Modeling Bridge Foundations for Seismic Design and Retrofitting

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Past experience and recent research indicates that proper modeling of abutments and foundations is very important in the evaluation of dynamic response of an overall bridge structure. This paper presents simplified procedures with accompanying design charts for the development of stiffness coefficients for abutments, piles and spread footing foundations for highway bridges. The presented procedures have been calibrated to design practice adopted by bridge engineers. Several examples are presented in the paper to highlight various sensitivity issues in abutment and foundation design.

INTRODUCTION

Highway bridges are known to be highly susceptible to damage under earthquake loading. One major reason for the poor performance relates to the complexities of the bridge structural and substructural systems as compared to other structures. Some of these complexities are listed below:

o There are wide variations in structural types and configurations (bridge decks, bents and abutments). Furthermore, the need for expansion joints to accommodate temperature effects increases the potential for collapse and introduces significant complexities in design.

o Variations in soil conditions are common along the length of a highway bridge which could lead to different types of abutments and foundation systems. The wide range of combinations of structures, abutments, foundations and connections introduces many unique design problems for bridge engineers.

o Bridges are supported on multiple support points that are spaced relatively wide apart. As a result, variations in ground motion (magnitude and phase shift) could result in differences in force excitation levels and differential movements at support points.

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An excellent literature survey (Iwasaki et al., 1973) chronicles earthquake damage to bridges up until 1971. This report concluded that foundation behavior played a major role on performance of highway bridges during past earthquakes. Extracts from the conclusion section of the report are quoted below to illustrate the significance of foundation systems in seismic performance of bridges:

o "Seismic damage, particularly to low bridges, are most commonly caused by foundation failures resulting from excessive ground deformation and/or loss of stability and bearing capacity of the foundation soils......"

o "Backfills exert large forces on abutments which can at certain times be in-phase with the seismic inertia forces developed in the superstructures. These forces in combination may cause severe failures, often of a brittle nature, in the substructures."

o "To minimize damage, bridge structures should be designed with proper recognition of the stability and bearing capacities of foundation soils, force-deformation and energy absorption characteristics of substructure, superstructure and connecting elements, the dynamic nature of structural response, and the dynamic characteristics of all forces acting on the complete soil-structure system."

Poor soil conditions (soft soil and a high water table) contributed to most of the structural and substructural damage during many of the past earthquakes, often of a severe catastrophic nature. Sites that have sustained heavy damage during past earthquakes due to ground motion amplification, slope failure or liquefaction would be likely candidates for damage in future earthquakes. Therefore, existing earth-science information (seismologic, geologic, geotechnical, hydrological) should be used in regional earthquake preparedness and planning programs as well as in site specific designs.

Geotechnical considerations for highway bridges can be divided into two categories:

- Ground Stability. Site response, liquefaction potential, embankment slope stability and assessments of the magnitude of ground deformations (including cyclic and permanent deformations of the free-field site soil) and the evaluation of the nature and the magnitude of foundation movements and their implications on structural loading, are critical elements in foundation design of highway bridges.

- Substructures and Abutment Models. Evaluation of the fundamental periods and mode shapes of the bridge structure is needed for the determination of the seismic force level for design. For some bridges, especially the shorter and low bridges, the effects of foundation stiffness could significantly affect the overall dynamic response characteristics. Therefore, evaluation of foundation stiffness is an important part of the overall bridge structure model.

This paper focuses on modelling procedures for estimating stiffnesses of abutments, spread footings and pile footings (pile groups) for dynamic response analysis of an overall bridge.

ABUTMENTS

For many highway bridges, the abutments attract a large portion of the seismic force, particularly in the longitudinal direction. Therefore, the stiffness of the backfill at the abutments must be considered. However, further research is needed to improve our understanding and to develop improved design procedures related to the following aspects:

- Stiffness of abutment systems in both the longitudinal and transverse loading directions.

- The magnitude and distribution of soil pressure on end and wing walls.

- The relative significance in the induced soil pressure from inertia forces of the bridge deck versus the inertia forces of the soil mass acting on the abutment walls and the appropriate procedures for selection of design soil pressure to account for the interplay of the two loading mechanisms.

Due to the wide range of combinations in connection details (bridge-abutment and abutment-foundation), many of the above issues remain to be resolved and currently, a wide variation in design practice of abutments prevails.

Some guidance is currently provided by the Caltrans Bridge Design Aids Section 14-"Dynamic Model Assumptions and Adjustments" (Caltrans) and the AASHTO Guide Specifications for Seismic Design of Highway Bridges" (AASHTO, 1983). Both documents recognize the highly nonlinear behavior in abutments due to failure of the backfills and from structural nonlinearity at expansion joints. The load-displacement characteristics for a typical monolithic and a seat-type abutment are shown in Figure 1.

An iterative design procedure as shown in Figure 2 and described in the following steps is needed to account for the nonlinear behavior of abutment systems in a linear dynamic response analysis.

(1) Assume an Initial Abutment Stiffness. This stiffness should be compatible with the configuration of the structure and connection details and the assumed peak displacement of the bridge deck.

For longitudinal loading, it should be recognized that in the course of an earthquake, the stiffness of only one abutment would be mobilized (i.e., soil resistance is mobilized when the structure is moving toward the soil, whereas, no soil resistance is mobilized when the abutment moves away from the soil). Therefore, the stiffness would be too high if the full soil stiffness were used at both abutments. For most non-curved bridges, as an approximation, one-half of the total stiffness should be allocated to each abutment at both ends. When the "half-half" stiffness approach is used, the resulting abutment forces should be doubled for design. For curved bridges, it may be necessary to assign the full abutment stiffness at one end of the bridge while assigning a zero stiffness on the other end. Two dynamic response analyses are needed (each run using a full stiffness at each abutment) for this "full-zero" stiffness approach.

