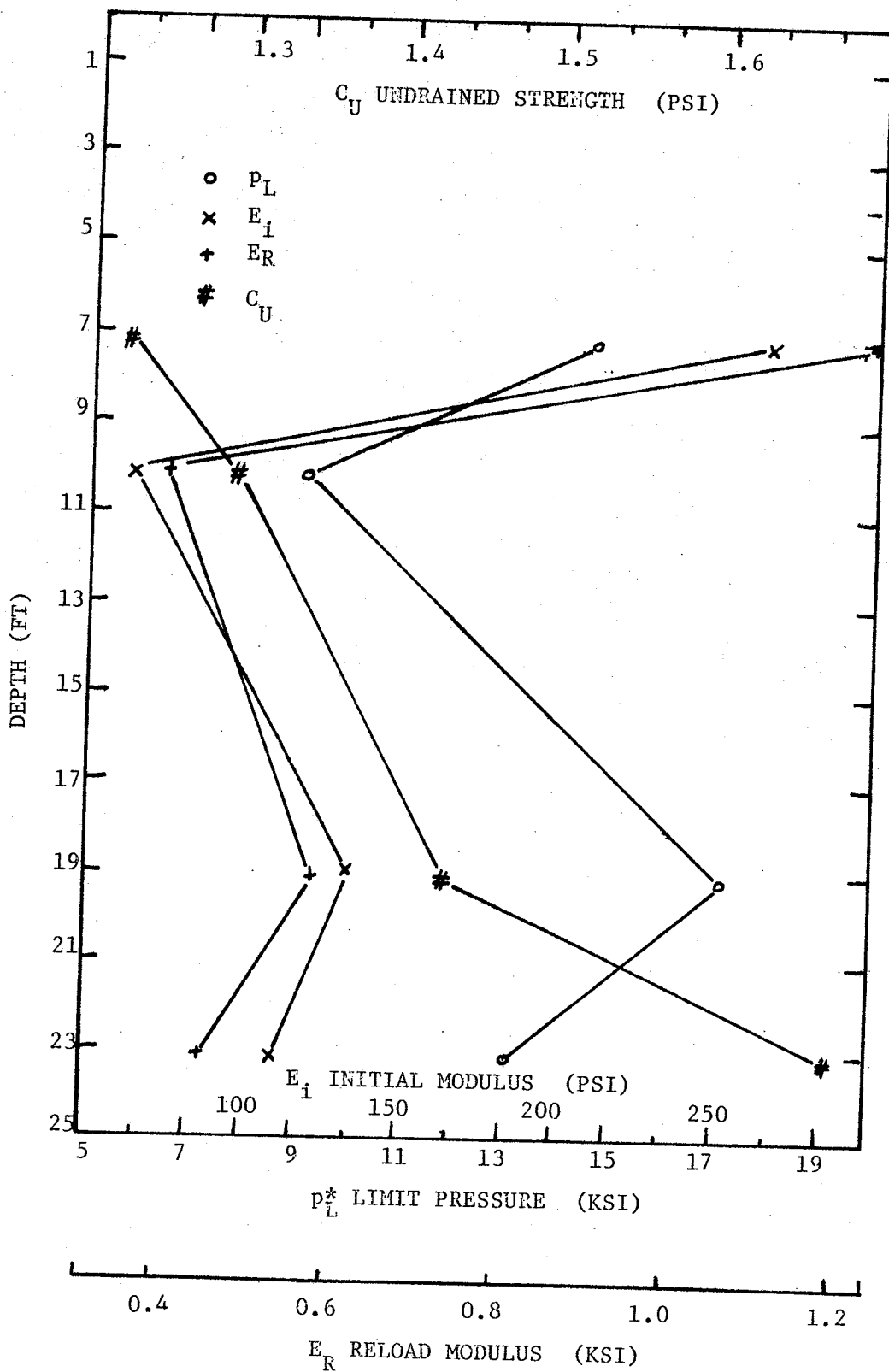


FIG. 27 - Pressuremeter Test Results and Undrained Shear Strength for the Sabine Site

