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Seismic Design of Pile Foundations: Structural and Geotechnical Issues

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Seismic Design of Pile Foundations: Structural and Geotechnical Issues

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SYNOPSIS Research on soil-pile-structure interaction under dynamic loading over the past 20 years has led to a variety of analysis approaches of varying complexity to address a range of dynamic problems. Many of these analysis approaches have been adapted for use for the seismic design of pile foundations. In this paper, the various analysis methods are only briefly reviewed. The focus of discussion is on design concepts and issues more routinely used or encountered by structural engineers during seismic design of new or retrofitted pile foundation systems representative of those used for bridges and buildings.

INTRODUCTION

The intent of this paper, is to focus on design concepts and issues related to the seismic design of pile foundation systems representative of those typically used for bridges and buildings. Pile foundations for such structures, as shown for example in Figures 1 and 2, are normally required in the presence of softer more compressible soils, where design concerns relate to bearing capacity and allowable settlement. However, from a seismic design point of view, several other design aspects must be considered, including:

(1) How are the free field earthquake ground motions modified by or transmitted through the pile foundation system for use in evaluating structural inertial loads?

(2) How should the stiffness and damping characteristics of the pile foundation system be computed to allow incorporation in earthquake structural response calculations such as natural frequency and mode shape or time history computations?

(3) How are the calculated earthquake structural inertial loads (expressed in terms of base shears, moments and vertical loads) distributed back in to the pile foundation components in terms of pile bending moments, shears and axial stresses?

(4) Are the seismically induced pile moments, shears and axial loads excessive in terms of design criteria?

(5) Is degradation of lateral stiffness or skin friction under cyclic loading or liquefaction a design concern in relation to pile load capacity?

(6) Are cyclic or permanent ground displacements a design concern?

(7) What are the pile seismic design criteria – bearing capacity/settlement/uplift/structural pile damage?

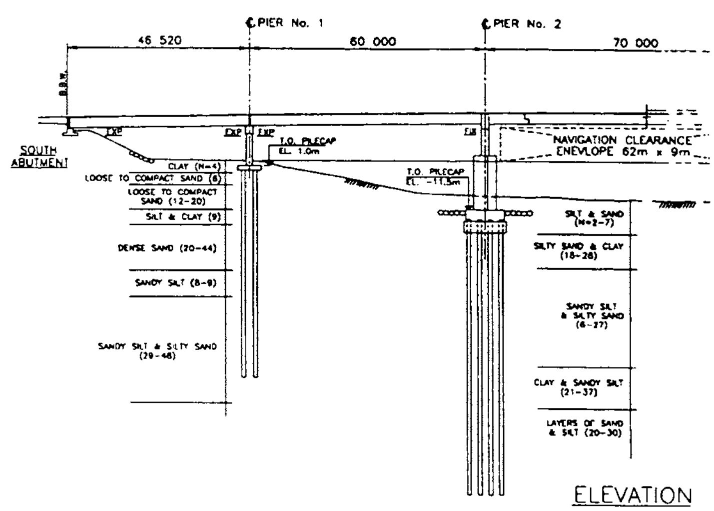


Figure 1 Representative Bridge Pile Foundation System

It is important to note that most pile foundation failures in past earthquakes have occurred due to ground liquefaction, as represented by the many case histories in the 1964 Niigata and Alaska earthquakes. Such failures can generally be attributed to either a loss of lateral or vertical pile support, or post liquefaction ground displacement or lateral spread. Reports of significant earthquake induced pile damage in non-liquefiable soils on the other hand are very rare. This is often attributed to the high factors of safety used for static design. Consequently, the role of soil-pile interaction on the seismic design of bridge and building structures in the case of non liquefiable sands or firmer cohesive soils is

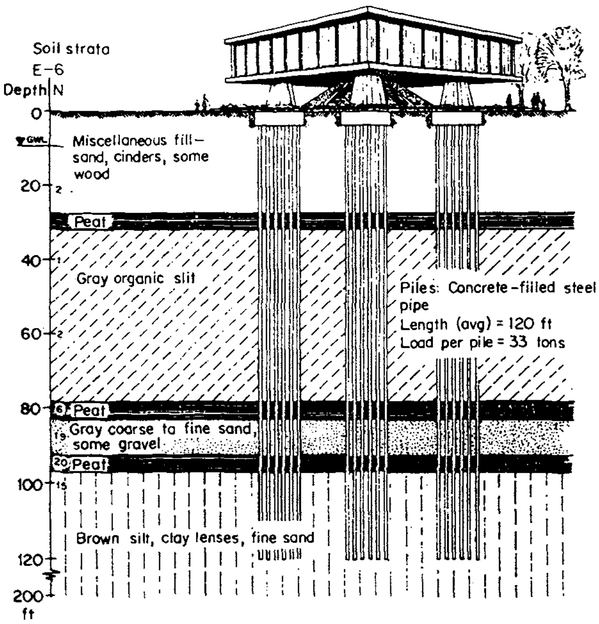


Figure 2 Representative Building Pile Foundation System

often given minimal attention by structural engineers.

Over the past several years, structural engineers in the United States have found the need to more closely examine the role of foundation interaction on the seismic design of bridge and building structures. This has resulted from the realization that many structures have been designed to older design codes, when the level of seismic loading was less than that currently accepted in revised codes. In approaching the seismic retrofit design problem, it is clearly desirable in economic terms, to reduce the level of conservatism in all aspects of the design. In the case of foundations, structural engineers are challenging the geotechnical engineer to examine more closely seismic performance criteria for foundations. As performance criteria for structures are now more often being evaluated in terms of non-linear time history or "pushover" analyses, the geotechnical engineer is being asked to determine the non-linear load-deformation characteristics of foundation systems and the consequences of pile foundations exceeding axial capacities. Such consequences could for example, be expressed in terms of permanent foundation deformations. By allowing transient foundation yield, in many cases the effect is to reduce structural seismic loading reducing both structural and foundation retrofit costs. However, the significance of accompanying permanent deformations need to be assessed.

In providing an overview of several of the above design concerns, the intention is to focus on the seismic design problems more from the point of view the structural engineer. The structural engineer, while recognizing the complexity of the problem with respect to both the analytical difficulties associated with a soil-pile-structure interaction problem under dynamic loading and variable soil stratigraphy, generally needs a relatively simple and pragmatic solution to the problem. This approach also provides an economic means for investigating the sensitivity of earthquake induced structural loading to uncertainties in foundation design parameters.

In considering historical approaches to soil-pile-structure interaction under dynamic loading, it is interesting to note that two distinct approaches have evolved. One approach evolved from slow cyclic lateral loading tests on piles in the 1970's, which were motivated originally by the need to develop pile design criteria for offshore structures subjected to wave loading. These studies led to analysis methods based on the use of non-linear Winkler spring concepts to model soil-pile interaction under cyclic lateral and axial loading, a modeling concept which goes back to Terzaghi. The second approach evolved from studies also in the 1970's originally motivated by machine foundation vibration problems, where the problem was driven by the need to develop frequency dependent stiffness or impedance functions to determine resonant frequency and amplitude characteristics or supporting pile foundations. In this approach, the model used was that of a vibrating mass supported by pile foundations in an elastic continuum.

Both of the above analysis approaches have been progressively developed and adapted to the problem of soil-pile-structure interaction under seismic loading over the past 20 years by numerous investigators, and provide good analytical tools for studying the seismic behavior of piles supporting structures. However, it is not the intention of this paper to provide a comprehensive overview of these developments, as recent state-of-the-art papers by Pender (1993) and Gohl (1993) provide an excellent summary of both analytical approaches and design implications. However, a brief summary of analytical concepts is given in the following section, together with the simplifications which are most commonly used in design practice for bridge and building pile foundations. These simplifications then provide basis for discussing design issues, first in relation to single pile behavior, and then with respect to pile groups.

ANALYSIS METHODS - BACKGROUND

The rigorous analysis of the dynamic response of a soil-pile-structure system to incoming seismic waves in a fully coupled manner is clearly a complex and difficult problem. If the soil is idealized as an elastic continuum, then a sub-structuring approach may be taken as illustrated in Figure 3. In this approach the problem is separated into a sequence of three sub-analyses:

(1) An analysis of the influence of the stiffness and geometry of a massless foundation system on the free-field ground motion, leading to modified structural input motions at the pile cap level (kinematic interaction).

(2) An analysis of the frequency dependent impedance characteristics of the foundation system under cyclic loading, in the form of a foundation stiffness matrix.

(3) An analysis of the inertial response of the structure to the pile cap input motions (from step 1), using the pile cap stiffness matrix to account for foundation compliance (inertial interaction).

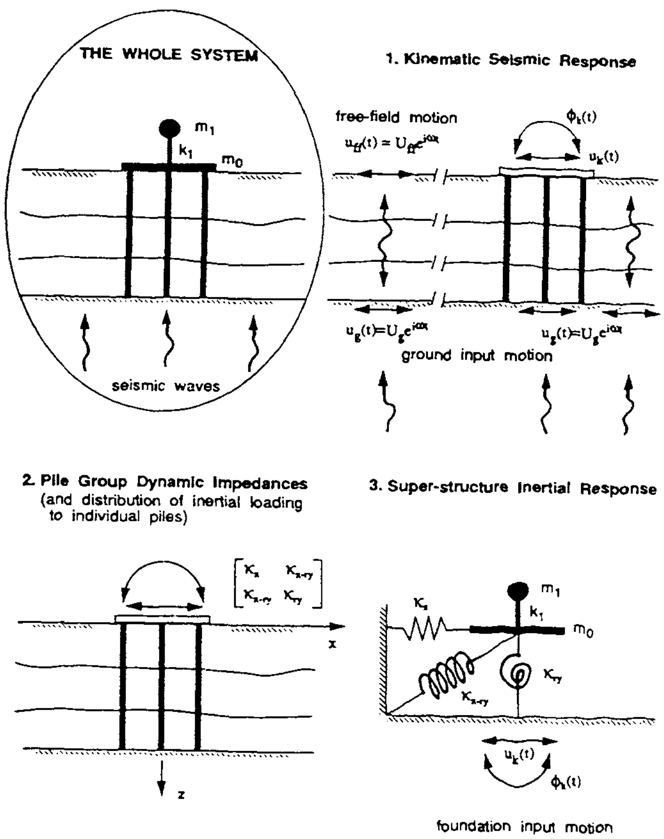


Figure 3 General Procedure for Seismic Soil-Pile Foundation-Structure Interaction (Gazetas et. al., 1992)

Gazetas et. al. (1992) reviews and describes this approach in more detail. An application of the approach to study bridge response to soil-pile-structure interaction under earthquake loading is described by Makris et.al. (1994).

The subject of elastic impedance functions for piles and pile groups has been studied by numerous researchers, and a large number of closed form analytical solutions are available. Comprehensive reviews of much of this work have been given by Novak (1991) and Pender (1993). Early research on this subject was stimulated by the need to establish analysis methods for vibrating machine foundations, where the higher frequencies of loading lead to radiation damping, and foundation stiffness is strongly frequency dependent. In addition, amplitudes of vibration are generally small, and the assumption of an elastic soil may not be unreasonable.

However, soil-pile interaction under earthquake induced inertial loading, can lead to strongly non-linear soil behavior particularly in the vicinity of the pile interface. Whereas non-linear interface springs and sliding elements can be coupled to elastic continuum solutions to obtain modified impedance functions such as in the studies described by Nogami et. al. (1992). The use of the above approach becomes impractical for more routine structural design under seismic loading. Fortunately, given the relatively low frequency range of earthquake inertial loading and the nature of representative pile foundation systems for bridges and buildings, stiffness functions are in most cases essentially frequency independent, and static loading stiffness values are often a reasonable approximation. In addition, the radiation damping component of energy loss arising from wave propagation away from the foundation is considerably reduced at lower frequencies, particularly in the presence of non-linear soil behavior. Whereas fully coupled non-linear solutions to the seismic loading problem using finite element methods are theoretically possible and have been used to a limited extent, the analytical complexity is again very daunting and impractical for routine design.

Given the complexity of non-linear coupled models, the Winkler model, represented by a series of independent or uncoupled lateral and axial springs (linear or non-linear) simulating soil-pile interaction in the lateral and axial directions, provides the most convenient means of analyzing the response of pile foundation systems to earthquake loading. A Winkler spring approach to coupled dynamic lateral pile analysis under earthquake loading reflecting both kinematic and inertial interaction, was developed by Matlock et. al. (1978, 1981) and is shown schematically in Figure 4. The pile is modeled by beam-column elements, supported by linear or non-linear spring elements. Free field earthquake ground motions determined from one dimensional site response analyses are used as displacement input motions for the spring elements. Pile cap rotational stiffness arising from pile group axial loads, may also be included. The structure is represented by a simple stick model. The analysis method is embodied in the computer program SPASM (Single Pile Analysis with Support Motion) described by Matlock et. al. (1978)

Whereas the effects of kinematic interaction can be significant for some pile foundation-soil configurations (for example, larger diameter piles in soft soils and for sudden changes in soil stiffness of depth), for most pile foundation systems, piles may be assumed to deform in a compatible manner with the free field. For such cases,

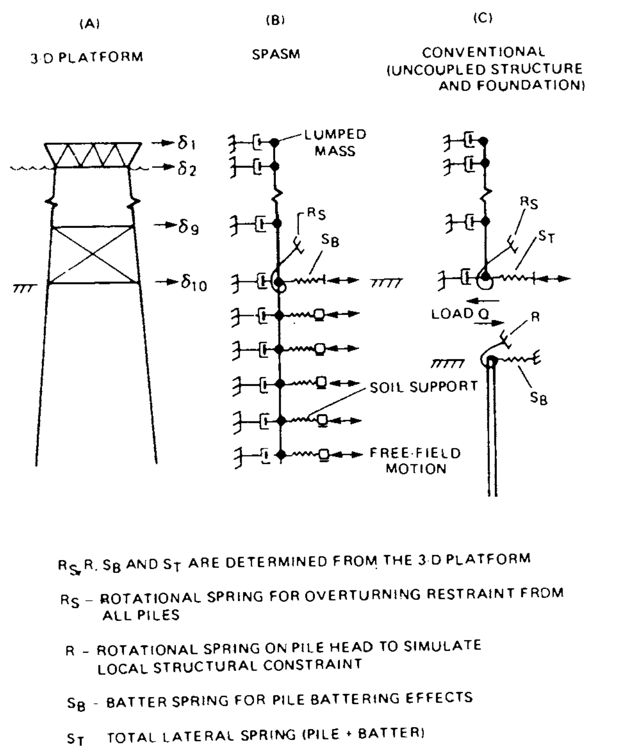


Figure 4 Schematic Idealization of Soil-Pile-Structure Model (Matlock et. al. 1978)