The transverse stiffness should reflect the potential reduction in stiffness arising from the deformability of wing walls (relative to the bridge) and the partial contact and sloping configuration of the backfills at the exterior surfaces of wing walls.

(2) Conduct Dynamic Response Analysis. Using the above abutment stiffness, conduct a dynamic response analysis of the overall bridge to determine the forces and displacements.

(3) Check Abutment Force Capacity. Using peak abutment force and the effective area of the abutment wall, solve for the peak soil pressure and check that the soil capacity has not been exceeded. The soil capacity should be based on the properties of the backfill. Caltrans have recommended a soil capacity of 7.7 ksf (370 kPa) for typical California abutment-backfill conditions (sandy soil with shear wave velocity of about 800 fps or 240 meter /sec). If the peak soil pressure exceeds the soil capacity, the analysis should be repeated with a reduced abutment stiffness to reflect plastic yielding of the backfill soil. Iterations should be conducted until the force levels are below the acceptable capacity of the abutment, prior to proceeding to the next step.

(4) Check Abutment Displacement. Compare the computed displacements against the value assumed in Step 1 in relation to the load-displacement characteristics (See Figure 1) of the abutment system for the configuration of the expansion joints and the soil capacity value. This cross check is needed to ensure that the assumed abutment stiffness reflects the load-displacement characteristics properly. If the error in the assumed stiffness is excessive, the analysis should be repeated with a revised stiffness.

The converged displacement value of the abutment (with respect to the nonlinear load-displacement characteristic) should then be evaluated against the acceptable level of displacement. Excessive deformations at the abutment may cause problems. Field inspections after the 1971 San Fernando earthquake suggest that abutments which moved up to 0.2 feet (6 cm) in the longitudinal direction into the backfill soil appeared to survive with little need for repair. Therefore, if possible, this limit should be maintained. Excessive deformations may create stability and integrity problems both at the abutment and at the bents. Deformations greater than 0.2 feet (6 cm) at abutments should be evaluated for these effects.

The above steps outlined the various design considerations that should be addressed in abutments. Development of the load-displacement characteristics of the abutment-backfill system forms the basic requirement in abutment modeling. It depends on three parameters:

(1) The initial abutment-backfill interaction stiffness prior to soil failure.

(2) The ultimate resistance of the load-displacement characteristics from backfill soil failure consideration.

(3) The magnitude of the gap at expansion joints for seat-type abutments.

Discussions of the latter two parameters (based largely on Caltrans recommendations) have been presented earlier. Caltrans Bridge Design Aids Section 14 recommended an abutment-backfill interaction stiffness coefficient of 200 kips per inch of deflection per linear foot (115 MPa) along the length of the abutment wall as a starting point for iterative analysis. This abutment-soil stiffness coefficient would be appropriate for typical California abutment backfill conditions (material with shear wave velocity of about 800 ft/sec (240 m/s) and approximately 8 feet (2.4 m) of effective height of abutment walls).

For abutment configurations and soil conditions that differ significantly from the above condition, a more general form of abutment wall-backfill stiffness equation, which considers the passive resistance of the soil, as recommended by Wilson (1988) could be used to develop the longitudinal stiffness of the end wall and the transverse stiffness of the wing wall. The equation is given by:

$$K_s = \frac{E_s}{(1-\upsilon^2)\ I} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (1)$$

where K_s is soil stiffness per unit deflection per unit wall width; E_s is the Young's modulus of the backfill soil; υ is the Poisson ratio of the backfill soil; and I is the shape factor as shown in Figure 3.

The above equation allows for input of site specific soil parameters and abutment wall configurations. As the length to height ratios for wing walls are somewhat smaller than end walls, the above equation suggests a lower shape factor I, or a higher soil stiffness coefficient (K_s) for wing walls as compared to end walls.