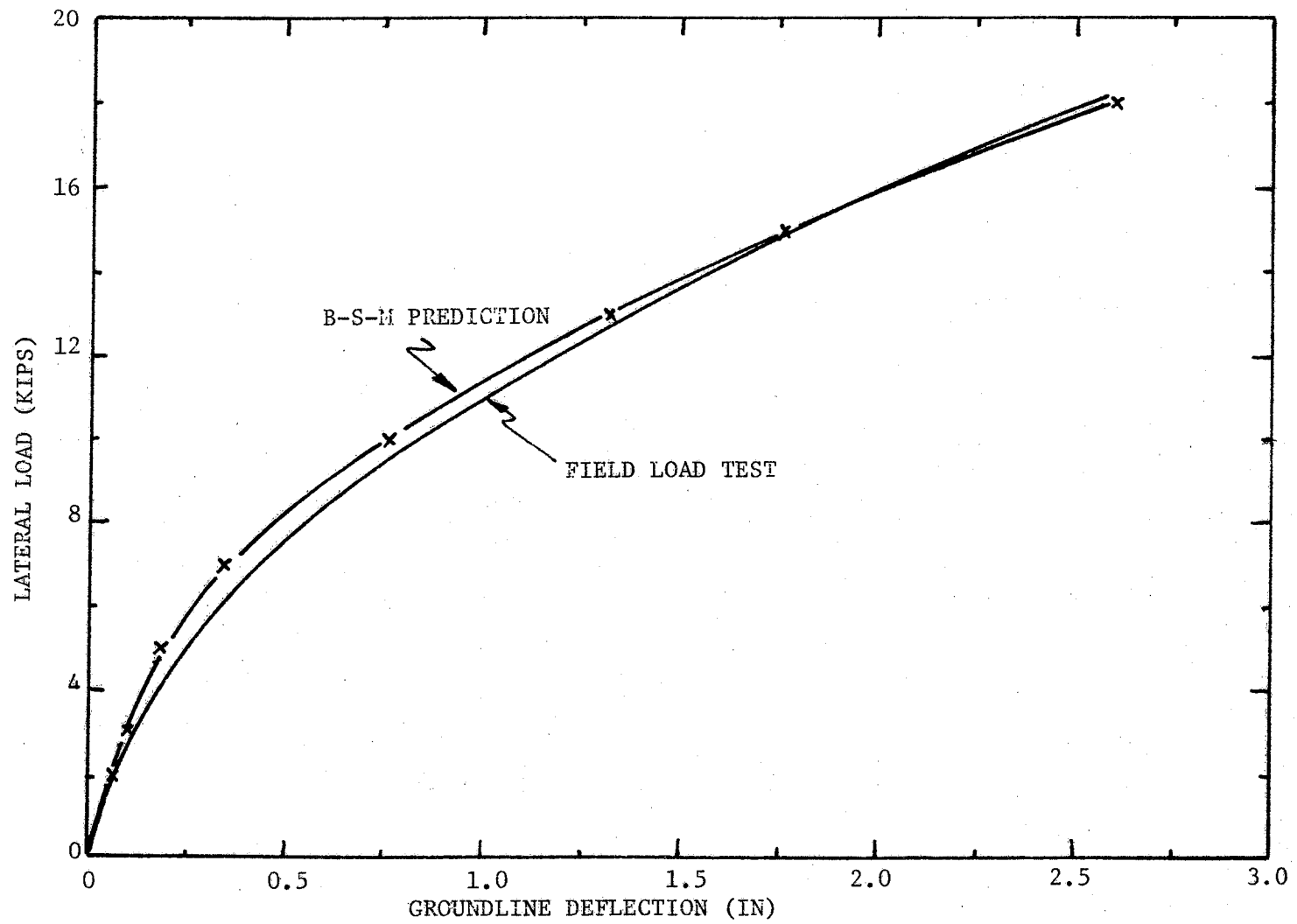


FIG. 28 - Sabine Site: Comparison of Groundline Deflection

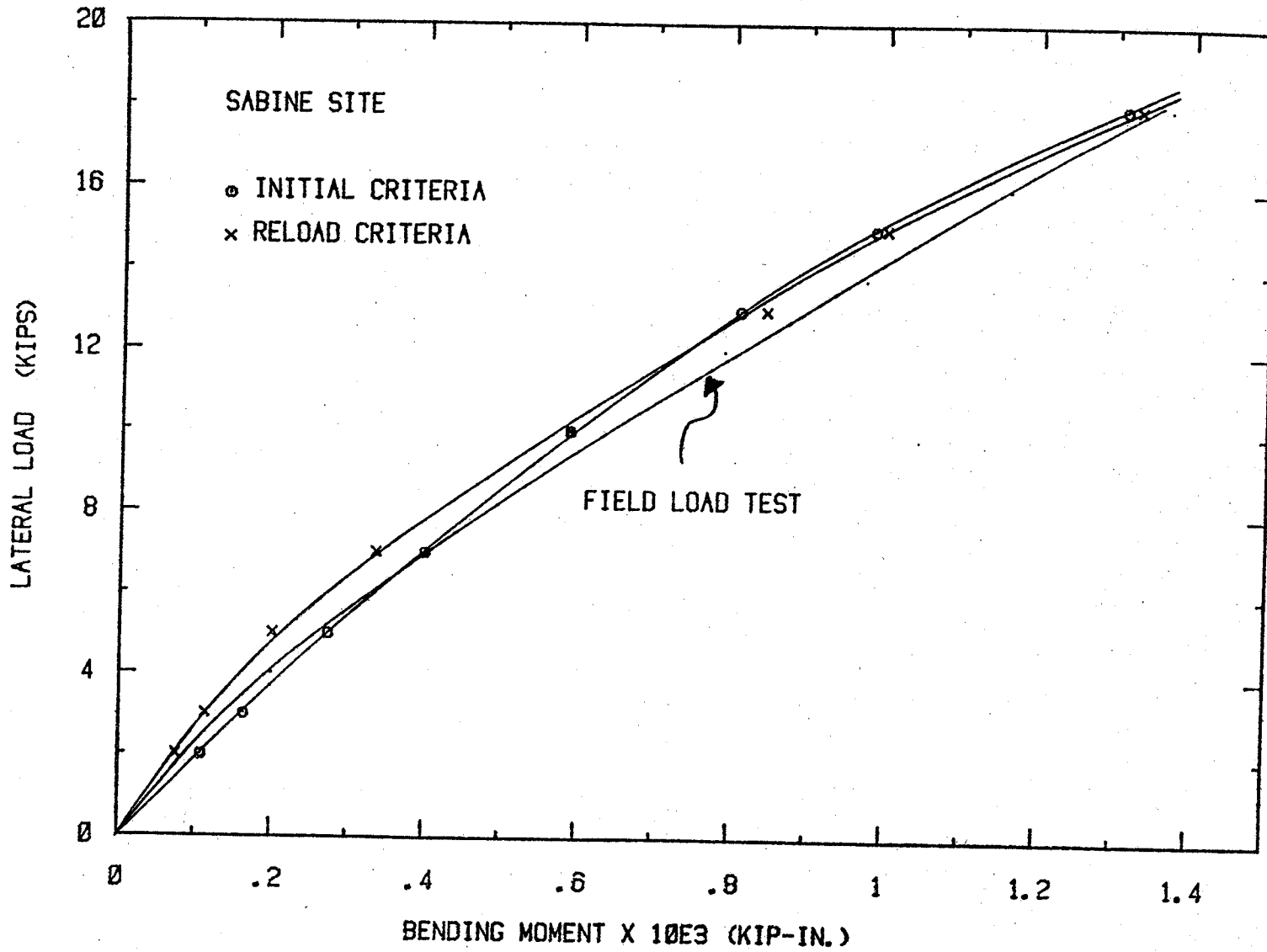


FIG. 29 - Sabine Site: Comparison of Maximum Bending Moment

installation, to be $10.9 \times 10^9 \text{ lb}\cdot\text{in}^2$ ($3.5 \times 10^4 \text{ kN}\cdot\text{m}^2$).