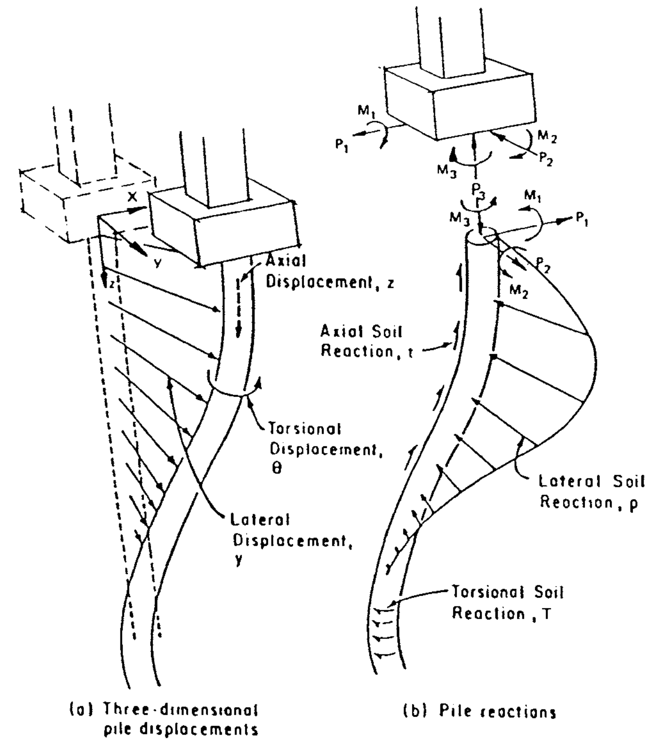


Figure 5 Three Dimensional Soil-Pile Interaction (Bryant and Matlock, 1977)

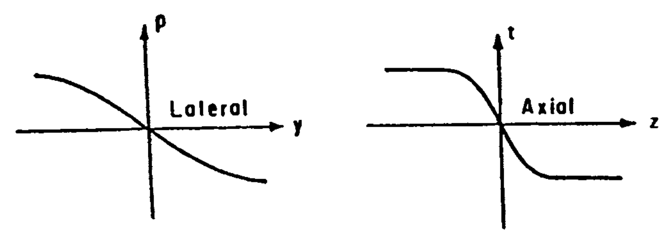


Figure 6 Non-linear Soil Support Curves

free field displacements are generally much less than those induced by inertial interaction. Hence we may assume inertial interaction dominates pile foundation response, and that stiffness functions may be represented by values under static or slow cyclic conditions. The problem then becomes similar to that shown in Figure 3 (step 3), with foundation input motions assumed to be near surface free field motions. This approach has had widespread application in the analysis of offshore structures to wave and earthquake loading, and for soil-pile-structure interaction analyses in general, as documented by Hadjian et. al. (1992) in a state-of-the-practice survey.

Three dimensional inertial interaction may then be represented by the seismic soil-pile interaction concept shown in Figure 5, with the corresponding non-linear p-y and t-z curves (defined by pile-load tests or theoretical analyses) represented conceptually in Figure 6. The remainder of the paper will focus primarily on this analysis approach, which, considering all the uncertainties and complexities of more rigorous solutions, is adequate for most engineering applications.

LATERAL LOADING - SINGLE PILES

As discussed above, the most widely used practical approach for the lateral loading and analyses of single piles, is the so called beam-column method. As shown schematically in Figure 7, this technique models the pile member as a series of beam-column elements, with discrete springs to model the soil support. With the above simplifications, the effects in non-homogeneous and nonlinear soil behavior on lateral loading response can readily be solved using computer programs such as LPILE (Reese, 1985) and BMCOL 76 (Bogard and Matlock, 1977).

Comprehensive overviews of the various methods developed to determine the spring paremeters have been presented by Pender (1993) and Goh (1993). Due to the complexity of soil behavior and the disturbance caused by pile driving, the empirical experimental approach for determining p-y curves still appears the most reliable approach at the present time. Construction of p-y curves at each depth involves formulation of the ultimate resistance p_u force per unit pile length), development of the initial tangent stiffness E_s, and fitting a hyperbolic or other non-linear curve shape.

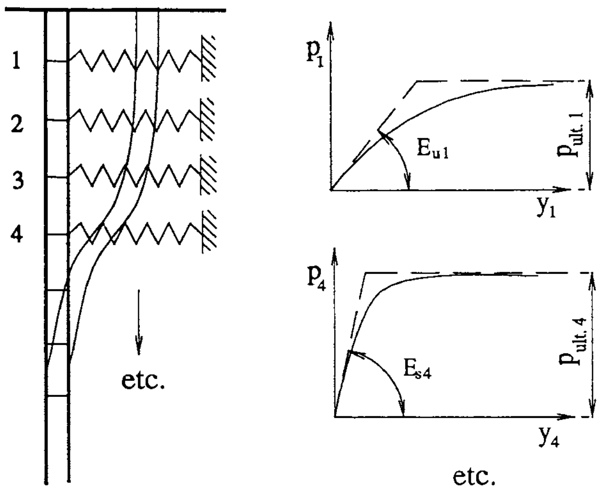


Figure 7 Beam-Colum Winkler Spring Model

For sands, the pile load tests conducted at Mustang Island and p-y correlation developed by Reese et. al.(1974) and the American Petroleum Institute (API) are often used, as are the simplifications proposed by O'Neill and Murchison (1983). Note that these correlations are based on the initial effective stress state and reflect drained conditions. The influence of pore pressure increases and potential liquefaction are discussed in later sections. Methods for constructing non-linear p-y curves in cohesive soils have been presented by Matlock (1970) Reese and Welch (1975) API and O'Neill and Gazioglu (1984). The approach used depends on factors such as whether the soil is normally consolidated or over-consolidated, the potential influence on cyclic degradation on lateral resistance, and the potential for near surface soil-gapping. The successful application of these methods to evaluate cyclic lateral load tests on bored piles, has been illustrated for example, by Goh and Lam (1988).

Sensitivity to boundary conditions

To illustrate the sensitivity of pile behavior to boundary conditions at the pile head, the case of a one foot diameter pile embedded 20 feet in a uniform sand ($\phi = 35^{\circ}$) is considered. p-y curves were computed using the API (1982) recommendations. Three different boundary conditions were assumed for lateral load-deflection analyses.

1) fully free (zero moment), 2) partially restrained (some finite rotational constraint), or 3) fully fixed (zero slope). The fully fixed pile-head condition can rarely be achieved in practice. In reality some finite rotational constraint should be assumed even though the pile may be rigidly cast into the pile cap. All three types of connection (fully free, fully fixed and partially restrained) are analyzed in the following example and their implications compared and discussed.

Solutions were obtained for a range of lateral loads on the pile head using the BMCOL 76 Program. The resultant pile-head load versus pile-head deflection curves are plotted in Figure 8 for the three pile-head connections. In practice, it is commonly assumed that the free-head and the fixed-head pile assumptions serve as extreme bounds. The solutions support this assumption as the load-deformation curve for the partially restrained case falls between the free and the fixed head cases. However, this assumption is invalid for peak moment, as discussed below.

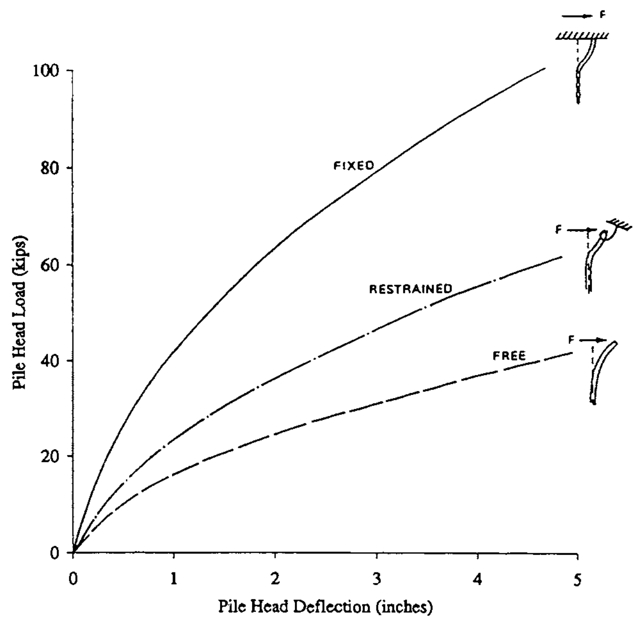


Figure 8 Sensitivity of Pile Head Deflection to Pile Head Constraint (Lam and Martin, 1984)

Figure 9 presents the cross-plot of maximum pile moment versus the corresponding pile head loads. This plot reveals that the pile-head load versus maximum moment curve for the partially restrained case falls below the range bounded by the free-head and the fixed-head curves. This anomaly is explained in Figure 10 which presents the moment distribution along the pile length for the three cases of pile-head conditions for a specific pile head load of 40 kips. A peak positive moment occurs at some depth below the pile top for a free-head pile, whereas a peak negative moment occurs at the pile top for the fixed head case. The partially restrained case gives a more balanced distribution of moment, where the peak negative moment at the pile top is roughly the same as the peak positive moment at depth. This balanced distribution results in a lower maximum moment as compared to either the free-head and the fixed head case.

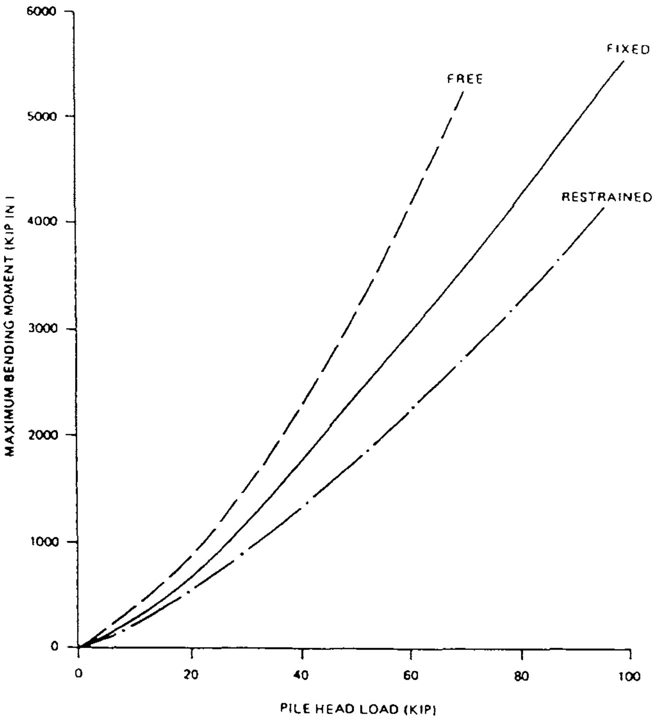


Figure 9 Sensitivity of Maximum Moment to Pile Head Constraint (Lam and Martin, 1984)

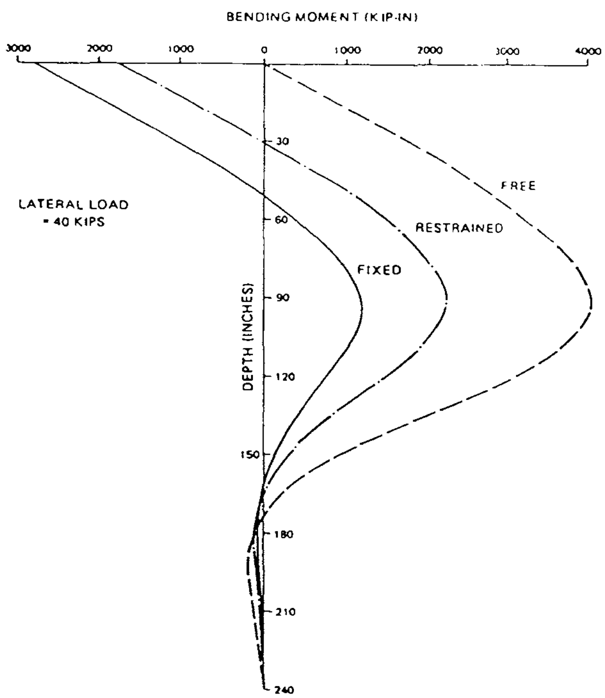


Figure 10 Sensitivity of Moment Distribution to Pile Head Constraint (Lam and Martin, 1984)

Sensitivity to soil support

The p-y curves for sand used in the above BMCOL solutions are based on the recommendations given by API for design of long offshore piles. Additional beam-column solutions are solved and presented below with the p-y curves modified to account for a number of soil-pile interaction aspects associated with earthquake considerations. The partially-restrained pile-head condition is chosen for the sensitivity studies. The resultant beam-column solutions of pile-head load versus deflection curves are compared to the bench mark case previously described, and are presented in Figure 11. Some general comments of these comparisons are briefly discussed below.

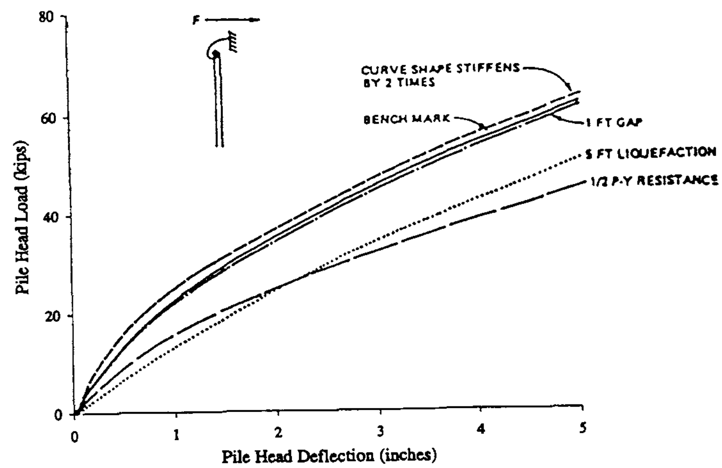


Figure 11 Sensitivity of Load-Deflection Curve to p-y Curves (Lam and Martin, 1984)

(a) p-y Curve Shape. The API recommendations on the p-y curves place emphasis on the large deformation range (the ultimate resistance). Less emphasis is placed on the selection of the initial stiffness of the p-y curve as it has little influence on the pile solution for the large deformation range. Under earthquake loading, smaller pile deflections may occur, where the magnitude of the initial stiffness may warrant a more critical examination. To illustrate the effect of stiffening on initial soil response without changing the ultimate resistance, the benchmark p-y curve was modified such that the abscissa (the pile deflection) of the p-y curves were uniformly reduced to half the deflection values of the benchmark curve. As a result, the pile-head load deflection curve could be expected to be stiffer than the benchmark case. However, the difference is seen to be very small, indicating that an accurate assessment of soil strength is more important then low strain stiffness in evaluating pile head stiffness.

(b) Soil Gap Effects. During cyclic loading, the formation of a conical gap at the soil surface has been observed by a number of researchers. To simulate this gapping effect, one pile diameter (one foot depth) of the soil support (p-y curves) was eliminated at the soil surface. Below the one-foot depth, the same benchmark p-y curves were used. As shown in Figure 11, the resultant pile stiffness was only slightly reduced because of the gap effect. However, gapping effects could be more significant for clays, where cyclic loading could lead to progressively increasing gap depths at the pile head.