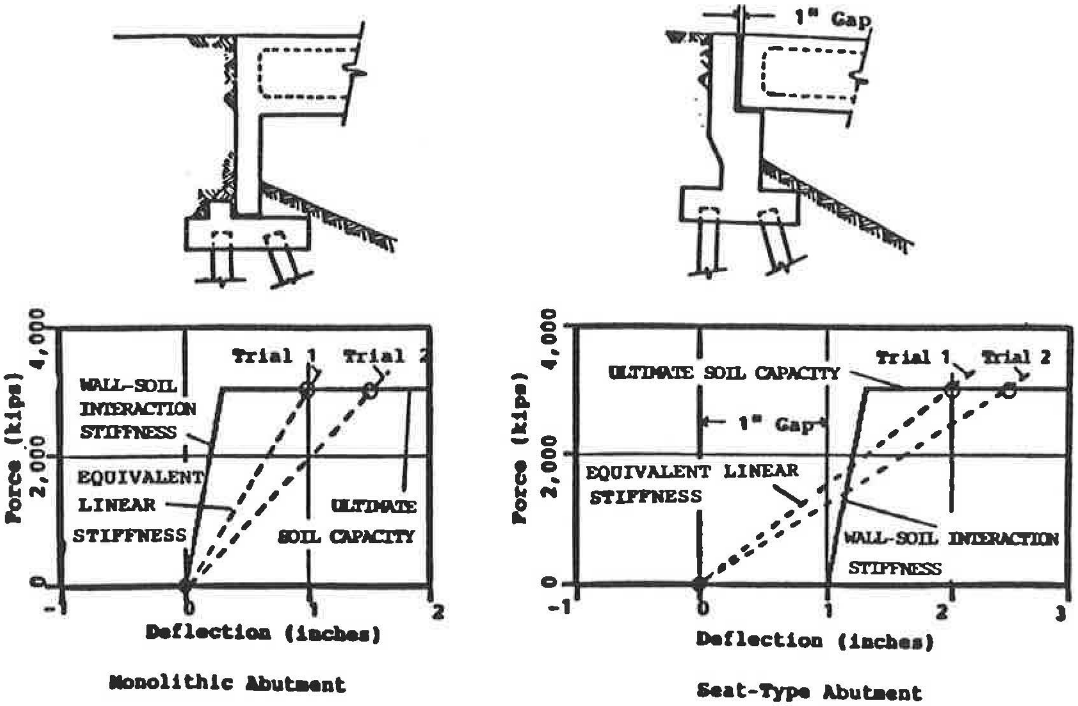


Figure 1. Load-Displacement Characteristics of Abutments

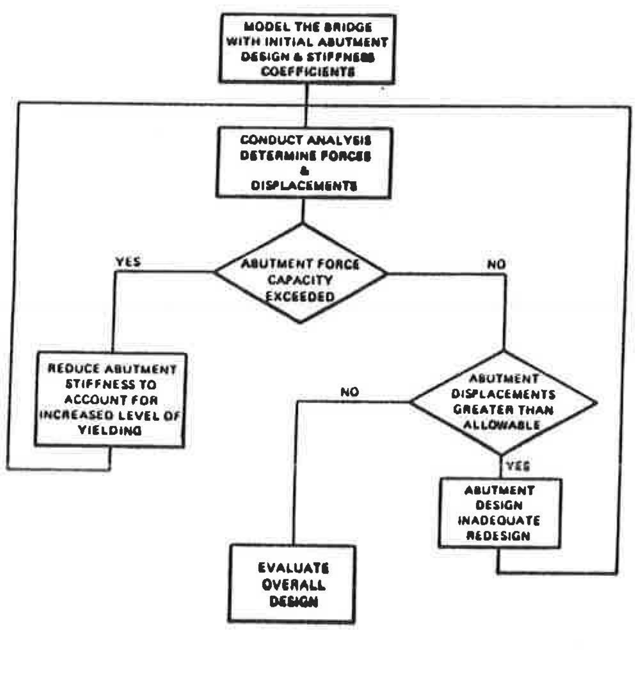


Figure 2. Iterative Procedure for the Determination of Abutment-Soil Interaction (After AASHTO, 1983)

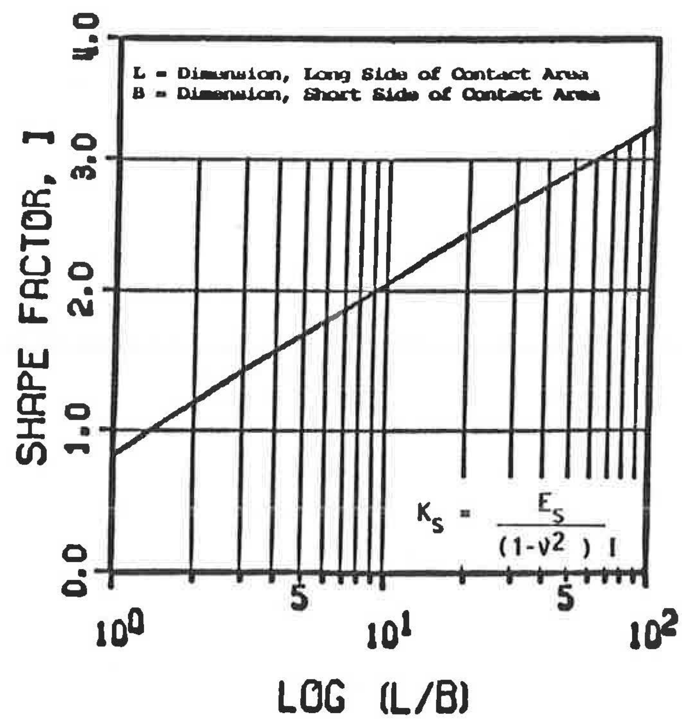


Figure 3. Shape Factor for abutment Stiffness in Equation 1

<u>Comparison to Caltrans.</u> The soil stiffness (K_s) from Wilson's equation can be compared to Caltrans recommendation through an example calculation for the longitudinal stiffness of a typical California abutment end wall-backfill system:

- Configuration of end wall: 50-ft (15.2 m) wide by 8-ft (2.4 m) high. For this configuration, a shape factor, I of 1.84 is obtained from Figure 3.

- Shear wave velocity of backfill: (800 ft/sec or 240 m/sec).

- Using a Poisson ratio of 0.3, a Young's modulus corresponding to a shear wave velocity of 800 ft/sec would be about 6.2 x 10^6 psf (300 MPa). However, a reduction factor is normally needed to adjust for a soil modulus based on shear wave velocity measurement to account for nonlinear soil behavior at higher strain levels. Some typical soil modulus variations with shearing strain curves have been recommended by Lam and Martin (1986). At a typical average shear strain value of about 0.01 percent, a reduction factor of 0.7 would be reasonable. The correspon-ding reduced Young's modulus is estimated at about 4.34 x 10^6 psf (210 MPa).

- The soil stiffness coefficient K_s for the above end wall is estimated at 216 kip/in of deflection/lineal foot of end wall (124 Mpa) as compared to the 200 kip/in /ft (115 MPa) as recommended by Caltrans.

It can be concluded that the Wilson's equation compares favorably with Caltrans recommendations and provides a rational basis for extrapolation to non California design conditions (different soil types).