Pressuremeter tests were conducted during November 1981 and the variation of limit pressure with depth is given in Fig. 30. The measured and predicted groundline deflection versus applied lateral load is shown in Fig. 31. Figure 32 is a plot of the measured and predicted maximum bending moment versus applied lateral load. The predictions are based on the Briaud-Smith-Meyer method.

4.4 Manor Site

Free head lateral load tests were conducted at a location five miles to the northeast of Austin, Texas, adjacent to US Highway 290.

The soil consisted of stiff preconsolidated clays of marine origin with a slickensided secondary structure. Unconfined compressive strengths varied from approximately $4,000 \text{ lb}/\text{ft}^2$ (191 kPa) at the surface, to $8,000 \text{ lb}/\text{ft}^2$ (383 kPa) at a depth of 15.0 ft (4.5 m).

Two tubular steel test piles of different diameters were selected to study scale effects. The first test pile was 25.25 in. (640 mm) in diameter for the top 24.0 ft (7.3 m) and 24 in. (610 mm) in diameter for the remaining 25.0 ft (7.6 m). Total embedment was 49.0 ft (14.9 m) below the test mudline. The second test pile was 6.625 in. (168 mm) in diameter with a total embedment depth of 30.0 ft (9.1 m) below the test mudline. Both piles were instrumented with electric strain gages and driven, open ended, to the design penetration.

The loading sequence comprised both cyclic and static lateral load tests in a free head condition for the 25.25 in. (640 mm) pile, and free and fixed head condition for the 6.625 in. (168 mm) pile. Deflection and inclination of the surface were recorded at each load step

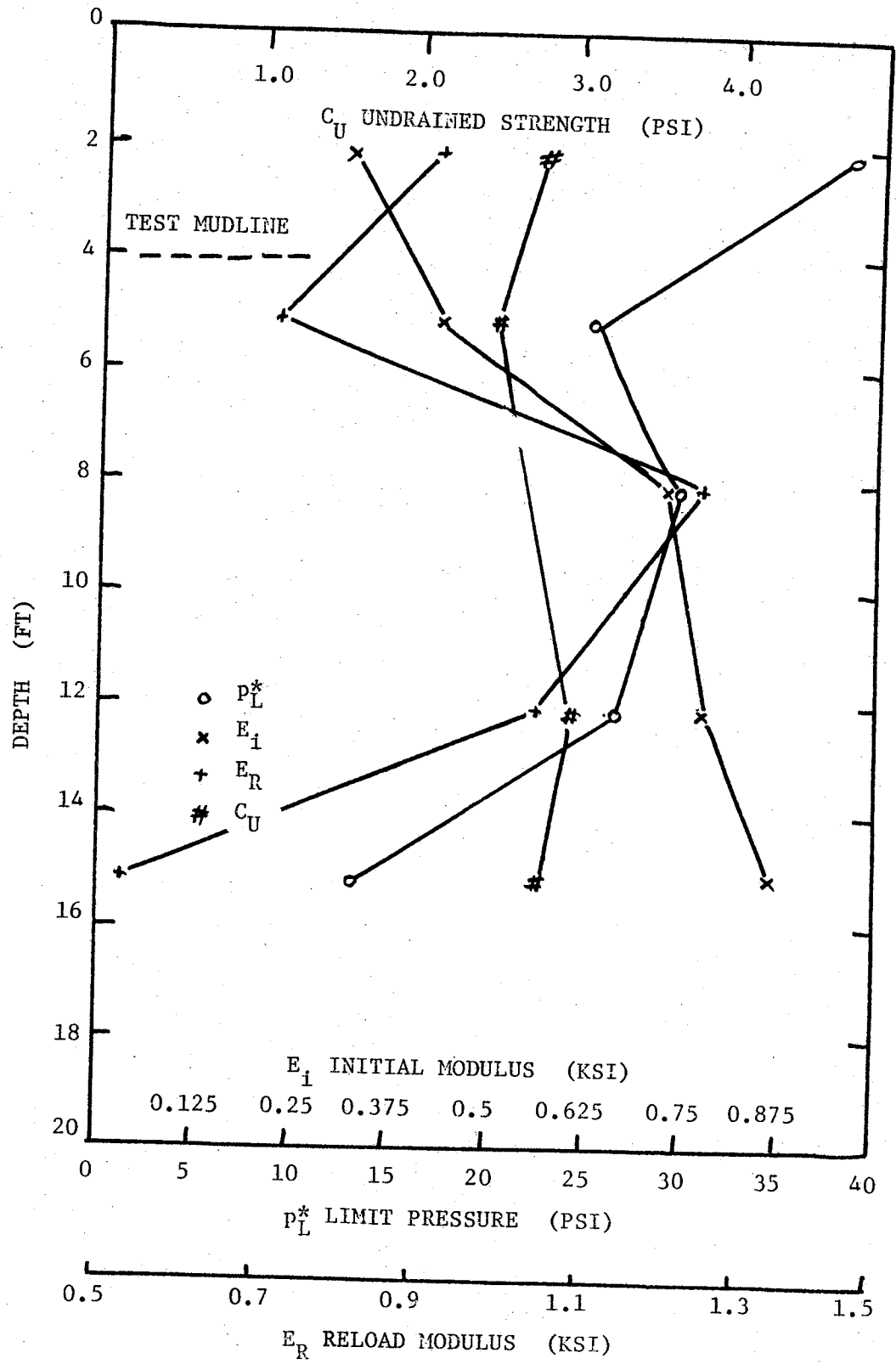


FIG. 30 - Pressuremeter Test Results and Undrained Shear Strength for Lake Austin Site