(c) Degraded p-y Curve. During cyclic and earthquake loading, the soil resistance may be progressively degraded by the effects of cyclic loading. The soil resistance on the p-y curves are reduced to half of the benchmark case, to arbitrarily account for potential cyclic degradation effects. The resultant pile-head load-deflection curve is shown in Figure 11, and is significantly reduced especially at the small deflection range.

(d) Liquefaction Effects. For loose sand deposits, the soil beneath the water table can potentially liquefy during an earthquake, resulting in a loss of soil resistance. To simulate a liquefaction condition, the p-y curves at zero to five feet depth were removed. However, the soil-support curves below five-foot depth were kept the same as the bench-mark case. The resultant pile-head load-deflection solutions are presented in Figure 11. The pile-head stiffness is significantly reduced, especially at the small deflection range. However, the load-deformation curve becomes more linear, indicating that a higher proportion of the pile-head deflection is associated with the compliance of the cantilever beam over the liquefied zone.

The above sensitivity studies suggest that the cyclic degradation and liquefaction effects can potentially be more significant in affecting pile behavior and a greater emphasis should be placed on developing design guidelines to account for these aspects. Further studies illustrating the effects of liquefaction on pile response are discussed later in the paper.

Equivalent linear subgrade modulus

Methods for construction of nonlinear p-y curves for both sand and clay have been cited above. Nonlinear pile solutions under lateral loading in layered soil deposits usually require the aid of computer models. However, in many cases, due to the insensitivity of overall pile behavior to the variation of soil support characteristics and because the significant zone of soil-pile interaction is very localized near the point of loading, linear representation of the soil stiffness yield pile solutions of reasonable accuracy. The following recommendation for an equivalent linear soil stiffness leads to a reasonable approximation of the nonlinear solution in limited ranges of pile deflection.

The modulus of horizontal subgrade reaction for sand recommended by Terzaghi and described by O'Neill and Murchison (1983) has been widely used in practical applications. The magnitude of the support spring on piles is assumed to be independent of the pile diameter and vary linearly with depth for sands, that is:

$$E_s = f\,z \qquad (1)$$

where

E_s = the stiffness of the support spring in force per unit length per unit deflection,
f = a coefficient which depends on the density or friction angle (see Figure 12), and
z = depth from grade level.

The values of f recommended by Terzaghi as shown in Figure 12, are smaller than those recommended by Reese et. al. (1975). The value recommended by Reese et. al. corresponds to the initial tangent stiffness of the load-transfer characteristics. The value recommended by Terzaghi corresponds to the secant stiffness of the load transfer behavior at typical design load levels.

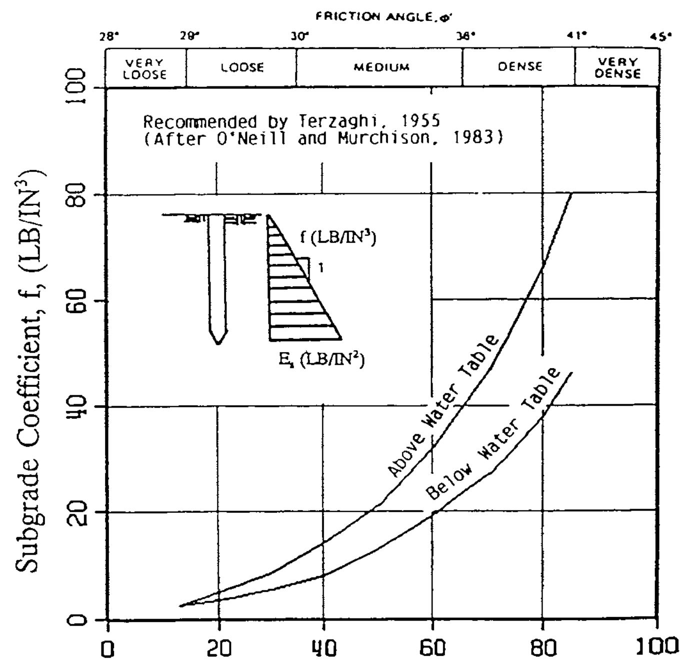


Figure 12 Coefficient of Variation of Subgrade Modulus with Depth for Sand

To provide practicing engineers with a feel for the range of validity of Terzaghi's recommendation of linear horizontal subgrade stiffness, a set of pile head load-deflection solutions is presented in Figure 13. Both free and fixed pile head boundary conditions are examined. The densities of the sand range from loose to dense with corresponding friction angles ranging from 30 to 40 degrees. A comparison of the solution using Terzaghi's linear stiffness with the nonlinear p-y approach indicates that Terzaghi's recommendations leads to an equivalent secant stiffness of the pile at about 0.2 to 1.0 inch of pile head deflection depending on the friction angle or the density of sand. Based on sensitivity studies, typical pile head deflections for many highway bridges under earthquake loading could range from 0.2 to 2.0 inches.

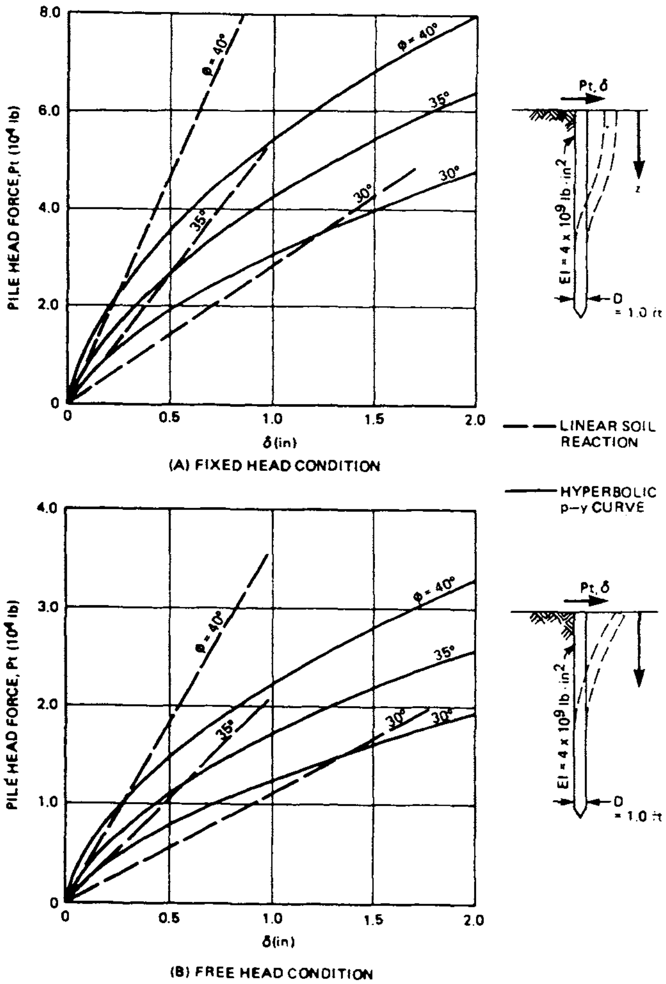


Figure 13 Comparisons of Linear and Non-Linear Solutions for Sand (Lam and Martin, 1986)

In the case of clays, Terzaghi recommended a model of constant subgrade stiffness depending on the shear strength for clay. From our experience, a model where subgrade stiffness increases with depth provides a better fit to pile load test data for soft to medium stiff clays than the constant stiffness model due to the following reasons:

(1) The shear strength of most soft to medium stiff clay sites tends to increase with depth.

(2) Most importantly, pile deflections decrease with depth under structural loading conditions and therefore the soil response tends to be more nonlinear at shallow depth resulting in an equivalent linear stiffness that increases with depth.

Lam and Martin (1986) presented an equivalent linear subgrade stiffness model based on the Matlock's formulation of soft clay p-y curve criteria in the following form:

$$E_s = k_0 + k_1 z$$

where

$$k_0 = 0.6c/\varepsilon_c$$

$$k_1 = \frac{0.2}{\varepsilon_c}\left(\gamma + \frac{J_c}{D}\right)$$

J_c is empirical constant (0.25-0.5)
z is the depth below the grade level
D is the pile diameter
c is the undrained shear strength
ε_c is the strain amplitude at one-half of the peak deviatoric stress level

γ is the effective unit weight

Results of a sensitivity study to compare the procedure described above for linear and nonlinear pile solutions for clay are shown in the pile head load-deflection solutions presented in Figure 14. Both free-and fixed-pile-head boundary conditions are examined. The undrained shear strength ranges from 0.5 ksf to 5.0 ksf. It can be seen from the comparison that the linear subgrade stiffness approach yields reasonable solutions up to 0.5 inches of pile-head deflection.

It should be recognized that there is no rigid rule on the appropriate subgrade stiffness model. A simpler formulation than the two parameter approach (k_0 and k_1) can be developed for clays using a single parameter model (E_s = fz) similar to that developed for sands. Lam et al. (1991) presents an approximate relationship for f as a function of cohesive shear strength as shown in Figure 15. The relationship was developed for a typical 12-inch diameter concrete pile such that the linear pile solution (using the linear subgrade stiffness) will match the corresponding nonlinear pile solution using the Matlock's soft clay criteria. Figure 15 also presents the relationship of f versus cohesion recommended in NAV - DM7.02 for fine grained cohesive soils.

The above relationship is considered appropriate for a specified range of conditions:

- Smaller piles with pile diameters less than 24 inches.
- Soft to medium stiff clays.
- Pile head deflection ranging from 0.5 to 1.5 inches.

For other conditions including larger pile diameters and for stiff to hard clays, other relationships may be more appropriate. The relationship in Figure 15 may be non-conservative especially for stiff to hard over-consolidated clays which can exhibit very pronounced strain - softening behavior. For this reason, we recommended a limit to the coefficient of variation in subgrade stiffness f of about 40 lb/in^3 corresponding to a cohesion of about 2.5 ksf.

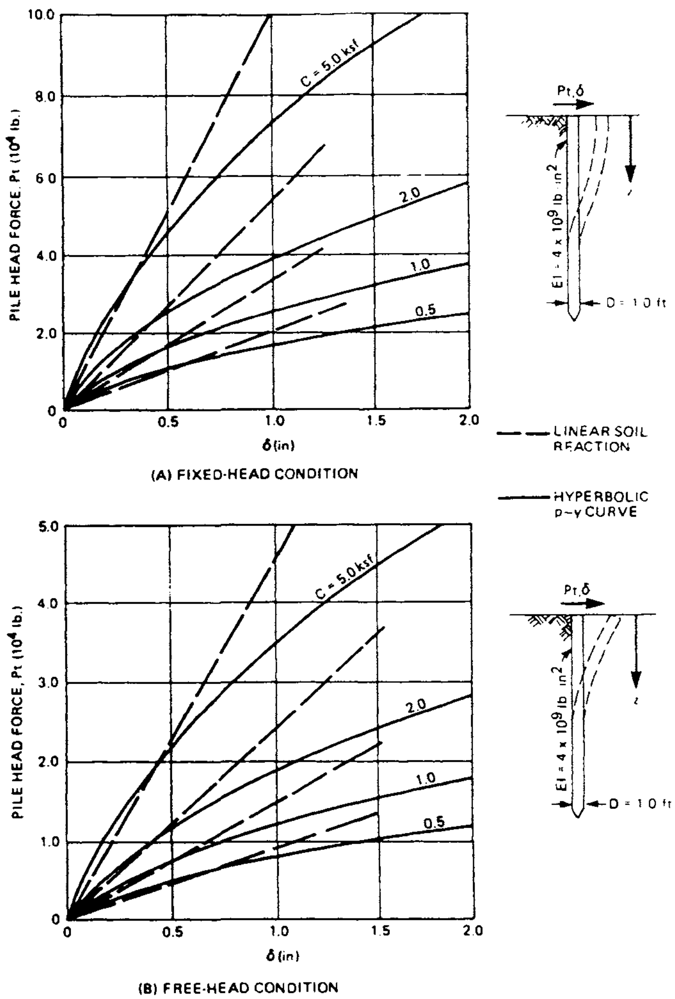


Figure 14 Comparisons of Linear and Non-Linear Solutions for Clay (Lam and Martin 1986)

Pile-Head Stiffness Matrix

For structural seismic response evaluations, the development of an equivalent linear pile head stiffness matrix reflecting the relationship between applied pile head lateral loads and moments and corresponding lateral deflections and relations is a necessary step. Whereas computer programs using site specific p-y curves may be used to determine equivalent linearized pile head stiffness coefficients, it is of interest to develop graphical relationships using the linearized subgrade modulus simplifications described above, for preliminary design or sensitivity evaluations. The pile-head stiffness relations may be expressed by the equations:

$$P_f = K_\delta \, \delta + K_{\delta\theta} \, \phi \qquad (2)$$

$$M_f = K_{\delta\theta} \, \delta + K_\theta \, \phi \qquad (3)$$

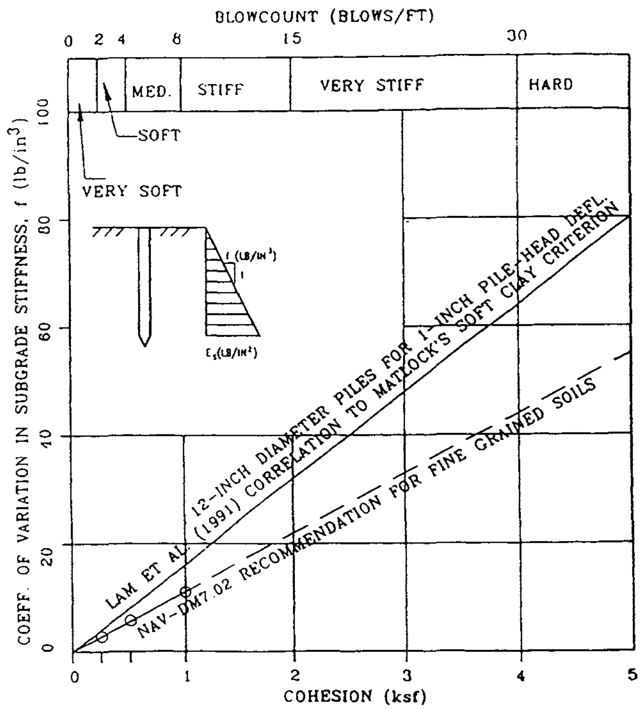


Figure 15 Coefficient of Variation of Subgrade Modulus With Depth for Clay

where the stiffness coefficients K_δ and K_θ, represent the force per unit horizontal deflection with zero rotation, and the moment per unit rotation with zero deflection at the pile head, respectively. The cross coupling coefficient $K_{\delta\theta}$ represents either the moment needed to maintain zero rotation on unit deflection or the force required to maintain zero displacement on unit rotation at the pile head.