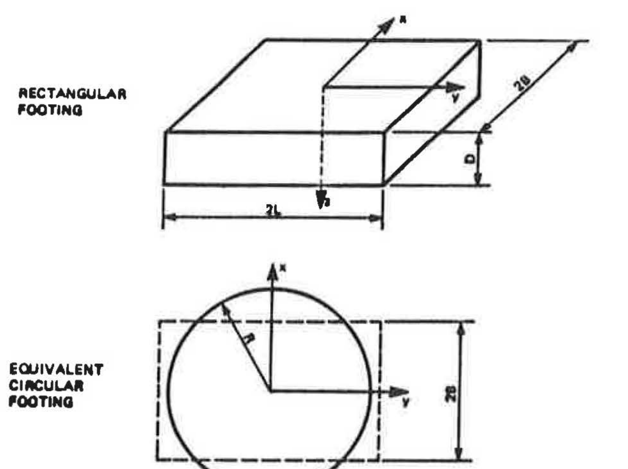
SPREAD FOOTINGS

The current state-of-practice in analyses of footings involves the use of stiffness equations of a rigid footing on a semi-infinite elastic half space. Typically, footings used for highway bridges are rectangular in shape and are embedded beneath a layer of ground cover soil. A procedure has been developed for solutions of stiffnesses of embedded rectangular footings. The procedure involves first solving for the radius of an equivalent circular footing as shown in Figure 4. The cross-coupling effects between moment and shear for most shallow footings are small and can be neglected. The diagonal stiffness terms in the stiffness matrix of an embedded rectangular footing can be obtained from equations 2.

Component	Stiffness Coefficient	
Vert. Translation	$\alpha\ \beta\ 4\ G\ R\ /\ (1-\upsilon)$	. . (2a)
Hor. Translation	$\alpha\ \beta\ 8\ G\ R\ /\ (2-\upsilon)$	. . (2b)
Tors. Rotation	$\alpha\ \beta\ 16\ G\ R^3\ /\ 3$	. . (2c)
Rocking Rotation	$\alpha\ \beta\ 8\ G\ R^3\ /\ 3\ (1-\upsilon)$	(2d)

where G and υ are shear modulus and Poisson ratio for an elastic half-space material; R is the equivalent radius as shown in Figure 4. α and β are the embedment and shape correction factors, respectively (See Figure 5).

It should be noted that the design chart was developed for a special case of zero ground cover thickness (soil thickness above the top of the slab). We recommend that, for conservatism, the thickness of the slab be used as the embedment depth (D in Figure 4) rather than the full depth from ground surface to footing base. This approximation is needed to avoid the need of an extended set of design charts to accommodate the wide combinations of ground cover and slab thicknesses.



EQUIVALENT RADIUS:

TRANSLATIONAL: $R = \sqrt{\frac{4BL}{\pi}}$

ROTATIONAL: $R = \left[\frac{(2B)(2L)^3}{3\pi}\right]^{1/4}$ .... (x-AXIS ROCKING)

$R = \left[\frac{(2B)^3(2L)}{3\pi}\right]^{1/4}$ .... (y-AXIS ROCKING)

$R = \left[\frac{4BL(4B^2+4L^2)}{6\pi}\right]^{1/4}$ .... (z-AXIS TORSION)

Figure 4. Procedures for Equivalent Radius of a Rectangular Footing

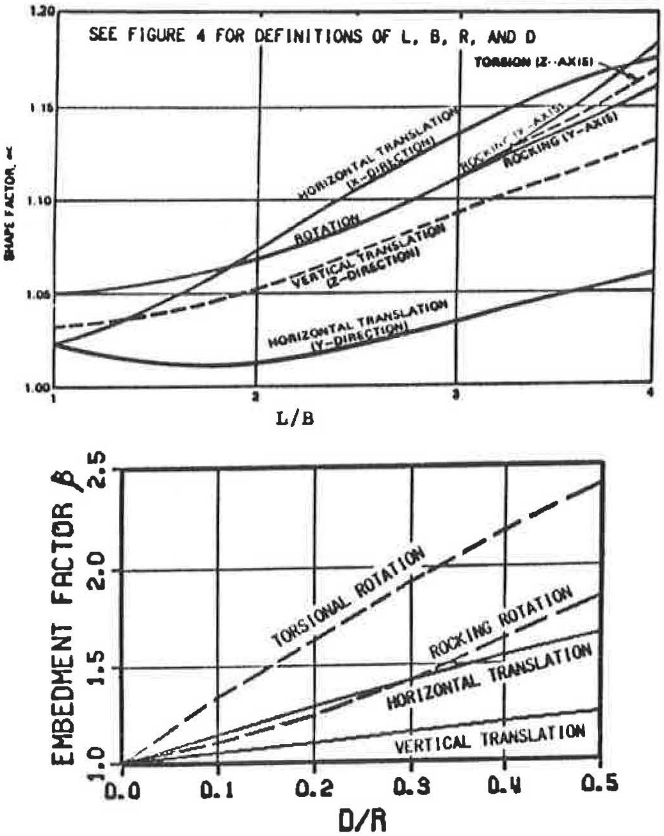
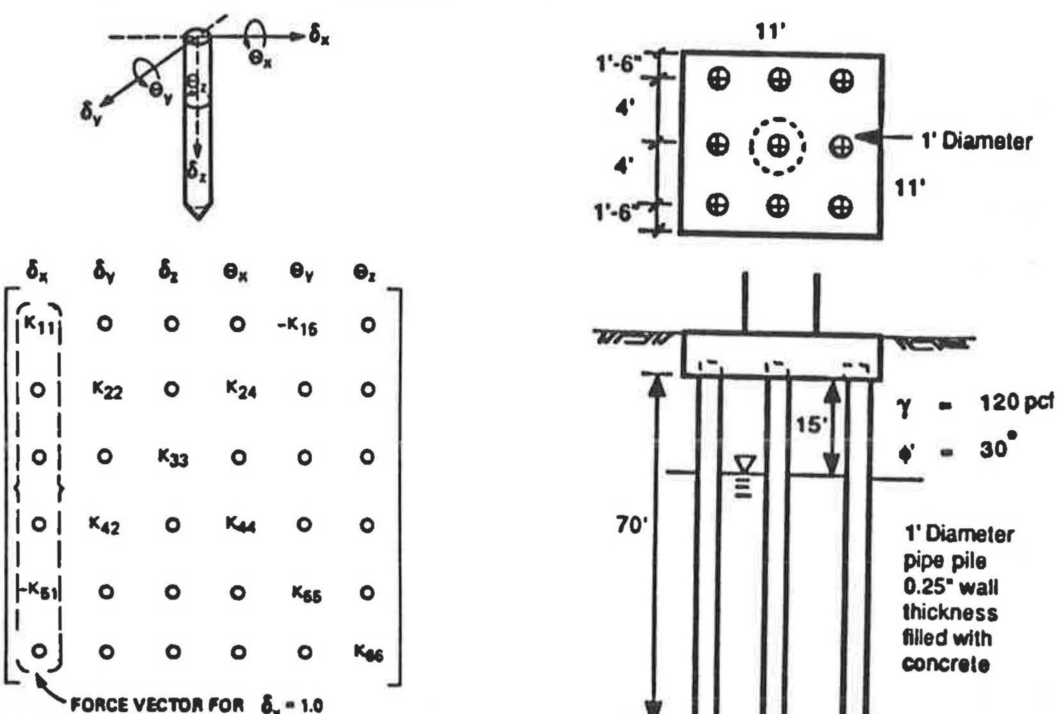