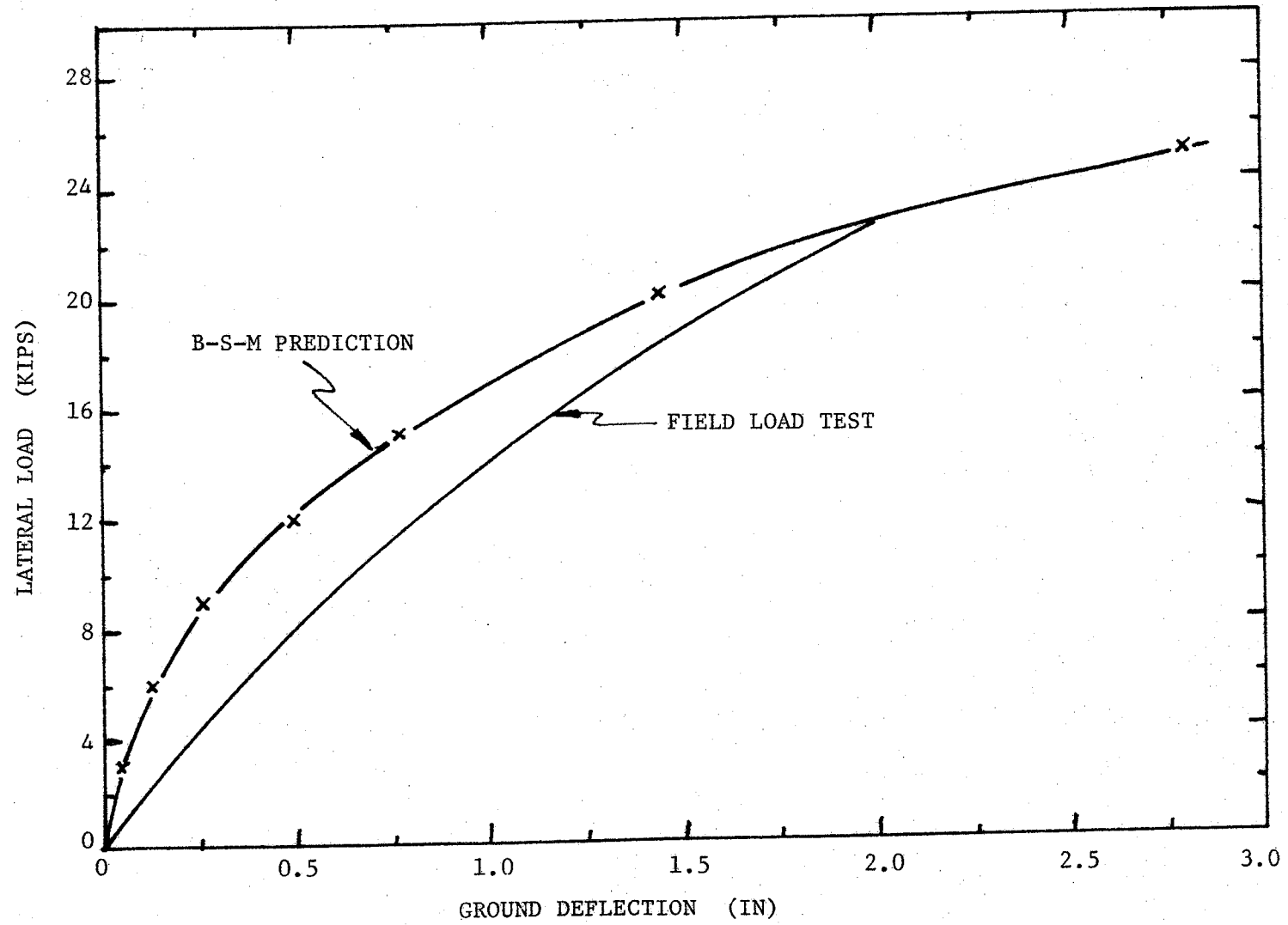


FIG. 31 - Lake Austin Site: Comparison of Groundline Deflection

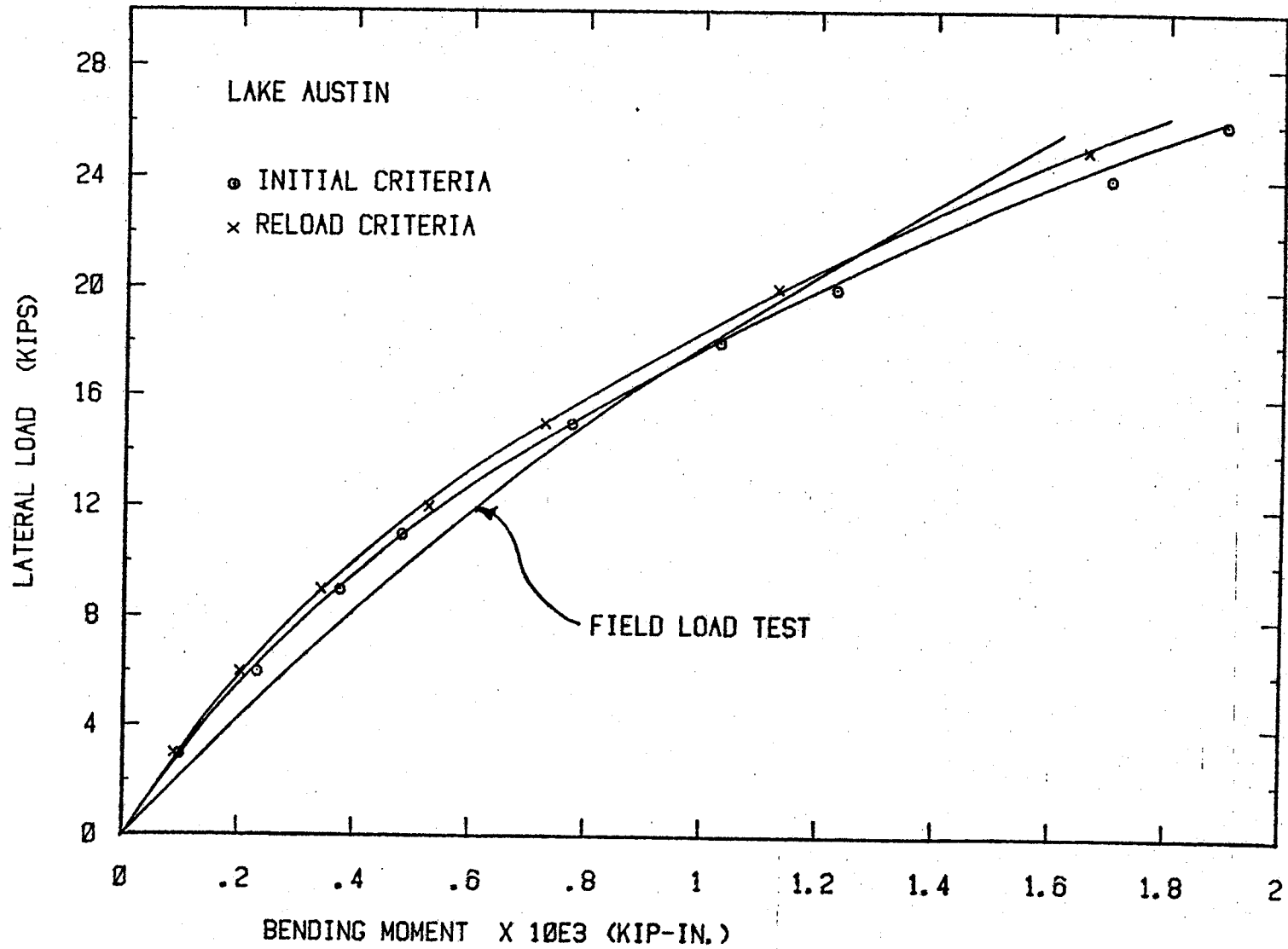


FIG. 32 - Lake Austin Site: Comparison of Maximum Bending Moment

together with measurement of bending strains. The flexural stiffness of the 25.25 in. (640 mm) pile was 1.7204×10^{11} lb·in² (4.97×10^5 kN·m²) and 5.867×10^{10} lb·in² (3.13×10^3 kN·m²) and 1.084×10^9 lb·in² (3.13×10^3 kN·m²) for the top and bottom sections respectively.

The variation of pressuremeter limit pressures with depth from an investigation conducted in November 1982 is presented in Fig. 33. The measured and predicted groundline deflection versus applied lateral load is presented in Figs. 34 and 35 for the 25.25 in. (640 mm) and 6.625 in. (168 mm) piles respectively.