Pile head stiffness coefficients K_δ, K_θ and $K_{\delta\theta}$ are shown in Figures 16 through 18 as a function of the bending stiffness of the pile EI and the coefficient of variation f of soil reaction modulus E_s with depth, for cases where the pile head is fixed and is located at the ground surface. In most cases where pile groups are used for foundation support, pile heads are embedded beneath the surface. The effect of embedment on the coefficient K_δ is shown in Figure 19.

To illustrate the use and implications of the above graphs, we consider a standard cast in-situ reinforced concrete pile commonly used by The California Department of Transportation (16 inch diameter, EI = 9.7×10^9 in^2-lb). Assuming a sandy soil ($\phi = 30^\circ$) and fixed head conditions, from Figure 12, the coefficient f = 10 lb/cu. in and from Figure 16, the lateral stiffness $K_\delta = 4 \times 10^4$ lb/in. From Figures 17 and 18, the rotational stiffness $k_\theta = 2.3 \times 10^8$ in-lb/rad and the cross coupling stiffness $K_{\delta\theta} = 2.3 \times 10^6$ lb. If the embedment depth is 5 ft., K_δ increase to 8×10^4 lb/in (as determined from Figure 19), which is twice the value for no embedment.

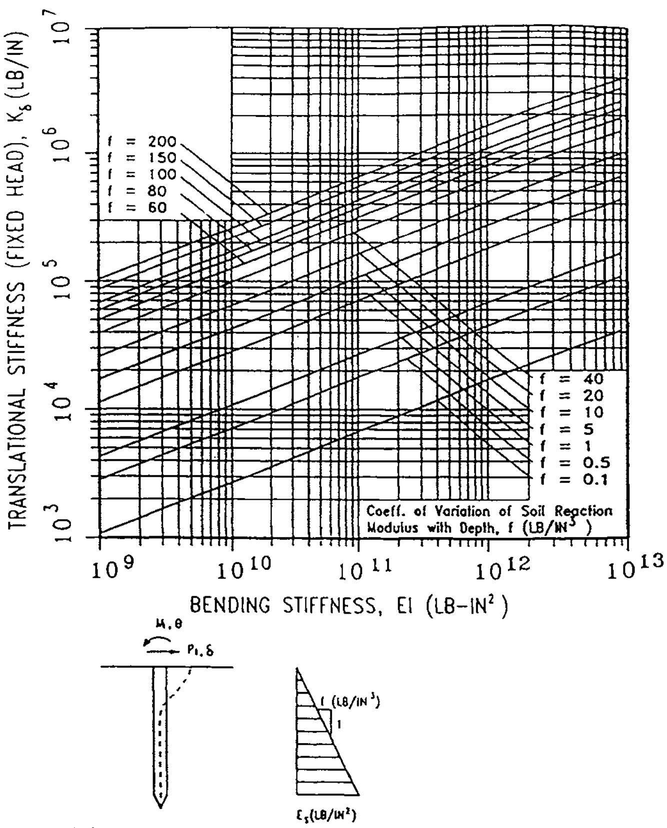


Figure 16 Coefficient for Fixed Pile Head Lateral Stiffness (Lam et.al., 1991)

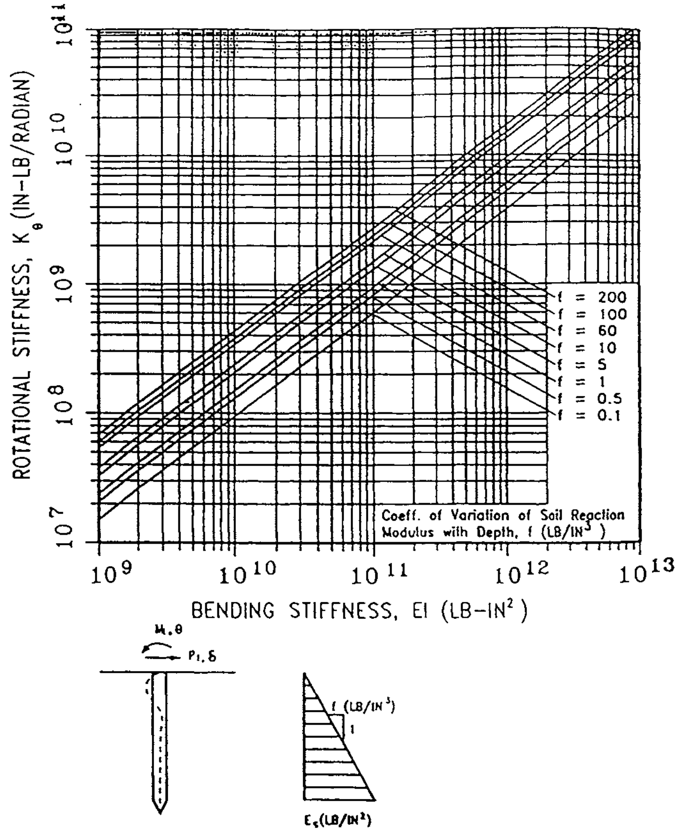


Figure 17 Coefficient for Fixed Pile Head Rotational Stiffness (Lam et. al., 1991)

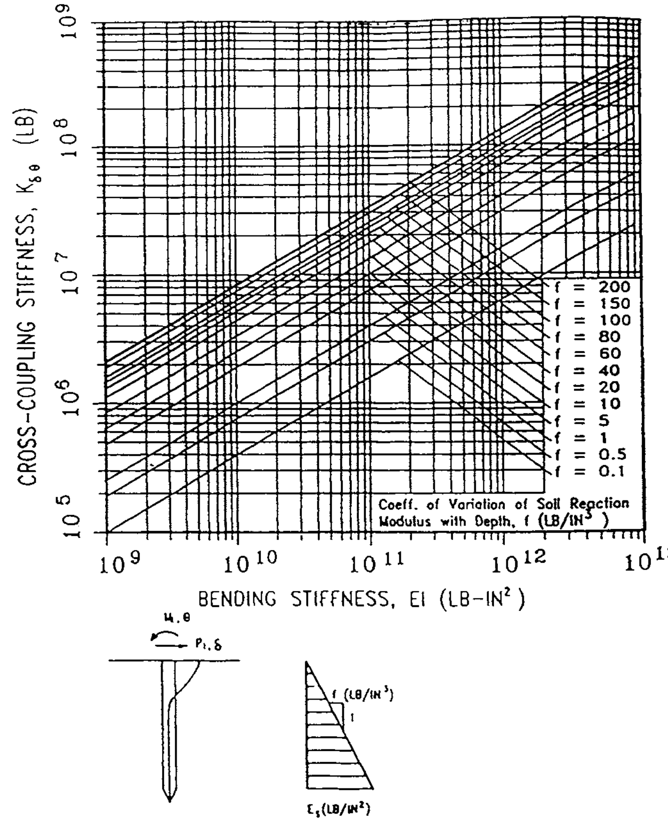


Figure 18 Coefficient for Fixed Pile Head Cross Coupling Stiffness (Lam et. al. , 1991)

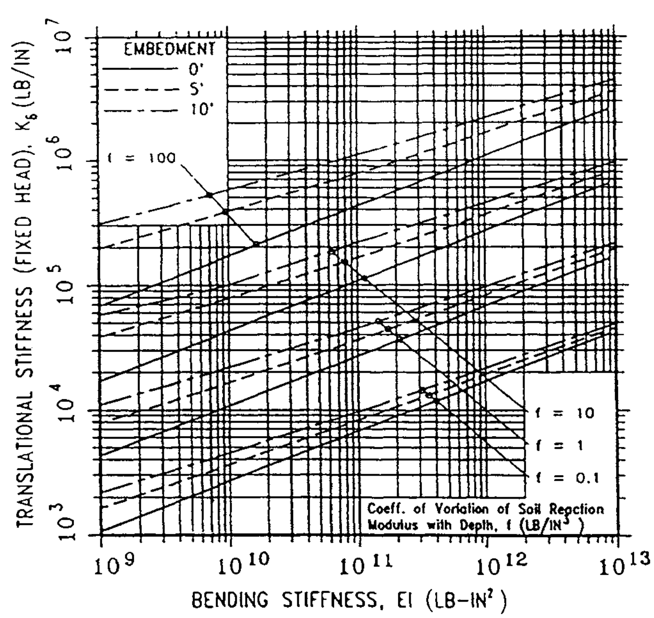


Figure 19 Coefficient for Fixed Pile Head Lateral Stiffness-Variable Embedment Depth

If the pile is pinned at the pile head, the zero pile head moment leads to the following relationship between pile head rotation θ and deflection δ:

$$\theta = -(K_{\delta\theta} / K_{\theta})\, \delta \qquad (4)$$

Substituting into the pile head force equations (2) and (3), the lateral stiffness of a free head pile is given by $K_{\delta} - K_{\delta\theta}^2/K_{\theta} = 1.9\text{x}10^4$ lb./in.

From this example, it can been seen that the lateral stiffness can vary significantly depending on both pile head connection fixity and embedment depth, again reinforcing the conclusion that pile head boundary conditions are of more significance than the precise selection of soil parameters.

Effects of Liquefaction

Another major lateral loading problem encountered by designers is that of pile design in liquefiable soils. Generally, the liquefaction problem affects surficial soil layers (say in the upper 30 to 50 feet) of a loose sand deposit. Very often zero lateral soil resistance (i.e. p-y resistance in pile analysis) is assumed for the liquefied soil layers. In many cases, if the soil liquefies (to depths greater than 20 feet), foundation design using conventional smaller diameter piles becomes virtually impossible and the only recourse would be to perform site remediation or to use very large diameter drilled shafts which are difficult to construct in liquefiable sites.

Progressive increases in free field pore water pressures in saturated sands arising from earthquake loading, will lead to similar progressive reductions in lateral pile resistance. The effects of such reductions on pile head stiffness have been illustrated by Finn and Martin (1980) by considering the case of a 48 inch diameter steel pipe pile embedded in 30m of loose sand overlying a firm alluvium. Progressive increases in free-field pore pressures under earthquake loading as a function of depth were generated by the dynamic effective stress response program DESRA (Lee and Finn, 1978), leading to liquefaction to a depth of about 10 ft. after 10 seconds of shaking. Initial p-y curves were established using API criteria. The slopes of the p-y curves at the origin were then degraded as a function of the reductions in the square root of the vertical effective stress, while for deflections greater than D/60 (D = pile diameter) where strength characteristics dominate lateral resistance, the p-y curve was degraded in proportion to the vertical effective stress, leading to zero resistance in liquefied zones. The influence of the degradation on lateral stiffness (normalized by initial stiffness) for fixed and free-head conditions and for a range of pile head deflections, is shown in Figure 20. Liquefaction to a depth of about 10 feet after 10 seconds, reduced the lateral stiffness to about one third of initial stiffness values.

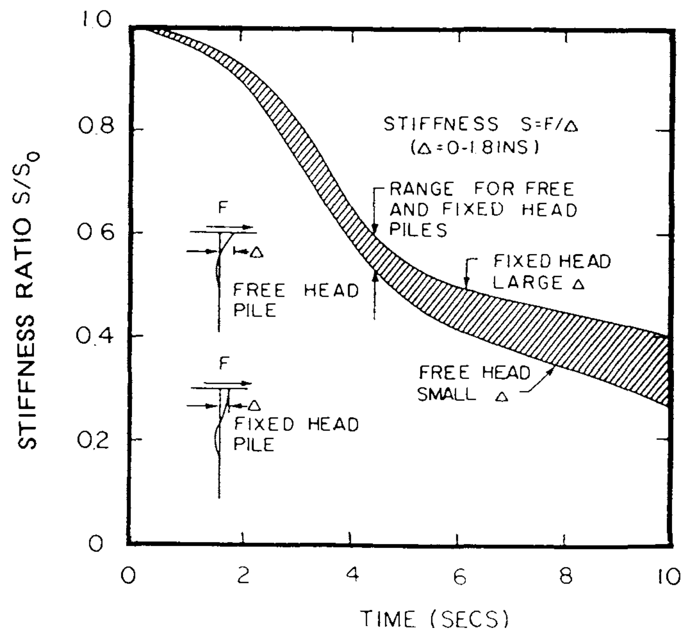


Figure 20 Lateral Stiffness Degradation with Pore Water Pressure Increase and Liquefaction (Finn and Martin, 1980)

Whereas pore pressures generated by free field earthquake response are likely to dominate degradation effects, the earthquake induced cyclic inertial interaction of the pile with the surrounding soil will also tend to generate localized pore pressure increases around the pile head. The effects of such pore pressure increases are illustrated in tests described by Scott et. al (1982). In these tests, an instrumented 24 inch steel pipe pile embedded in saturated sand was subjected to dynamic lateral loads by the use of two counter-rotating mass vibratory shakers as shown in Figure 21. Loading rates were in the range 1-8 hertz. Free field pore pressure measurements indicated that partial liquefaction occurred close to the pile head (small sand boils were observed along with some subsidence at the ground surface). Figure 22 illustrates representative back calculated cyclic p-y curves from the test results, (Ting, 1987). Note the strain softening, hysteric and gapping phenomena at shallow depths and the nearly linear behavior at depths of 5-6 pile diameters.