Figure 5. Design Charts for Footings

PILE FOUNDATIONS

Pile footings (pile group systems) are the most commonly used foundation systems for support of bridge structures. In a dynamic response analysis of an overall bridge structure, a pile foundation can be incorporated in the bridge model by several methods, including: (1) equivalent cantilever models, (2) uncoupled base springs models and (3) coupled foundation stiffness matrix models.

Among the three methods, the equivalent coupled foundation stiffness matrix model is the most general method of representation of foundation stiffness in a dynamic response analysis of the overall bridge. In fact, it can rigorously represent both the equivalent cantilever and the uncoupled base spring models. Because of its generality, it can represent all types of foundation systems including abutments, spread footings, pile groups and drilled shafts. The main draw back relates to the added effort to develop the coefficients in the stiffness matrix. Solutions of stiffness coefficients of a full 6 X 6 stiffness matrix as shown in Figure 6a are needed for this method. A simplified procedure has been developed for the solution of the stiffness matrix coefficients of pile groups.



(a) DEFINITIONS OF COORDINATE SYSTEMS

(b) EXAMPLE OF A PILE FOOTING

Figure 6. Form of a Pile Head Stiffness Matrix

The five basic steps involved in this simplified procedure for a pile footing such as that shown in Figure 6b are:

(1) Solve for the stiffness matrix of a single pile under lateral loading.

(2) Solve for the stiffness matrix of a single pile under axial loading.

(3) Superimpose the stiffness of individual piles to obtain the pile group stiffness.

(4) Determine the stiffness contribution of the pile cap (pile footing).

(5) Superimpose the stiffnesses of the pile cap to the pile group.

Details of the 5-Step Procedures are described below.

Lateral Load-Deflection Methods. Currently, in practice, the lateral pile-soil interaction problem is obtained by solving the problem of a beam member (modeled by finite elements, difference equations, or by discretized mechanical analog) supported on closely spaced linear or nonlinear elastic soil springs. Due to the dominance of the elastic pile stiffness over the nonlinear soil and the localized zone of influence (confined to the upper five to ten diameters), linear solutions were found to be adequate for pile stiffness evaluations (Lam and Martin, 1986). Most available linear non-dimensional solutions have been geared toward development of the total pile solution (including distributions of deflection, slope, moment and shear along the entire pile length). For the purpose of foundation stiffness evaluation, the total pile solutions can be simplified to provide coefficients of pile-head stiffness matrix as shown in Figures 7 through 9. Pile-head stiffness coefficients can be obtained for a combination of bending stiffness of the pile (EI) and the coefficient of variation of soil reaction modulus E_s with depth (f).

Recommendations for the coefficient f for sand are available in the literature (Terzaghi, 1955; and O'Neill and Murchison, 1983). At normal working load levels (pile-head deflection between 0.5 to 1.0 inch or 1.3 to 2.5 cm), Terzaghi's recommendation is considered reasonable for sand and his recommendation is presented in Figure 10. Recommendations for the coefficient f for clays have been developed from correlations of nonlinear computer solutions at a pile-head deflection (fixed head condition) of about 1-inch (2.5 cm) by