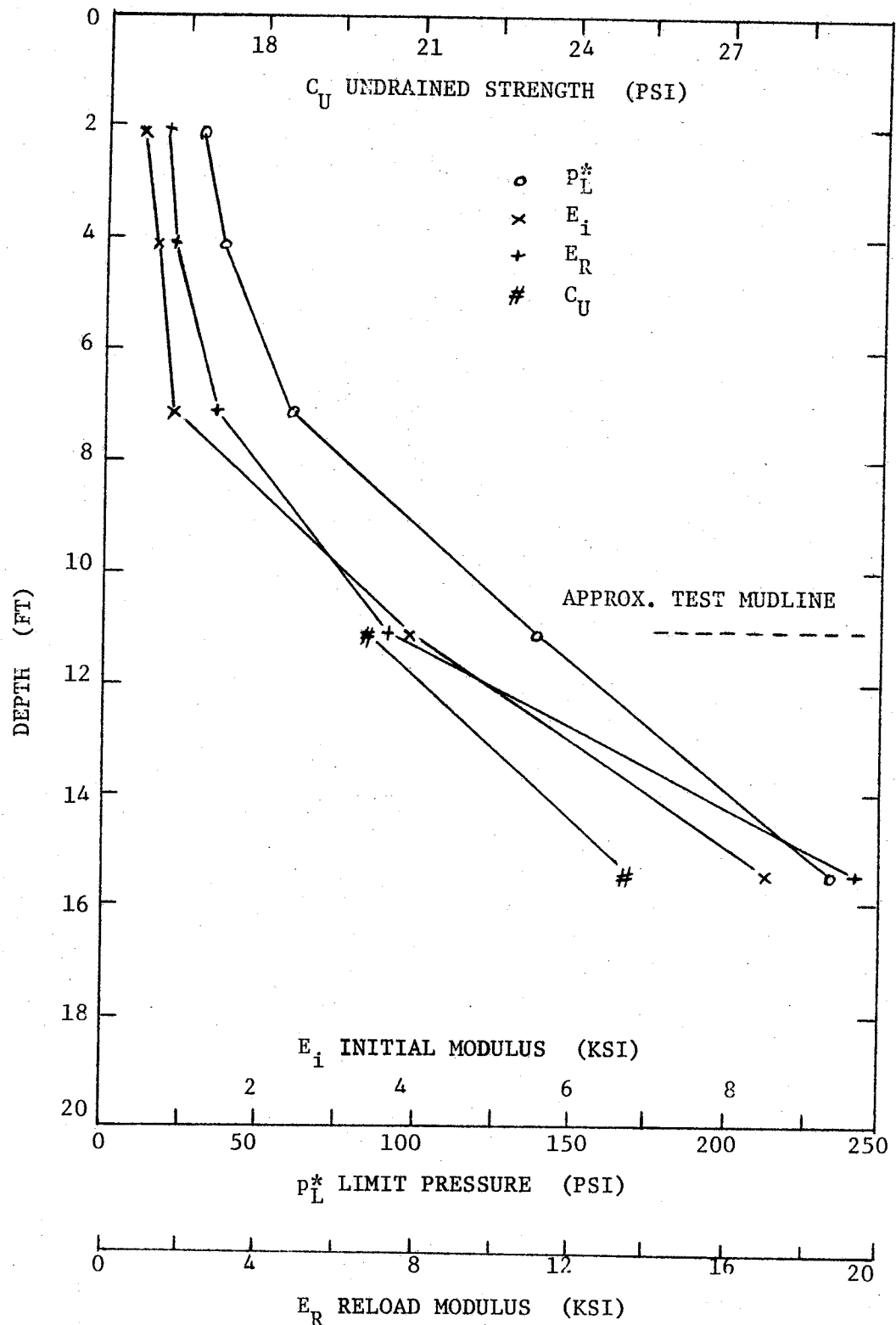


FIG. 33 - Pressuremeter Test Results and Undrained Shear Strength for Manors Site