Figure 23 presents the measurements in pile head stiffness during the vibratory test as shown by the load-deflection measurements from two series of tests: (i) at a relatively lower cyclic load (Test No. 6)of 2.47 kips and (ii) at a higher cyclic load (Test No. 9) of 6.12 kips. In addition to the two points showing measurements from the vibratory tests, a series of hindcast beam-column analyses were conducted and are also shown in figure 23. The solid line presents results of the nonlinear beam-column analyses using the p-y curve approach using conventionally adopted soil parameters for the site soil condition (i.e. a friction angle of 35 degrees and a

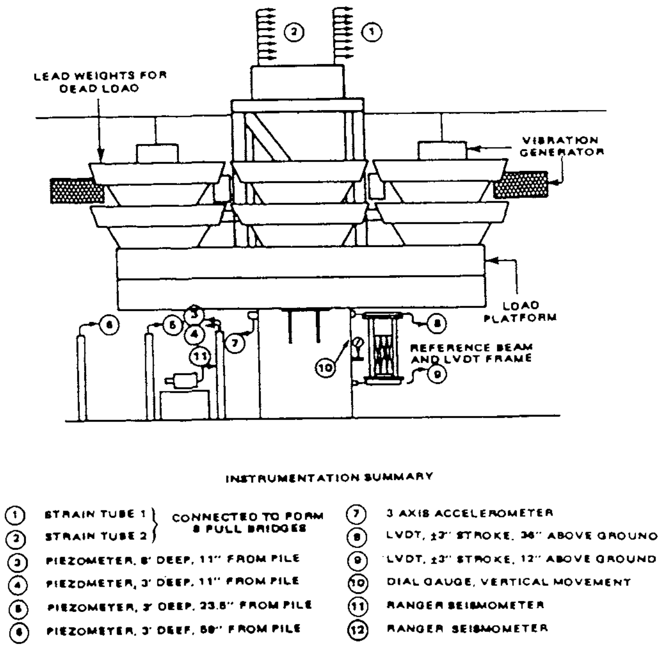


Figure 21 Test Configuration for Full-Scale Vibrating Pile Test (Scott et. al., 1982)

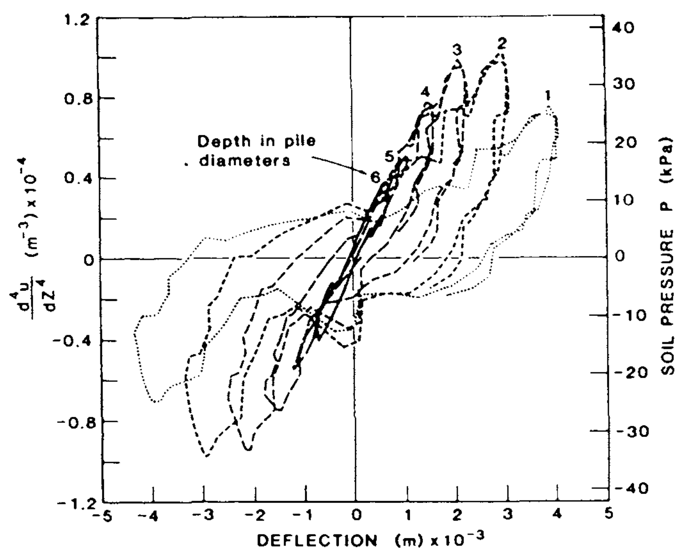


Figure 22 Back Calculated p-y Curves from Vibrating Pile Load Test (Ting, 1987)

coefficient in variation in subgrade stiffness of 80 pci to define the initial tangent stiffness of the p-y curves). This set of soil parameters have been found to provide a reasonable fit to most of the full-scale hydraulic ram pile load tests at the loading rates that can be achieved, i.e. at minutes per cycle. As shown in the figure, for the lower loading amplitude (pile head deflection less than 0.1 inch), Reese's p-y criteria give a very good prediction of the pile stiffness, even though the vibratory test was at a much faster loading rate. The good agreement is because the pore pressure effects would be small at such a low loading amplitude.

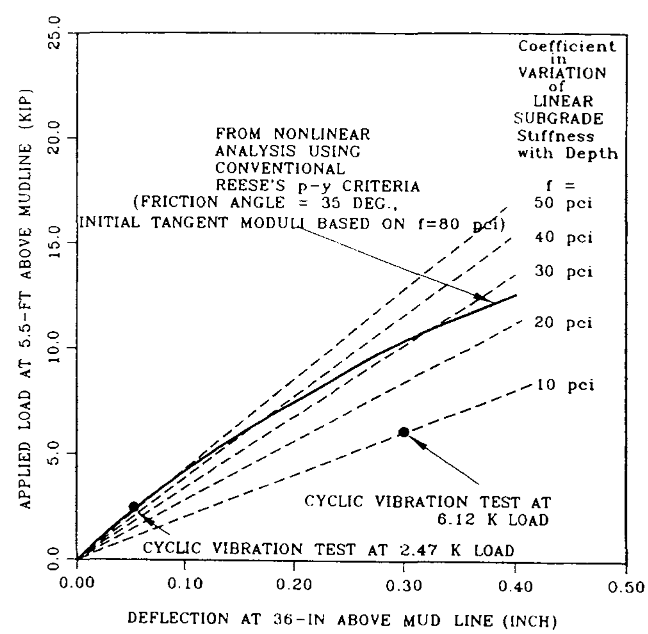


Figure 23 Hind-Cast Analyses of Full-Scale Lateral Pile Load Test (Lam, 1994)

However, at the higher load amplitude of 6.12 kip pile load, the measured pile stiffness during the vibratory test is lower than the prediction using Reese's criterion. In addition to the nonlinear p-y analysis, Figure 23 presents a series of solutions using a linear soil stiffness approach where the stiffness increases linearly with depth starting from a zero stiffness with depth. Solutions for f varying from 10 pci to 50 pci are presented in Figure 10. Furthermore, using the nonlinear solution as a basis, the equivalent secant pile stiffness at 0.3 inch would correspond to a f coefficient of 30 pci. When it is compared to the f value of 10 pci that best fits the vibratory test data, it can be observed that Reese's p-y criterion over predicts the faster rate vibratory test data by a factor of 3. Therefore, a multiplication factor or about 1/3 on the resistance values of conventional p-y criterion may be appropriate to account for the localized pore-pressure effects of saturated sands at the fast loading rates. The softer stiffness can be attributed to the undrained condition at the faster loading rate as compared to the drained or partially drained condition during slow rates from conventional hydraulic ram pile load tests.

As previously discussed, the above localized pore pressure effects do not capture the probable dominant free field liquefaction effects during earthquake loading, where high pore pressure buildup would be expected over a wider zone of soil mass independent of the pile size. A full-scale experiment to create such free field liquefaction effects is not feasible and centrifuge model tests are required. As part of an FHWA/NCEER research program, a series of centrifuge model pile load tests are being conducted at Rensselaer Polytechnic Institute (RPI) to address the problem of soil-pile interaction including freefield liquefaction effects.

Preliminary results have been presented by Liu and Dobry (1995). Figure 24 shows the configuration of the RPI centrifuge tests. A model pile (0.375 inches in diameter or 15 inches diameter in the prototype scale under a 40g acceleration) was embedded in a saturated sand deposit where the sand was prepared to a relative density of 60 percent. The sand box was placed on a shaking table and shaken to achieve various degrees of pore pressure ratio. The test pile was laterally loaded immediately at the end of shaking to observe the p-y response of the liquefied soils. Instrumentation included lateral load and deflection measurements, strain gages to measure pile moment at various depths and pore pressure measurements in the free field. Accelerometers were also installed within various points in the soil to monitor the acceleration response during shaking of the soil box. The soil samples were saturated with a deaired water-glycerin mixture to retard the pore pressure dissipation immediately following shaking. The dissipation rate was adjusted to 10 times slower than would be expected using pure water.

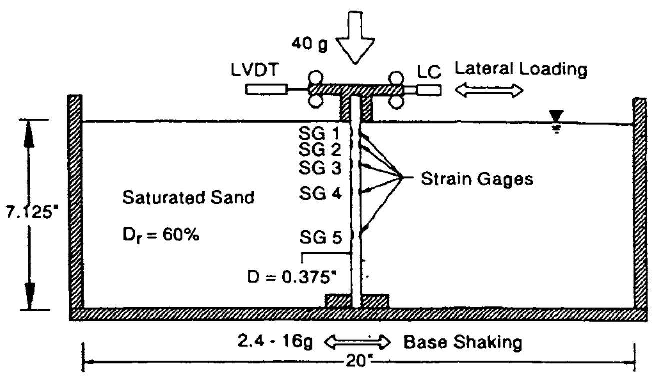


Figure 24 Centrifuge Model Configuration for Lateral Load Test (Liu and Dobry, 1995)

Tests conducted included:

- Calibration tests with no sand to observe the degree of rotational constraint at the pile head and at the pile tip for the pile test setup shown in Figure 24.
- Calibration tests to observe the p-y stiffness of a baseline soil condition where the freefield excess pore pressure amplitude is zero.
- Pile loading tests following various levels of freefield shaking amplitudes to observe the p-y response at various pore pressure ratios.

Figure 25 shows preliminary results for the pile head force measurements (at a pile head deflection of 2 inches) at the first quarter cycle of the pile load test immediately following the freefield shaking plotted against the pore pressure ratios measured at various depths in the free field soil. The calibration point of 53.7 kip was developed from a test (Calibration PS01 test) representing the upper bound pile head load for a zero pore pressure. The calibration of point of 10 kip was developed from a test representing the lower bound pile head load when the soil resistance is zero. The four pile head load values in between the bounding cases represent the data obtained from four series of centrifuge experiments where the shaking amplitudes were varied from 0.06g to 0.40g to achieve various levels of pore pressure ratio at the onset of the pile load test.

As shown in the figure the pore pressure ratios generally increase with depth. At the 0.40g shaking amplitude level, the entire soil mass has achieved a fully liquefied condition corresponding to a pore pressure ratio of 1. Based on hindcase analyses, we conclude that the pile head stiffness is most sensitive to the upper 5 feet of soil mass and therefore, the observed pile head stiffness would largely be a function of the pore pressure measurement at 2.8 feet. This is further confirmed by the trends in the three pile-head force versus pore pressure ratio curves as they are extrapolated to a zero pore pressure ratio (shown as dashed lines). It can be observed that the trend at the 2.8 ft. pore pressure measurement provides a reasonable extrapolation to the upperbound pile head force of 53.7 kip.

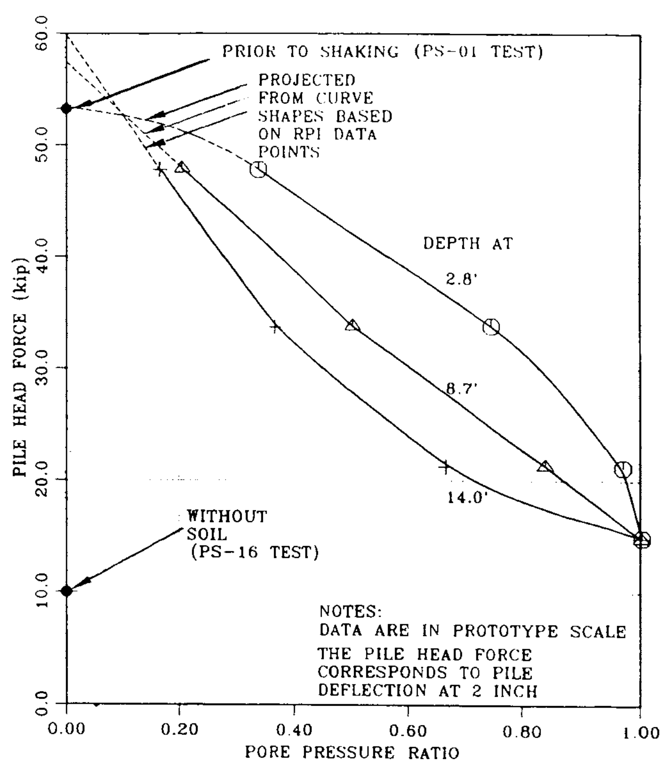


Figure 25 RPI Centrifuge Data-Pile Force vs. Pore Pressure Ratios (Lam, 1994)

Continuing analysis of test data will provide more insight as to the nature of stiffness degradation during free field pore pressure build up. The data will also provide insight as to the question of residual undrained strength following liquefaction. The availability of a small residual strength to p-y curves in liquefied soils, has a big impact on pile performance under lateral loading in liquefied soil.

It is interesting to note that the Winkler spring model (including pore pressure degradation effects) can also be adapted to study the problem of liquefaction induced lateral spread effects on pile foundations, as described by Myerson et. al. (1992).

AXIAL LOADING - SINGLE PILES

Whereas lateral loading of piles is often emphasized for earthquake design considerations, the rotational or rocking behavior of a pile group may have a significant influence on the seismic response of a structure, particularly in the case of bridge structures. Also, rotational compliance of pile group foundations for a moment frame building structure, could significantly influence column moments. Analyses show that the rotational stiffness of a pile group is generally dominated by the axial stiffness of individual piles. Hence, simplified procedures to evaluate pile head load-deflection characteristics in the axial direction are needed for seismic design. Earthquake induced axial loading of pile groups may also be of design importance in the analysis of the seismic rocking response of rigid shear walls for buildings when subjected to lateral loading.

Load Transfer Characteristics Under Axial Loading

Although elastic solutions exist for the pile head stiffness for piles embedded in linear elastic media (Poulos and Davis, (1980), Pender (1993)), the complexities of the non-linear load transfer mechanisms to the pile shaft and tip make the selection of an equivalent linear elastic modulus for the soil very difficult. As for the case of lateral loading, the use of the non-linear Winkler spring approach provides an alternate procedure which has been widely adopted in practice.

The various components of the axial pile load transfer problem are illustrated in Figure 26. The overall pile behavior depends on the axial pile stiffness (AE) and the load transfer characteristics (t-z curves) along the side of the pile and at the pile tip (tip q-z curve). The fundamental problem in an analysis of piles under axial loading relates to the uncertainties of the load transfer characteristics at the side and at the pile tip which in turn influence the pile head load-deflection behavior. Factors which need to be considered in developing the load transfer characteristics, include, (a) the side-friction capacity along the length of the pile (b) the ultimate resistance at the pile tip, and (c) the form of the load transfer-deflection curves associated with each of the above forms of soil resistance.

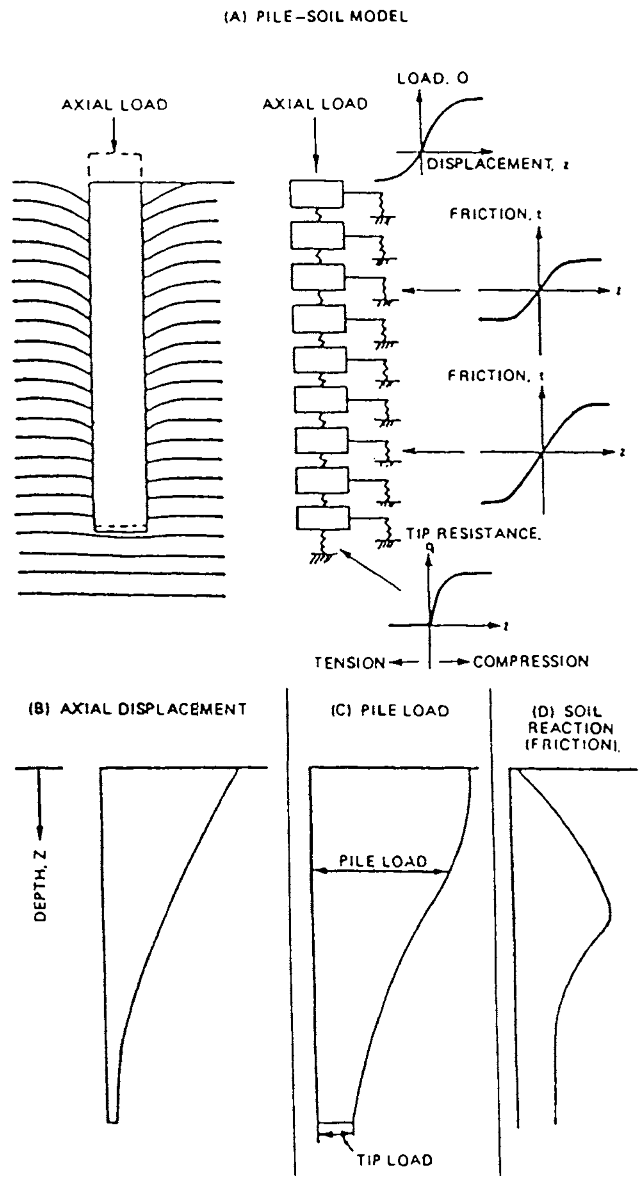


Figure 26 Schematic Representation of Axial Pile Loading (Matlock and Lam, 1980)

The ultimate capacity of a pile depends on numerous factors including (a) the soil conditions and pile type, (b) the geologic history, and (c) the pile installation methods. Numerous methods have been proposed to predict the axial capacity of piles and lead to widely varying capacity estimates, as documented in Finno (1989). Discussion of these methods are beyond the scope of this paper. However, incorporation of site-specific pile load test data has been perceived to be the most reliable method for pile capacity determination. In view of the above variation in viewpoints and the fact that the aspect of pile capacity is usually addressed in other design considerations (e.g., static design), no specific procedure is recommended here for the axial pile capacity.