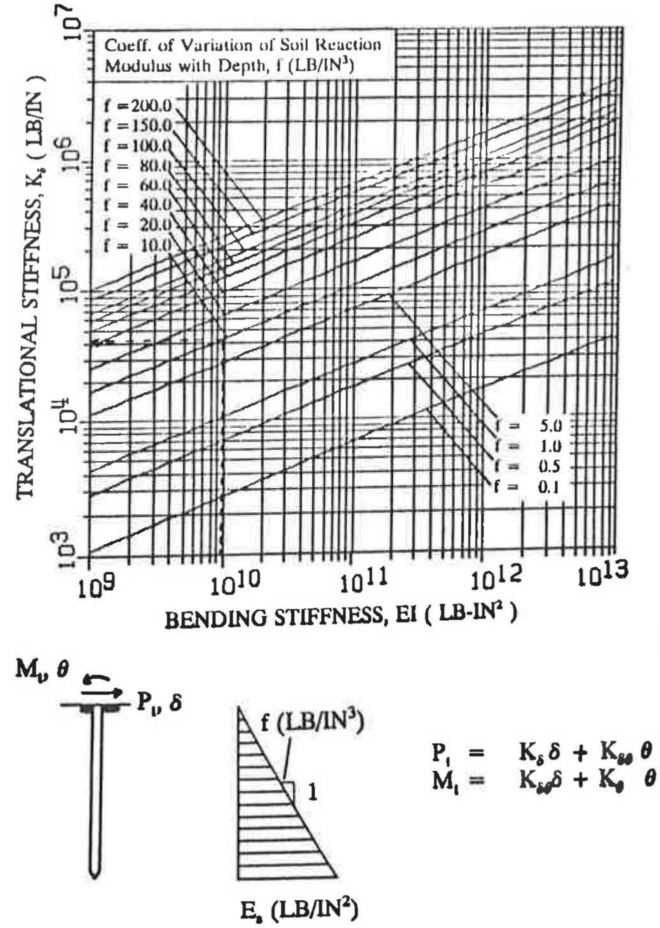


Figure 7. Pile Translational Stiffness, K_δ

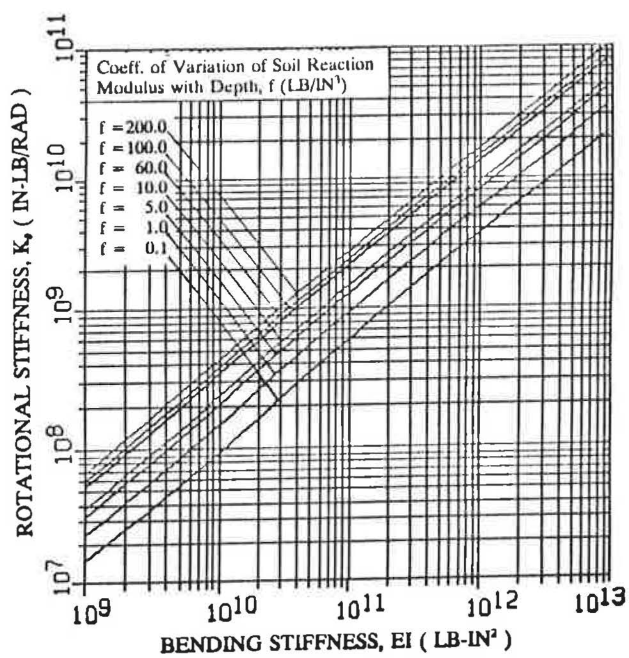


Figure 8. Pile Rotational Stiffness, K_θ

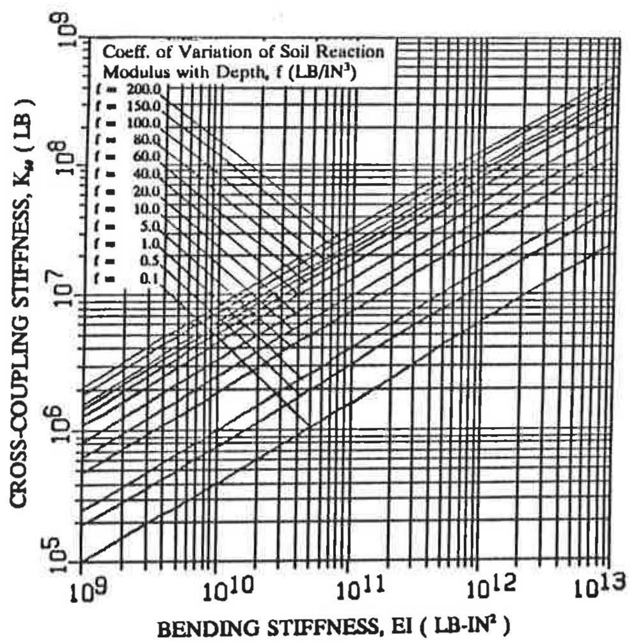


Figure 9. Pile Cross-Coupling Stiffness, $K_{\delta\theta}$

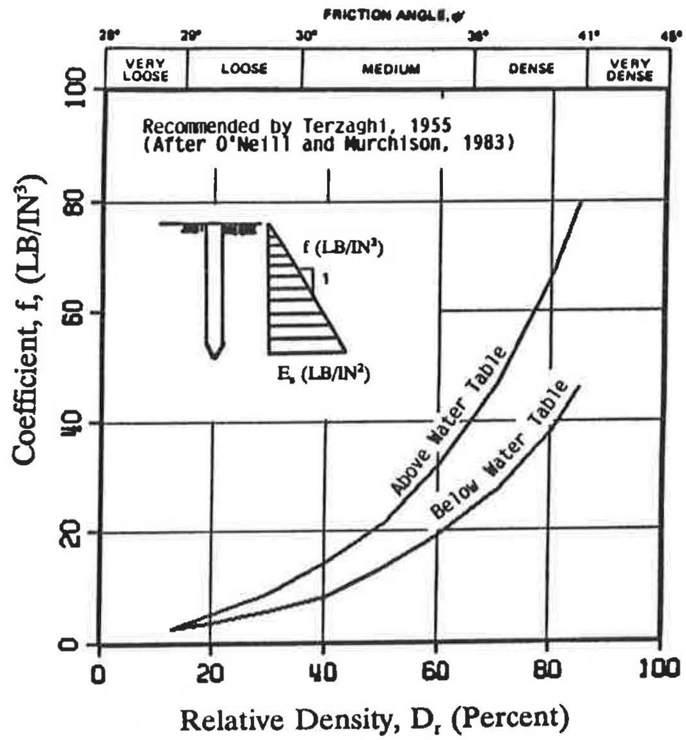


Figure 10. Recommendations for Coefficient f for Sands (Note: 1 LB/IN^3 = 0.27 N/cm^3)

the authors. This recommendation and results of the correlation for clay are shown in Figure 11. Only the upper five diameters of soils (soil type and ground water) need to be considered in usage of the presented design charts.