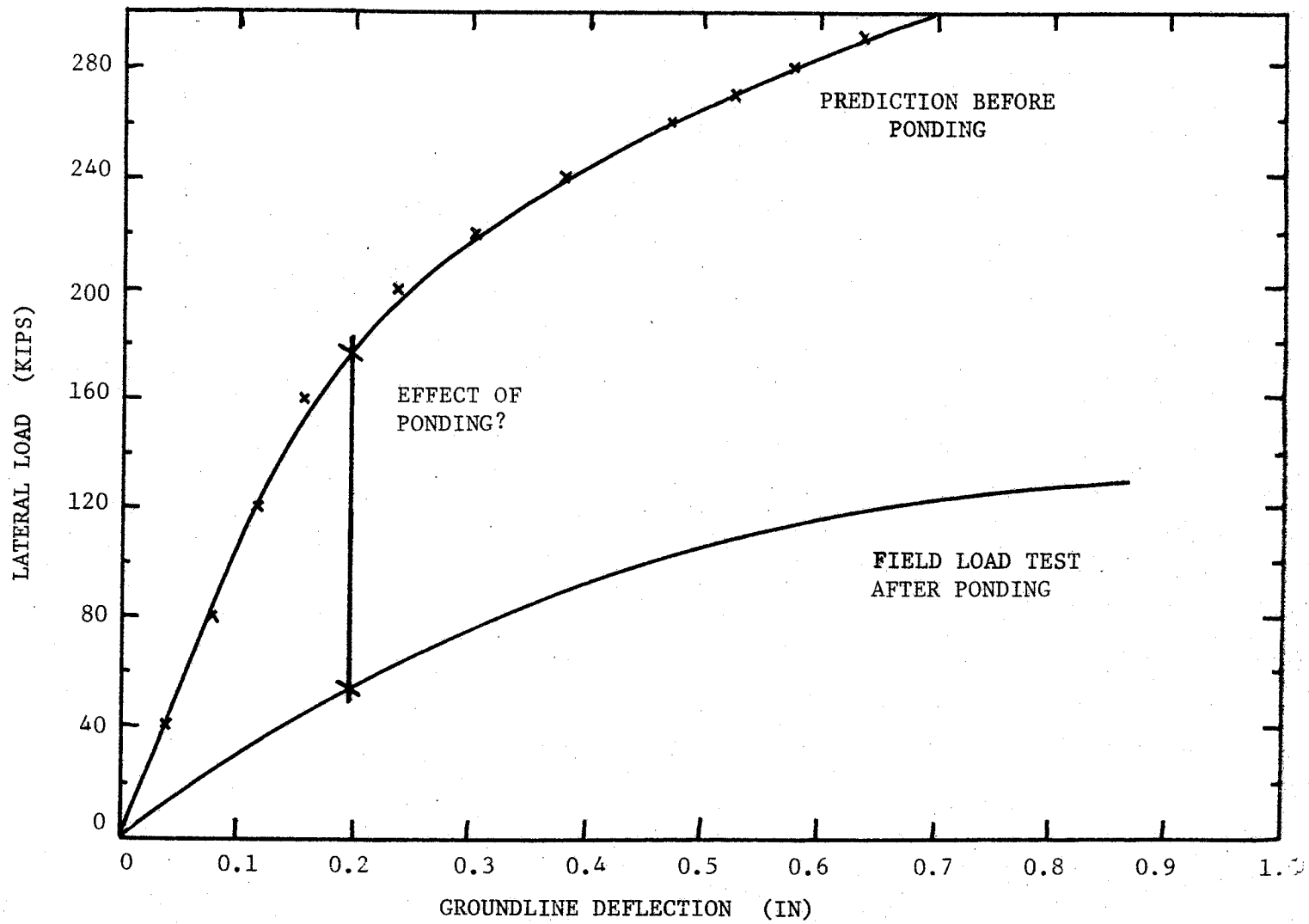


FIG. 34 - Manor Site: Comparison of Groundline Deflection