In addition to the ultimate side friction and end-bearing capacity, some assumptions need to be made to develop the load transfer-displacement relationships (for both side friction and end bearing) to evaluate the overall pile behavior. The form of the load transfer-displacement relationship is again complex, and there is no uniform agreement on the subject. One empirical approach is for example, described by Heydinger (1989). A bi-linear modeling approach for t-z and q-z curves is described by Trochanis et. al. (1987). However the load transfer-displacement relationships described below (based on Vijayvergiya, 1977) are relatively simple and have been used in practice:

Side Friction: $f = f_{max} \cdot \; 2\sqrt{z/z_c} - z/z_c)$ (5)

Where:

f = unit friction mobilized along a pile segment at displacement, z,

f_{max} = maximum unit friction, and

z_c = the critical movement of the pile segment at which f_{max} is fully mobilized.

A z_c value of 0.2 is recommended for all soil types.

End Bearing: $q = (\frac{z}{z_c})^{1/3} q_{max}$ (6)

where:

q_{max} = maximum tip resistance

q = tip resistance mobilized at any value of $z < z_c$, and

z_c = critical displacement corresponding to q_{max}.

A z_c value of 0.05 of the pile diameter is recommended.

Analysis Methods for Axial Loading on Piles

A computer approach can again provide the most convenient means of solving axial pile behavior. Many of the well established computer programs such as BMCOL 76 and, PILSET (Olsen, 1985) allow for prescription of the t-z curves at various depths along the length of the pile (e.g., at the boundaries of each soil layer) and will automatically perform interpolations to develop support curves at all the pile stations. The t-z curves for side friction are usually assumed symmetrical and the q-z curve at the pile tip nonsymmetric (see figure 26). An example of the use of the program PILSET to compute the axial pile lead load versus deflection curve for a 78m long 915mm O.D. steel pipe pile is shown in Figure 27 (after Gohl, 1993).

Uncertainty in axial soil-pile interaction analysis relates largely to uncertainties in soil parameters including the

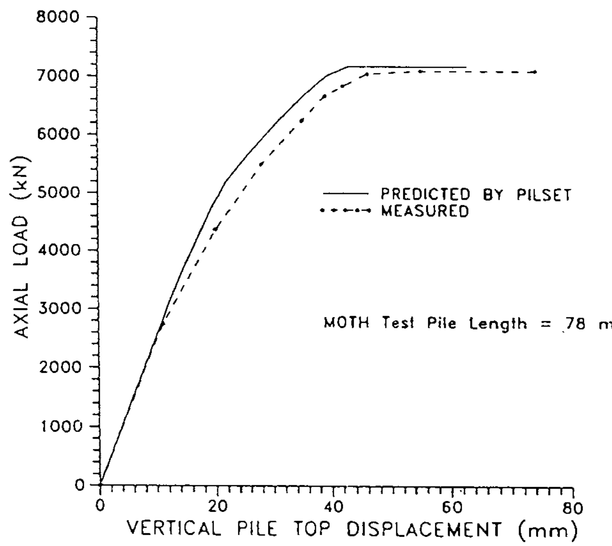


Figure 27 Computed versus Measured Axial Pile Response using the Program PILSET (After Gohl, 1993)

ultimate pile capacity (skin-friction and end-bearing) and load-displacement relationships. Computer solutions can be used for a rigorous nonlinear solution. However, an approximate nonlinear graphical solution method has been presented by Lam and Martin (1984, 1986), and is described below. The procedure is schematically shown in Figure 28 (for a 70ft. long, 1ft. diameter pipe pile embedded in sand, ϕ = 30°) and involves the following steps:

(1) Soil Load-Displacement Relationships. Side-friction and end-bearing load-displacement curves are constructed for a given pile capacity scenario (accumulated skin-friction and ultimate tip resistance). Various forms of curve shape recommended by researchers can be used to develop the above load-displacement curves. Vijayvergiya's recommendation (1977) was adopted (for simplicity) in the example shown in Figure 28 (skin friction is assumed mobilized at a displacement of 0.2 inches, and end bearing at a displacement of 0.5 times the pile diameter).

(2) Rigid Pile Solution. Using the above load-displacement curves, the rigid pile solution can be developed by summation of the side-friction and the end-bearing resistance values at each displacement along the load-displacement curves.

(3) Flexible Pile Solution. From the rigid pile solution, the flexible pile solution cab be developed by adding an additional component of displacement at each load level Q to reflect the pile compliance. For the most flexible pile scenario, corresponding to a uniform thrust distribution along the pile shaft, the pile compliance is given by:

Pile Compliance $\delta_c = QL/AE$ (7)

Where L is the pile length; A is the cross-sectional area, and E is the Young's modulus of the pile.

(4) Intermediate Pile Stiffness Solution. The "correct" solution, as indicated by the computer solution in Figure 28, is bounded by the above rigid pile and flexible pile solutions. In most cases, a good approximation can be developed by averaging the load-displacement curves for the rigid and the flexible pile solutions. The above graphical method can be used to solve for the load-displacement curve for any combination of pile/soil situation (end-bearing and friction piles) as well as any pile type or pile material.

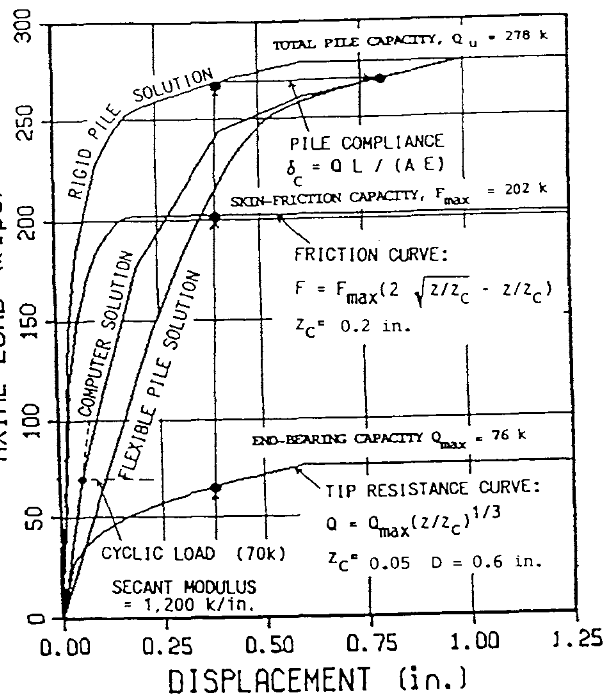


Figure 28 Graphical Solution for Axial Pile Stiffness (Lam et. al. 1991)

Under earthquake conditions, some magnitude of cyclic axial load will be superimposed on a static bias load (e.g., the static dead weight). Figure 29 illustrates the various factors which come into the picture due to a static bias loading. As shown in Figure 29, in a normal design range, where the maximum load level (from superimposing the cyclic load on the static bias) does not exceed the pile capacity (for both the peak compressive or tensile load), the static dead weight can be neglected in solving for the secant stiffness of the pile. The magnitude of cyclic loading along with the backbone load-displacement curve can be used to develop the secant stiffness of the pile at the various load levels. However, the load-displacement behavior of the pile will be more complex when the pile capacity (compressive or tensile) is exceeded. In general, permanent displacement of the pile will occur for the above condition.

A described by Gohl (1993), as an even simpler approximation, pile head stiffness values under normal loading (not exceeding the capacity) may be expressed as some multiple α of AE/L with the constant α depending on the proportions of shaft and end bearing resistance mobilized. For example, a lower bound stiffness AE/L would be appropriate for an end bearing pile on rock with negligible shaft friction. Values of α closer to 2 would be reasonable for friction piles with negligible end tip resistance.

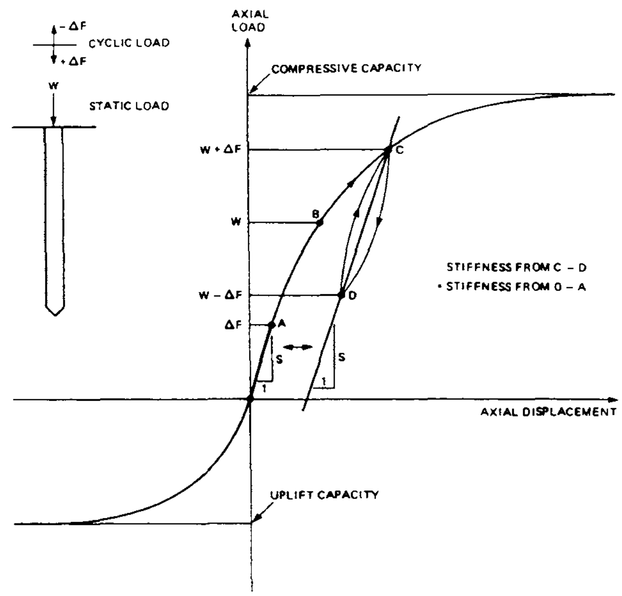


Figure 29 Load-Displacement Characteristics Under Axial Loading (Lam and Martin, 1986)

PILE GROUP ANALYSES

The previous discussion has focused on the behavior of single piles under lateral and axial loading. In practice, pile foundation systems for bridge and building structures are most often found in the form of a pile group such as that idealized in Figure 30 . In a dynamic response analysis of a structure, the pile group foundation system for inertial interaction models, can be conveniently incorporated into the structural model by either (a) an uncoupled base spring model or (b) a fully coupled foundation stiffness matrix model. The latter model is the most general and rigorous approach, albeit it requires the determination of the stiffness coefficients in a generalized 6 x 6 stiffness matrix for a pile, as shown in Figure 31. This will allow a 6 x 6 stiffness matrix for the pile group to be assembled, representing the lateral and rocking stiffness for the two horizontal axes (including cross coupling terms), the vertical stiffness and the torsional stiffness.

The embedded pile cap represents an additional but very

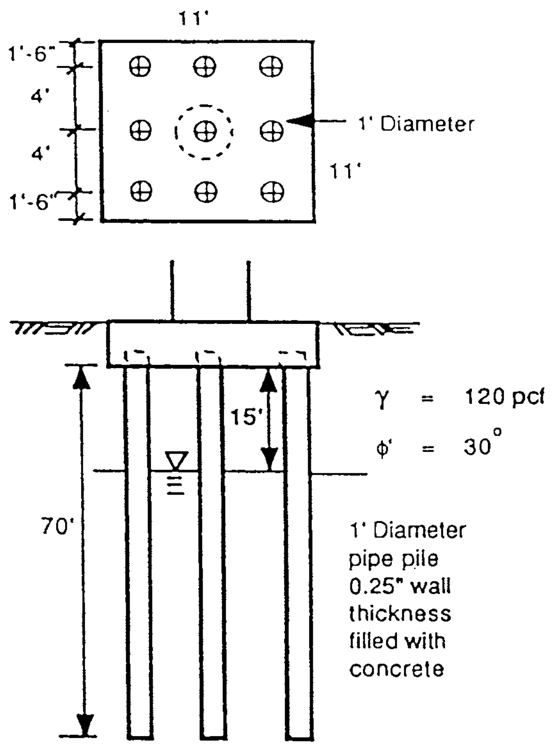
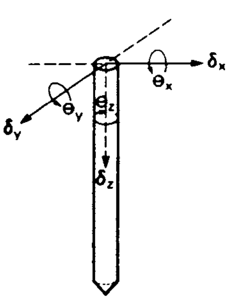


Figure 30 Idealized Pile Group in Sand



$$
\begin{array}{cccccc}
\delta_x & \delta_y & \delta_z & \theta_x & \theta_y & \theta_z
\end{array}
$$

$$
\begin{bmatrix}
K_{11} & 0 & 0 & 0 & -K_{15} & 0 \\
0 & K_{22} & 0 & K_{24} & 0 & 0 \\
0 & 0 & K_{33} & 0 & 0 & 0 \\
0 & K_{42} & 0 & K_{44} & 0 & 0 \\
-K_{51} & 0 & 0 & 0 & K_{55} & 0 \\
0 & 0 & 0 & 0 & 0 & K_{66}
\end{bmatrix}
$$

FORCE VECTOR FOR $\delta_x = 1.0$

Figure 31 Full Fixed Head Pile Stiffness Matrix

important complexity. In general, the pile cap is uncoupled from the piles to determine stiffness coefficients, which are then added to the pile stiffness matrix. Further complexities arise in the presence of battered piles. The discussion below provides a brief overview of approaches used in practice to address the pile group stiffness matrix, the influence of the pile cap, and effects of battered piles. Finally, design issues related to the moment-rotation capacity of pile groups are discussed.

Pile Group Stiffness Matrix

To determine the pile group stiffness matrix, the pile head stiffness coefficient for a single pile must first be determined. To illustrate the procedure, the idealized pile group shown in Figure 30 is used, assuming a fixed pile head condition. The stiffness coefficients under lateral and axial loading for a single pile were determined using the linearized procedures previously described, as illustrated in Figures 16 - 18 and Figure 28. The resulting stiffness coefficients are shown tabulated in Table 1. Note that the torsional stiffness of a single pile can usually be assumed zero.

The single pile stiffness matrix in Table 1 can be used in the next step to develop the pile group stiffness matrix. For a vertical pile group such as that shown in Figure 30, the form of the stiffness matrix will be identical to the individual pile (as shown in Figure 31). Also the stiffness summation procedure is relatively straight forward. For battered-pile systems, computer solutions are recommended. A PILECAP computer program that can be used to conduct the summation of individual pile-head stiffnesses for an overall pile group stiffness matrix has been documented in an FHWA report by Lam and Martin (1986). The program can also be used to distribute the overall foundation load to individual piles.