Limitations of Approach. There are several simplifying assumptions in the presented approach. The coefficient f is not an intrinsic soil parameter. The recommendations for f presented in Figures 10 and 11 are appropriate for piles in typical highway bridge foundations (i.e. smaller piles). Furthermore, the embedment effect has not been taken into account in the procedure. Therefore the recommendations are conservative and appropriate for shallow embedment conditions (say less than 5 feet or 1.5 m).

Although correlations for the coefficient f can be conducted for other conditions (e.g. larger piles and bigger embedment depths), the additional complexity negates the merits of the use of simplified linear elastic solutions. For such cases, computer solutions, which can readily accomodate nonlinear effects and more general boundary conditions, are recommended.

Comparison to Caltrans Practice. The above procedure can be compared to the practice adopted by Caltrans. In Caltrans

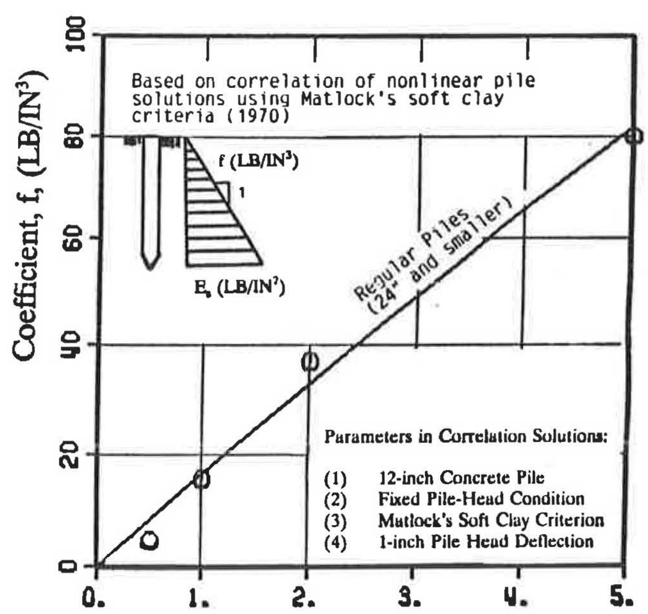


Figure 11. Recommendations of Coefficient f for Clays (Note: 1 LB/IN^3 = 0.27 N/cm^3)

Bridge Design Aids Section 14, a pile head stiffness of 40 kips per inch deflection (70 kN/cm) is recommended for a standard 16-inch (40-cm) CIDH (Cast-in-Drilled-Hole) pile. The bending stiffness (EI) of the above pile is estimated at 9.7×10^9 in^2-lb (2.8×10^{11} N-cm^2). For a typical sandy soil condition (friction angle = 30°), the coefficient f, obtained from Figure 10, is about 10 pci (2.7 N/cm^3). For the above combination of bending stiffness and subgrade stiffness, a lateral pile-head stiffness coefficient (K_δ) of 42 kips per inch (73.5 kN/cm) of deflection is obtained from Figure 7. The 42 kips per inch compares favorably with the Caltrans recommendation of 40 kips per inch (70 kN/cm).

<u>Sensitivity of Boundary Condition</u>. The above stiffness coefficient, K_δ, represents the lateral stiffness of a fixed-head pile (zero rotation). The rotational and the cross-coupling stiffness terms for the above (16-inch or 40.6-cm CIDH) pile can be obtained from Figures 8 and 9, respectively and summarized in the following paragraph:

- o Rotational stiffness (K_θ) (From Figure 8) = 2.3×10^8 in-lb/rad (26 MN-m/rad)
- o Cross-coupling stiffness ($K_{\delta\theta}$) (From Figure 9)= 2.3×10^6 lb (10 MN)

The above coefficients can be used to solve for the load/moment-deflection/rotation relationship for any combination of shear and moment or pile-head constraint condition. For example, if the pile head condition is a pinned head connection detail, the zero pile-head moment leads to the following relationship between pile head deflection (δ) and rotation (θ):

$$\theta = -(K_{\delta\theta} / K_\theta)\ \delta \quad . . . . \quad (3)$$

The above relationship can be substituted into the pile head force equation in Figures 7 through 9 for the lateral stiffness (ratio of force to pile-head deflection) of a free-head pile as presented below:

Lateral stiffness of free head pile
$= K_\delta - K_{\delta\theta}^2 / K_\theta$ = 19 kips/in (33.3 kN/cm)

From the example, it can be observed that the lateral stiffness could vary from 42 to 19 kips per inch (70 to 33.3 kN/cm) for a fixed versus a free pile-head condition. It can be concluded that a realistic representation of the pile-head connection is very important, and often of more significance than the selection of soil parameters.

<u>Role of Axial Stiffness</u>. The role of lateral loading on piles is usually emphasized for earthquake consideration. However, in a pile group system, the base moment from structural loading is reacted largely by the moment couple from variation in axial load among individual piles as compared to reaction from pile-head bending. Experience from recent earthquakes and research have provided ample evidence (Rosenblueth, 1986; and Douglas et al. 1984) that the rotational stiffness of a pile group, which is related to the axial pile stiffness, will have a dominating effect on the overall structure as compared to the lateral stiffness. Therefore, evaluation of axial pile stiffness is critical for realistic modeling of foundation stiffness.