for 25 in. Pile

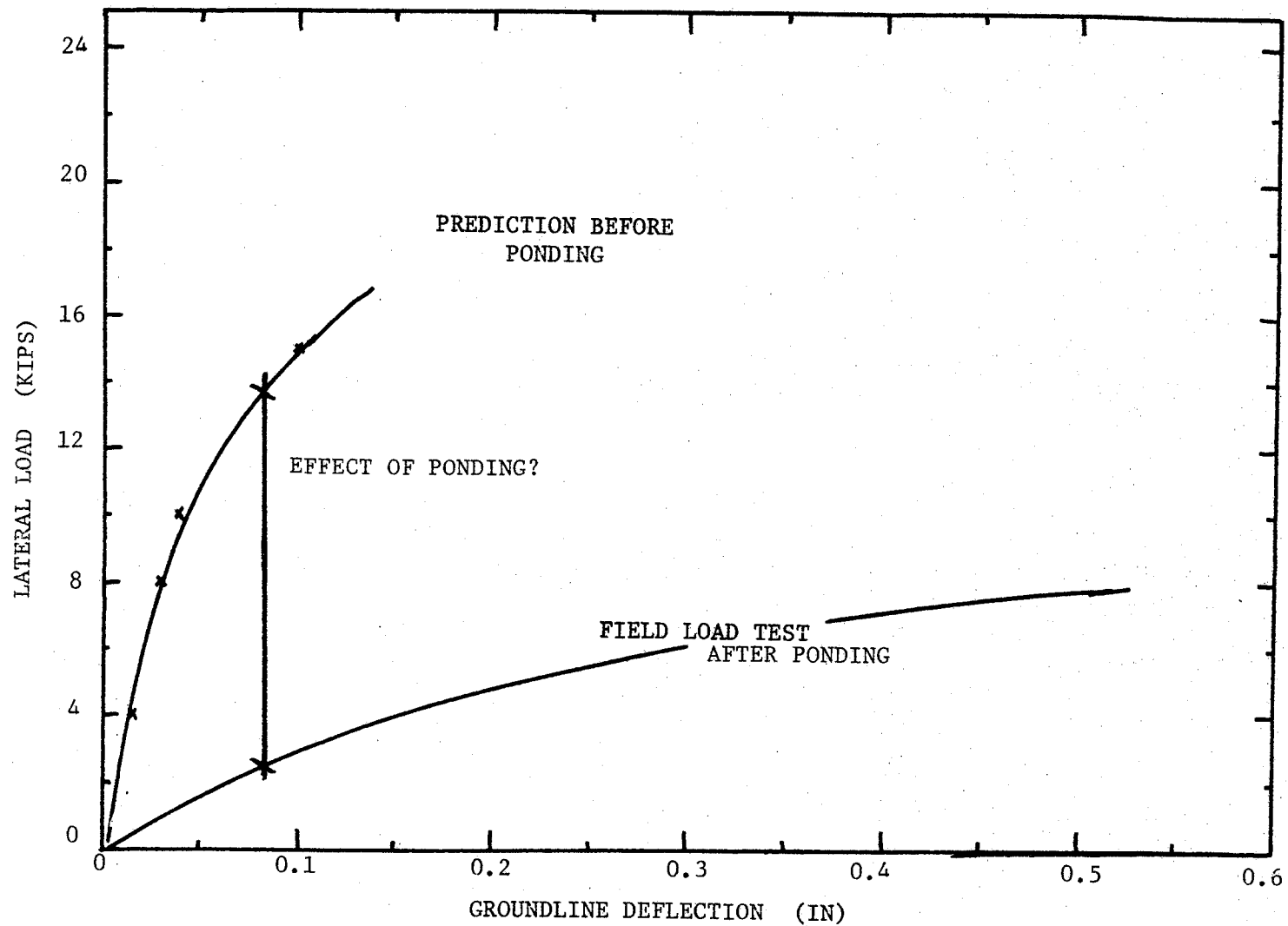
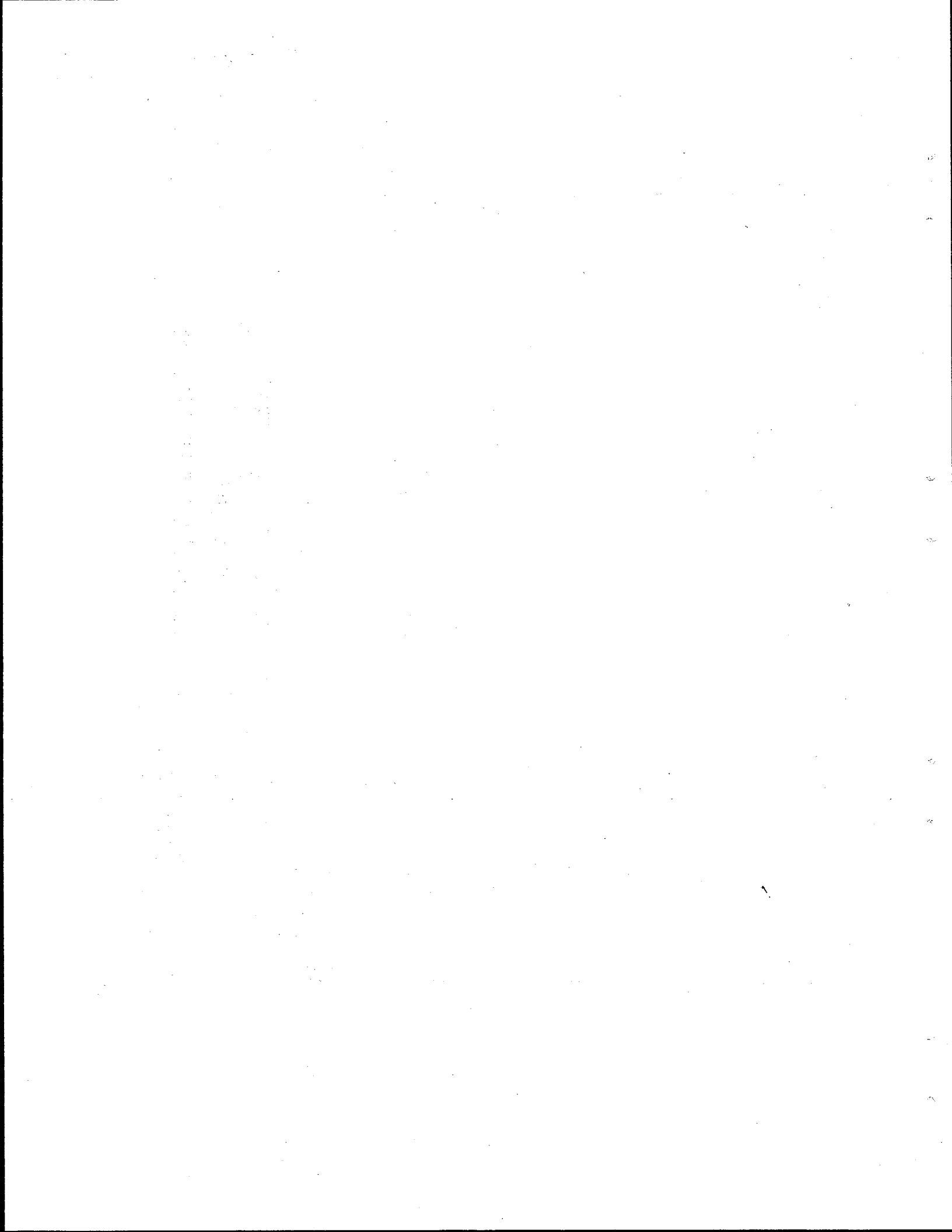