Table 1. Pile Stiffness Solution

Stiffness Coefficient	Single Pile	Pile Group
Lateral Translation $k_{11} = k_{22}$, (kip/in)	42	9 x 42 = 378
Vertical Translation k_{33}, (kip/in)	1,200	9 x 1,200 = 10,800
Rocking Rotation $k_{44}=k_{55}$ (in-kip/rad)	193,000	$NR_{\eta} + \Sigma^N K_{\delta n} S_n^2$ = 1.74 x 10^6 + 1.66 x 10^7 = 1.83 x 10^7
Torsional Rotation K_{66}, (in-kip/rad)	0	$4 \times 42 \times 48^2 + 4 \times 42 \times (48^2+48^2)$ = 1.16 x 10^6
Cross-Coupling $k_{15}=k_{51}=-k_{24}=-k_{42}$, (kip)	-2,250	9 x -2,250 = -20,250

For a vertical pile group, the stiffness for the translational displacement terms (the two horizontal and the vertical displacement terms) and the cross-coupling terms can be obtained by merely multiplying the corresponding stiffness components of the individual pile by the number of piles. However, the rotational stiffness terms (the two rocking and the torsional rotations) require consideration of an additional stiffness component. In addition to individual pile-head bending moments at each pile head, a unit rotation at the pile cap will introduce displacements and corresponding forces at each pile head (e.g. vertical forces for rocking rotation and lateral pile forces for torsional rotation). These pile-head forces will work together among the piles and will result in an additional moment reaction on the overall pile group. The following equation can be used to develop the rotational stiffness terms of a pile group.

$$R_g = N R_p + \sum_{n=1}^{N} K_{\delta n} S_n^2$$

where R_g and R_p are the Rotational Stiffness of the pile group and an individual pile, respectively. N is the No. of piles in the pile group. $K_{\delta n}$ is the appropriate stiffness coefficient of an individual pile (vertical or lateral) and S_n is the spacing between the n^{th} pile and the point of loading (center of the pile group).

The subscript $_n$ denotes the pile no. Summation is conducted for all the piles in the pile group in the above equation.

Using the described procedure, the pile-group stiffness of the overall pile group system shown in Figure 30 is developed and presented in Table 1. It can be observed that the rocking rotational stiffness coefficients of the pile group are dominated by the axial stiffness of the piles.

The above procedure for a pile group does not account for the "group effects" which relate to the influence of the adjacent piles in affecting the soil support characteristics. There exists a wide range of opinions among geotechnical engineers on the significance of the "group effects". The importance of "group effects" would depend on many factors including the configuration of the pile group (number of piles, spacing, direction of loading in relation to the group configuration), soil types and pile installation methods.

In the case of lateral loading, a p-y multiplier or interaction coefficient approach is often used to soften p-y curves to reflect shadowing effects at close pile spacing. The interaction factors defined by Poulos and Davis (1980) based on elastic continuum solutions over predicted the group effect where the effects of nonlinearity localize deformations near the pile shaft. Where nonlinear behavior occurs, pile spacings of less than about 3 - 5 pile diameters (depending on the above factors) are generally necessary before the effects of pile interaction become significant inpractical terms.. Experimental studies by Brown et. al. (1988) and McVay et. al. (1995) provide useful data on p-y multipliers for close pile spacing in sands. The latter studies indicate about a 20% effect for pile spacing of 3 diameters.

Influence of Pile Cap

In many cases, the stiffness of a buried pile cap, particularly in the lateral direction, can have a significant if not dominant influence on the stiffness of the pile group as a whole. In the case of lateral stiffness, contributions arise from passive resistance on the vertical face, and tractional resistances from the sides and possibly the base of the pile cap , as shown in Figure 32. For practical purposes, a pile cap is generally considered as an uncoupled footing to determine stiffness contributions to the group stiffness matrix. Elastic solutions are often used to determine equivalent linear stiffness coefficients in translational, vertical and rocking deformation modes, using published solutions where soil is treated as an elastic medium. However, as for pile loading problems, it is difficult to establish appropriate elastic constants to use because of non-linear soil response.

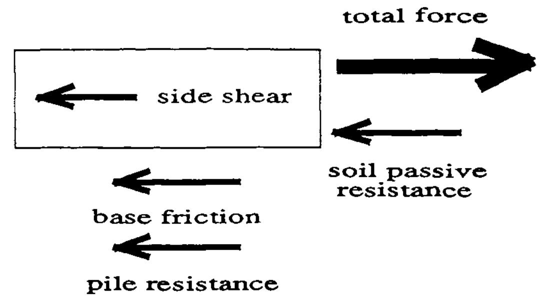


Figure 32 Lateral Stiffness and Capacity - Effect of Pile Cap

To clarify the nature of the non-linear load-deformation characteristics of pile caps under lateral loading including passive capacity, it is clear that more experimental data is needed either from field load tests or centrifuge tests. Abcarius (1991) presents lateral load tests results on pile group foundations excavated to expose the footings of the Cypress Viaduct damaged during the Loma Prieta earthquake. Two adjacent footings were jacked against each other as shown in Figure 33, which also shows results interpreted by Lam and Martin (1993) as load per pile versus deflection. The nominal design stiffness of piles was 40 kip/inch and the nominal lateral capacity , 40 kips.

The above tests did not include contributions from lateral passive pressures. Ideally, in conducting field load tests, progressive footing excavation should be considered so resistance components could be isolated by sequential testing. Such a process is presently being utilized in model pile cap tests being conducted in the Rensselar Polytechnic Institute centrifuge. In these tests model aluminum pile caps (approximately 4 ft. x 4 ft. x 2' -6" deep in prototype scale) are embedded to a depth a 3 ft. in a dense sand. Representative test results, (Dobry (1995), are shown in Figure 34, where cyclic lead amplitudes were progressively increased to levels where passive capacity was reached. Similar tests are being conducted to progressively remove side wall effects to separate stiffness contributions. A final test sequence will be conducted to include support by calibrated piles.

Battered Piles

Battered pile group systems are often used to provide

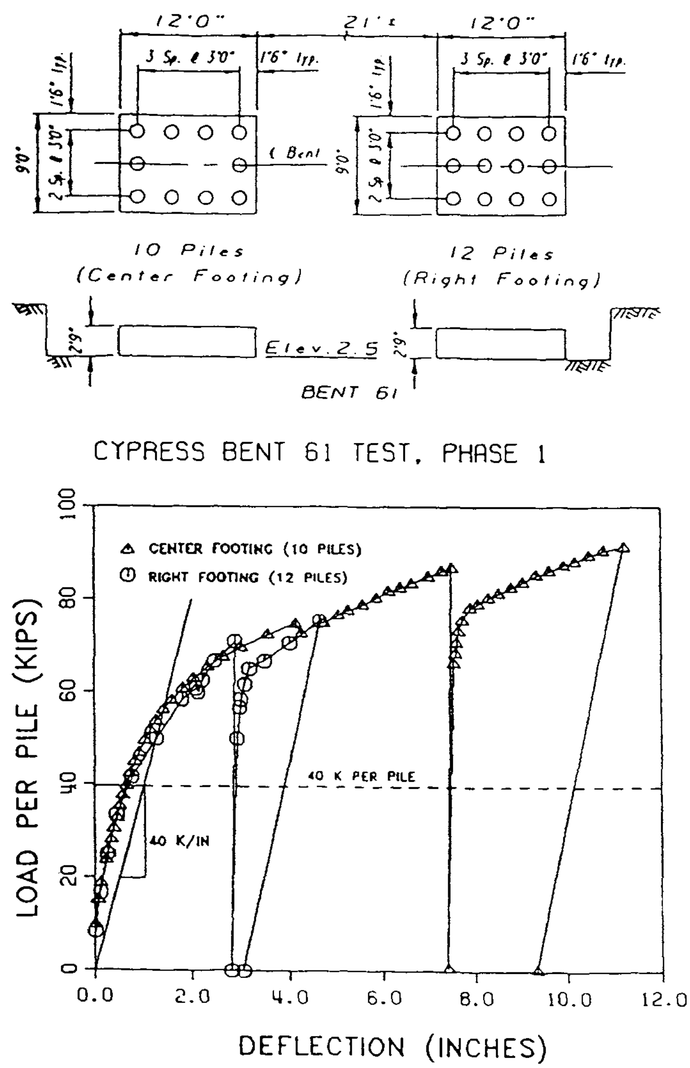


Figure 33 Caltrans Footing Test Data for Cypress Viaduct Interpretation of Bent 61 Test Data (Lam and Martin, 1992)

additional lateral capacity or stiffness compared to vertical pile groups, particularly in the case of soft soil conditions. However, in the design of such piles, care must be taken to correctly analyze the group action. The often used simple assumption that battered piles resist lateral load through axial loading only, neglects the importance of the high bending moments that can be induced by fixity into a pile cap.

To illustrate the comparative behavior of a vertical versus a battered pile group, the simple case of the group configuration show in Figure 35 is considered. The one foot diameter piles are embedded in uniform sand ($\phi = 35°$), and rigidly connected to the pile cap. p-y curves were computed using the API (1975) recommendations, and t-z curved constructed using the recommendations by Vijayvergiya (1977). Two series of analyses are conducted: 1.) monotonic increases of the lateral force and 2.) monotonic increases of the moment lead. In each series of analyses, a vertical load

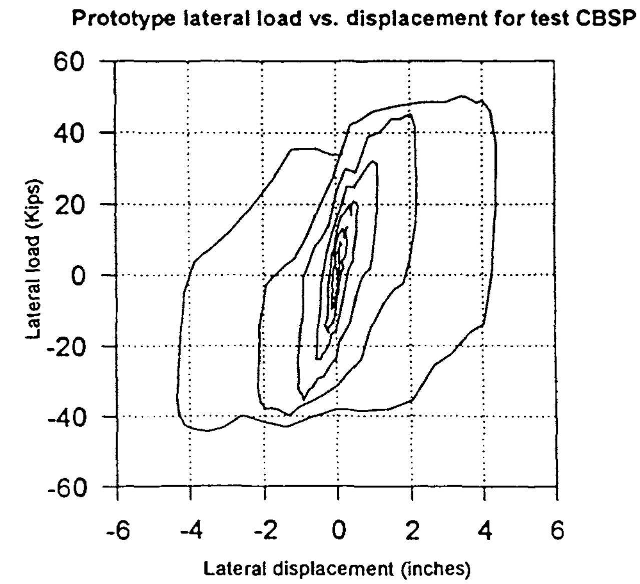


Figure 34 Centrifuge Test Results - Lateral Loading of Pile Cap (Dobry, 1995)

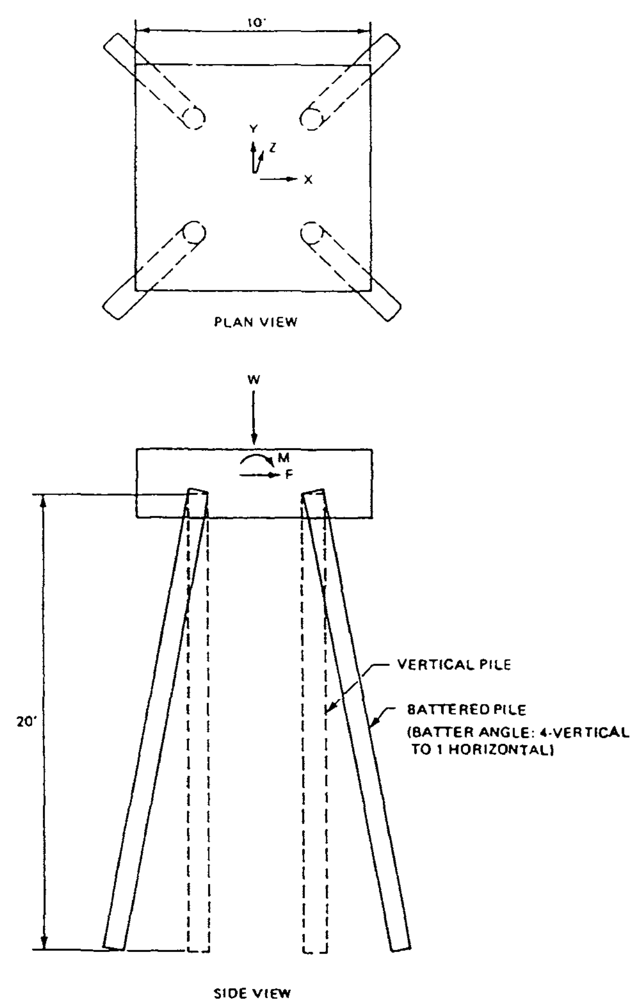


Figure 35 Battered Pile Group Configuration

of 138 tons was used to simulate the vertical load, which corresponded to 75 percent of the ultimate axial pile capacity. This is higher than normal practice, but was used to illustrate potential impacts of moment loading on axial capacity.

The computer programs used to solve the problem are described by Lam and Martin (1984). The two series of pile group solutions are presented in Figures 36 and 37 for the pile group under a lateral force and moment loading, respectively. As shown in Figure 36, the lateral load versus pile group displacement for the battered pile group is 30 percent stiffer than the corresponding vertical pile group curve. The axial compliance of the pile in this example problem is largely due to the deformation of the soil. For some other situations, for example, if the pile tip is embedded into bedrock, the axial stiffness of the pile can be dramatically increased, and the lateral stiffness of a pile group can be significantly stiffened by the battered effect.

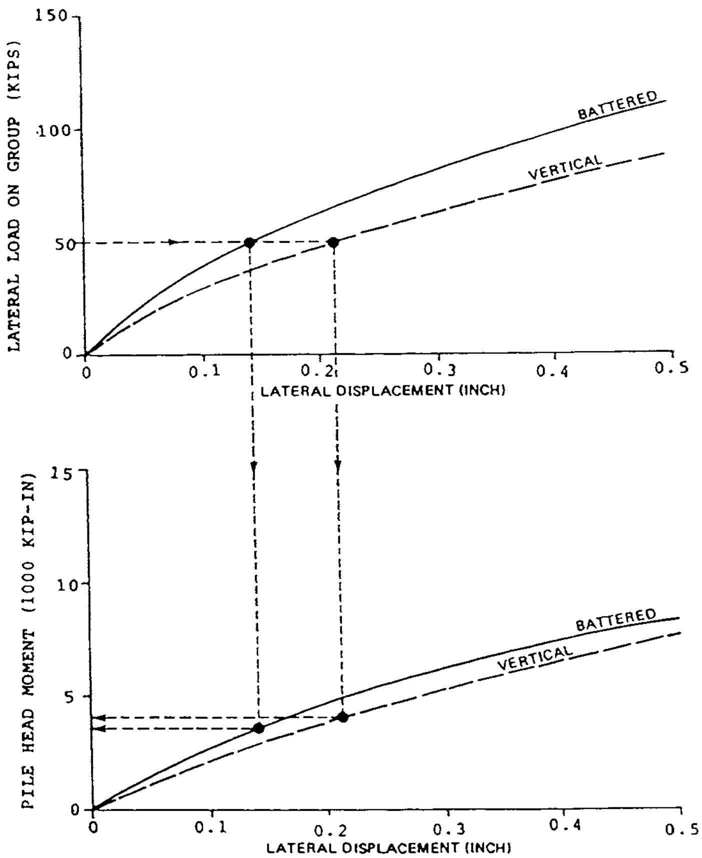


Figure 36 Comparison of Vertical and Battered Solutions from Lateral Loading (Lam and Martin, 1984)

The bending moment on the pile-head induced by the lateral loading, differs only slightly among the four piles, and it is also plotted against the corresponding lateral displacement of the pile cap in Figure 36. In the design of a battered pile system, the shear force and moment at the pile cap is often assumed to be reacted primarily by the axial forces in battered piles due to an assumed axial pile stiffness. As a result the piles may not be designed for bending. This is clearly erroneous for non end bearing piles, where for a battered pile system, significant lateral load is taken out in bending due to lower axial stiffnesses. For a given lateral load (say 50 kips) the resulting bending moment for the battered pile system is indeed lower than the vertical pile group. However, a significant bending moment remains on the pile member. The above results suggests that bending moment in a battered pile system should not be neglected. Even if the pile-heads are designed for free head condition at the pile cap, significant bending moment can potentially occur at a deeper depth.