FIG. 35 - Manor Site: Comparison of Groundline Deflection for 6 in. Pile



CHAPTER 5. References

1. Baguelin, F., "Rules for the Structural Design of Foundations Based on the Selfboring Pressuremeter Test," Symposium on the Pressuremeter and its Marine Applications, Paris, April 1982.
2. Baguelin, F., Jezequel, J.F., and Shields, D.H., The Pressuremeter and Foundation Engineering, Trans Tech Publications, Rockport, Mass., 1978.
3. Briaud, J.-L., and Shields, D.H., "A Special Pressure Meter and Pressure Meter Test for Pavement Evaluation and Design," ASTM Geotechnical Testing Journal, September 1979.
4. Briaud, J.-L., Smith, T.D., and Meyer, B.J., "Laterally Loaded Piles and the Pressuremeter: Comparison of Existing Methods," ASTM Special Technical Publication on the Design and Performance of Laterally Loaded Piles and Pile Groups, June 1983.
5. Briaud, J.-L., Smith, T.D., and Meyer, B.J., "Pressuremeter Gives Elementary Model for Laterally Loaded Piles," International Symposium on In Situ Testing of Soil and Rock, Paris, May 1983.
6. Briaud, J.-L., Smith, T.D., and Meyer, B.J., "Using the Pressuremeter Curve to Design Laterally Loaded Piles," Offshore Technology Conference, Houston, May 1983.
7. DiGioia, A., Davidson, H.L., and Donovan, T.D., "Laterally Loaded Drilled Piers: A Design Model," Proceedings of a session entitled "Drilled Piers and Caissions", at the ASCE National Convention, St. Louis, October 1981.
8. Dunand, M., "Etude Experimentale du Comportement des Fondations Soumises au Renversement," These de Docteur Ingenieur, Institut de Mecanique de Grenoble, France, 1981.
9. GAI Consultants, Inc., "Laterally Loaded Drilled Pier Research Volume 1: Design Methodology, Volume 2: Research Documentation," Research Reports EPRI E1-2197, January 1982.
10. Gambin, M., "Calculation of Foundations Subjected to Horizontal Force Using Pressuremeter Data," Sols-Soils, No. 30/31, 1979.
11. Hansen, B.J., "The Ultimate Resistance of Rigid Piles Against Transversal Forces," The Danish Geotechnical Institute Bulletin No. 12, 1961.
12. Hughes, J.M.O., Goldsmith, P.R., and Fendall, H.D.W., "Predicted and Measured Behavior of Laterally Loaded Piles for the Westgate Freeways Melbourne," a paper presented to the Victoria Geomechanics Society, Australia, August 1979.

REFERENCES (Continued)

13. Imai, T., "The Estimation Method of Pile Behavior Under Horizontal Load Using LLT Test Results," submitted for publication, Proceedings of the Symposium on the Pressuremeter and its Marine Applications, Paris, 1982.
14. Kasch, V.R., Coyle, H.M., Bartoskewitz, R.E., and Sarver, W.G., "Lateral Load Test of a Drilled Shaft in Clay," Research Report 211-1, Texas Transportation Institute, Texas A&M University, 1977.
15. Menard, L., Bourdon, G., and Gambin, M., "Methode Generale de Calcul d'un Rideau ou Pieu Sollicite Horizontalement en Fonction des Resultats Pressiometriques," Sols-Soils, No. 20/23, 1969.
16. Smith, T.D., "Pressuremeter Design Method for Single Piles Subjected to Static Lateral Load," a dissertation submitted to Texas A&M University in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 1983.
17. Reese, L. C., Cox W.R., Koop, F., "Analysis of Laterally Loaded Piles in Sands," Sixth Annual Offshore Technology Conference, Houston, OTC paper 2080, 1974.
18. Reese, L.C., and Welch, R.C., "Lateral Loading of Deep Foundations in Stiff Clay," Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT 7, July 1975, pp. 633-649.
19. Matlock, H., "Correlations for Design of Laterally loaded Piles in Soft Clay," Preprints, Second Annual Offshore Technology Conference, Houston, 1970, OTC 1204, pp. 578-588.