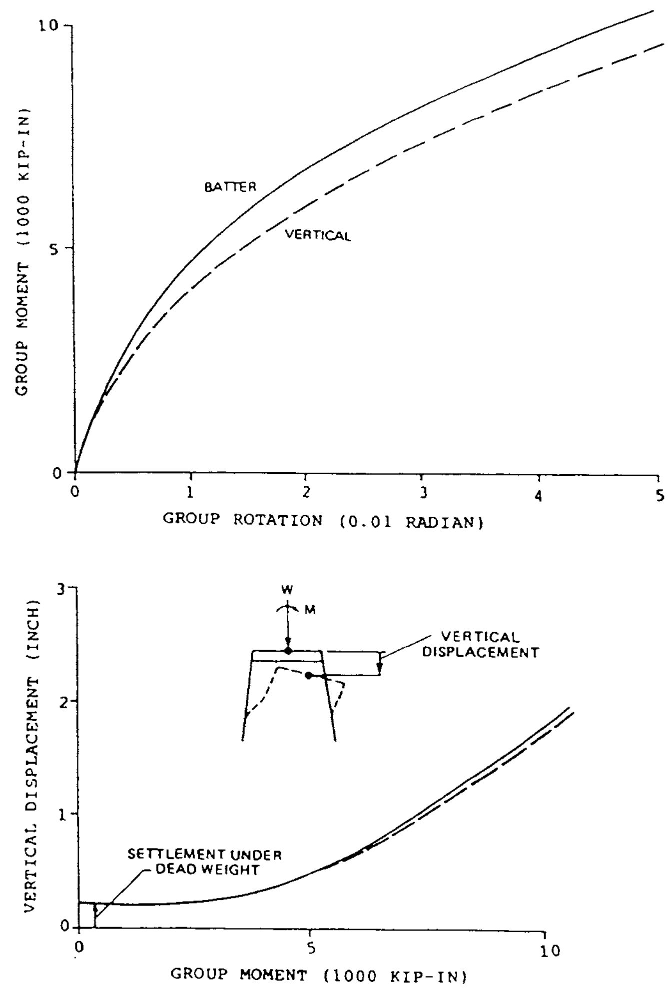


Figure 37 Comparison of Vertical and Battered Solutions from Moment Loading (Lam and Martin, 1984)

The pile group moment versus pile group vertical displacement solutions are shown in Figure 37 for the moment loading. Under the combined dead weight and the moment loading on the pile group, a significant vertical displacement may occur at high moments as shown. The vertical displacement of the pile group is associated with the plunging failure of the compression pile, where the combined axial loading from dead weight and moment mobilizes the full axial capacity. A significant amount of displacement could potentially be irrecoverable. Hence normal seismic design should allow for axial loading from seismically induced moment (that is, axial capacity not to be exceeded by dead plus seismic load) unless permanent displacements of the pile group can be tolerated. This is discussed further below.

Moment Rotation Capacity

The lateral stiffness of pile groups is often the focus of attention in seismic design of bridge foundations. However, experience gained from recent research has provided evidence that the moment-rotational characteristics of a pile group can have a more dominating effect on the response of the overall structure as compared to the lateral stiffness issue.

In a high seismicity region, the foundation design (e.g. the number of piles and the size of the pile cap or pile footings) is often controlled by the earthquake load case. A key element in the design relates to the provision of adequate foundation capacity to resist the base overturning moment arising form the inertial forces of the superstructure. In high seismic areas, the number of piles (which is the most costly item in a foundation) is often governed by the earthquake moment load case rather than other service load cases, even though a much lower factor of safety (i.e. unity) is usually adopted for seismic design. Therefore, in terms of both the overall bridge behavior and economics, the moment capacity of pile footings is the most important factor in foundation design.

The moment-rotational characteristics and the capacity of a pile footing depends on the following factors:

- The configuration (number of piles and spatial dimension) of the pile footing.

- The capacity of each pile for both compression and uplift loading.

To illustrate the above concern, Lam (1994) presents an example problem involving a typical pile footing as shown in Figure 38. The analyses presented assume a rigid pile cap for the footing, and are quasi-static analyses. The load-displacement curves for each individual pile in the pile group are shown in Figure 39. The pile is modeled as an elastic beam-column and nonlinear axial soil springs are distributed along the pile to represent the soil resistance in both compression and uplift. It can be seen from the figure that the ultimate soil capacities of the pile for compression and tension are 180 and 90 kips per pile, respectively, if the connection details and the pile member are adequate to enforce the failure to take place in the soil. The pile has been assumed to be a 50 ft. long, 12 inch concrete pile driven into uniform medium sand which has a design load capacity of 45 tons per pile. The adopted ultimate capacity values (i.e. 180 kip compression and 90 kips uplift) are the default values commonly assumed by Caltrans in seismic retrofit projects for the 45-ton class pile. In the example it is assumed that the footing has been designed for a static factor of safety of 2, or the piles are loaded to half of the ultimate compression capacity prior to the earthquake loading condition.

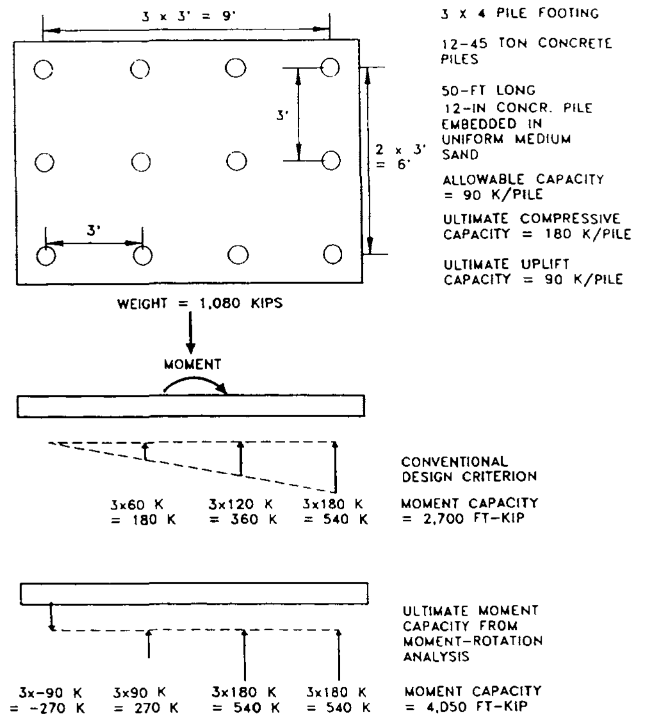


Figure 38 Pile Footing Configuration for Moment Rotation Study (Lam, 1994)

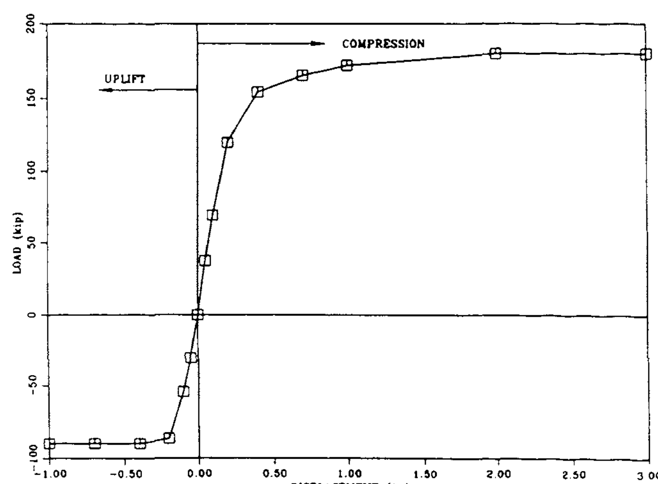


Figure 39 Axial Load-Displacement curve for a Single Pile (Lam, 1994)

The lower part of Figure 38 presents various capacity criteria for the pile footing. Under conventional practice, the moment capacity of the pile footing would be 2,700 ft.-kip. This capacity arises from assuming a linear distribution in pile reaction across the pile footing. The moment capacity of 2,700 ft.-kip is limited by the ultimate compressive capacity value of the most heavily load pile (180 kip per pile) while maintaining vertical equilibrium of the overall pile group (i.e. static load of 1,080 kips). The bottom of Figure 38 presents the moment capacity that can be achieved from a nonlinear moment-rotation analysis of the pile footing, where as the moment load increases above the conventional capacity, nonlinear load-displacement characteristics of the pile are simulated to allow additional load be distributed to the other less loaded piles in the pile group. As shown in Figure 38, a maximum ultimate capacity of 4,050 ft.-kip (1.5 the conventional capacity) can potentially be achieved by virtual of such nonlinear analysis.

Figure 40 presents the cyclic moment-rotation solutions associated with the footing example problem discussed above. The dotted line in the moment-rotation plot defines the monotonic loading path of the moment-rotation relation often referred to as the backbone curve. Solutions for two uniform cyclic moment loads are presented: 1.) a lower cyclic moment level of 2,700 ft.-kip corresponding to the conventional design capacity, and 2.) a higher cyclic moment load of 4,000 ft.-kip. As shown in Figure 40, at the lower cyclic moment of 2,700 ft.-kip, the moment-rotation characteristic is quite linear, and both the moment-rotation characteristics and settlement will equilibrate to the final value very quickly within a few cycles of loading. However, at the higher cyclic moment load of 4,000 ft.-kip, progressive settlement of the footing can occur and within about four cycles of loading, the footing can settle almost 5 inches. The moment-rotation relationship also indicates that some level of permanent rotation of the footing will likely occur even if the load is symmetric between positive and negative cyclic moment. The potential for the permanent rotation is associated with the change in the state of stress in the soil from a virgin (unstressed) condition to the equilibrated state after cyclic loading, unloading and reloading. A similar analysis using a static factor of safety of 3 (instead of 2) corresponding to a dead load of 720 kips, resulted in a ultimate moment capacity of 1.3 times the conventional capacity, and a reduced settlement of about 0.25 inches under loading cycles at the increased ultimate capacity level.

Considering the inherent conservatism in pile capacity determinations (especially for compressive loading), most existing pile footings probably have an inherent static factor of safety for dead load of over 3. Hence, it can be speculated that the potential for significant settlement or rotation of a pile footing would not be too high, except for poor soil sites where cyclic degradation of soil strengths can be significant. Typically, the most likely cause of foundation failure would be some form of permanent rotation of the pile group if the size of the footing and the number of piles are inadequate. Therefore, it is of importance to have a better appreciation of the magnitude of foundation rotation that is tolerable by the pile supported structure, particularly for retrofit seismic design, where unnecessary conservation can be expensive.

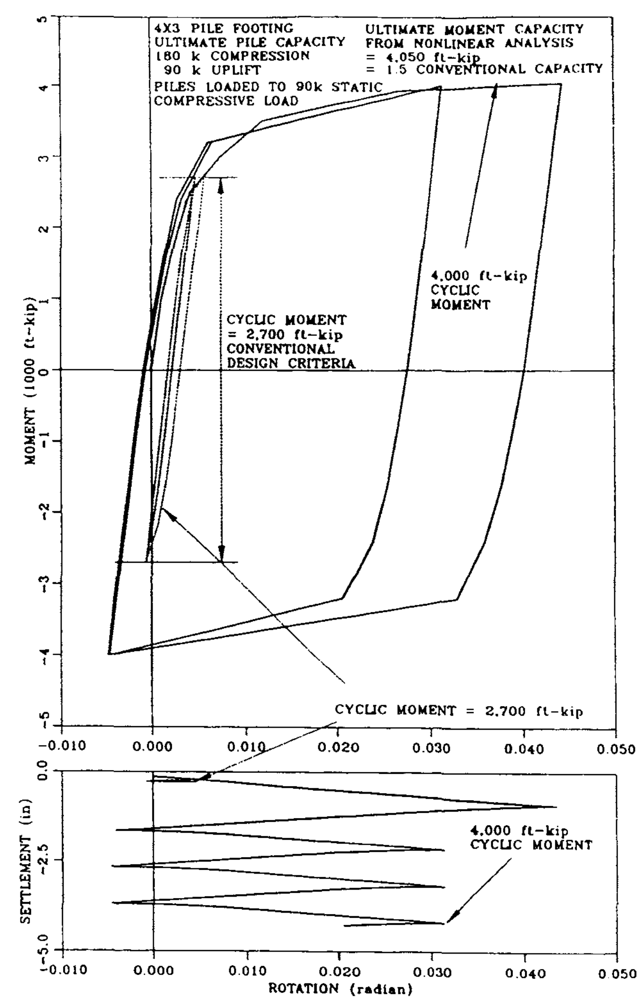


Figure 40 Cyclic Moment-Rotation and Settlement-Rotation Solutions (Lam, 1994)

CONCLUDING REMARKS

Although the paper has focused on simplified design approaches to the soil-pile-structure interaction problem, considering primarily a Winkler spring approach to inertial interaction, there is a continued need for developing an improved understanding of the mechanics of non-linear fully coupled behavior under earthquake loading. Perhaps this can be best achieved through carefully planned and designed experiments, involving the combined strengths of non-linear numerical analyses and centrifuge experiments.

A number of design and analysis issues have been identified and discussed in the paper, which have significant influence on design analyses and practice. These include:

- Effects of pile installation methods
- Effects of pile fixity
- Effects of pile stiffness (intact versus cracked)
- The effects and role of the pile cap
- The effects of free field and localized liquefaction
- Moment rotation capacity

There is clearly a need for practical research to address these issues, to provide improved seismic design guidelines.

Finally, there is also a clear need for improved communication between the geotechnical engineer responsible for providing guidance on soil parameters and pile stiffness characteristics for design, and the structural engineer. We as geotechnical engineers, need to develop an improved appreciation of structural design issues and the constraints provided by structural analysis methods. This will provide the means for improving and optimizing simplified design methods which we recommend to the structural engineer.

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