For a generalized soil-pile condition, the following factors need to be accounted for in evaluation of the overall axial pile behavior and pile head load-displacement characteristics:

- o The stiffness characteristics of the pile (AE) and pile length (L).
- o The shear transfer-displacement characteristic of the soil along the length of the pile shaft (related to the cumulative ultimate skin-friction capacity).
- o The end-bearing load-displacement characteristic of the soil at the pile tip (related to the ultimate end-bearing capacity).

In a normal design condition, a pile foundation derives a significant portion of the soil reaction throughout the pile shaft as well as the pile tip. Unless the pile is very lightly loaded, plastic slippage at the pile-soil interface will occur along a significant upper portion of the pile. Furthermore, the nonhomogeneous, layering nature of soil deposits must be accounted for in axial stiffness evaluation. Due to these complexities, linear analytical procedures would be of limited practical applications and nonlinear analyses are preferred for axial pile response.

<u>Solutions for Axial Stiffness</u>. Uncertainty in axial soil-pile interaction analysis relates largely to uncertainties in soil parameters including the ultimate pile capacity (skin-friction and end-bearing) and load-displacement relationships. Computer solutions can be used for a rigorous nonlinear solution. An approximate nonlinear graphical solution method has been developed and presented by Lam and Martin (1984, 1986). It will be described below. The procedure is

schematically shown in Figure 12 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 12.

(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 can 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 Equation-4.

$$\text{Pile Compliance } (\delta_c) = \frac{Q\,L}{A\,E} \quad . \; . \; . \; . \quad (4)$$

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

(4) Intermediate Pile Stiffness Solution. The "correct" solution, as indicated by the computer solution, 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 combinations of pile/soil situations (end-bearing and friction piles as well as any pile type: long or short and any pile material).

(5) Selection of Secant Stiffness. The secant axial pile stiffness appropriate for use in dynamic response analyses should reflect the unloading and reloading behavior. The secant stiffness from the origin of the load-displacement curve to the cyclic load level (without the static bias load) would be appropriate for dynamic response analyses.

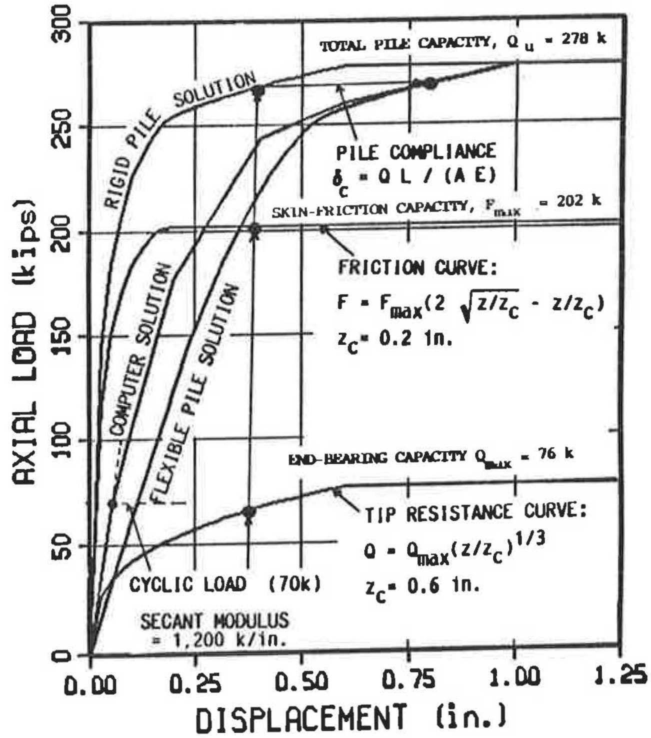


Figure 12. Graphical Solution of Axial Pile Stiffness

Stiffness Matrix of a Single Pile. Using the above described procedures, results of the pile-head stiffness coefficients of a single typical pile for the pile footing shown in Figure 6b are tabulated in Table 1. It should be noted that the torsional stiffness of a single individual pile can usually be ignored and assumed to be zero. The torsional moment on a pile group is usually reacted by the torsional moment couple from variation in lateral shears among individual piles in a pile group.

Stiffness for a Pile Group. 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 6b, the form of the stiffness matrix will be identical to the individual pile (as shown in Figure 6a). 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 stiffness for an overall pile group stiffness matrix has been documented in the FHWA report by Lam and Martin (1986). The program can also be used to distribute the overall foundation load to individual piles.

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 an 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 translational 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:

$$R_g = N\, R_p + \sum_{n=1}^{N} K_{\delta n}\, S_n^2 \quad \ldots\ldots\ldots\ldots \quad (5)$$

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 translational stiffness coefficient of an individual pile, 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 stiffnesses of the overall pile group system shown in Figure 6 is developed and presented in Table 1. It can be observed that the rocking rotational stiffness coefficients of the pile group are dominated by translational stiffnesses of piles. The rotational stiffness from pile bending can virtually be ignored for most bent foundations.

The above presented 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 view of the lack of evidence that "group effects" contributed to failure of bridges and the lack of well proven approaches for treatment of "group effects"; and above all for the sake of simplicity, we have neglected "group effects" in our presented procedures.

<u>Stiffness of Pile Cap.</u> So far, the presented procedure deals with stiffness of the pile-soil system. In a typical highway bridge situation, there will be additional stiffnesses arising from the pile cap. Our experience indicates that the lateral stiffness from (1) passive resistance on the vertical surface/s and (2) tractional shear forces at the base of a pile cap could be very significant as compared to the lateral pile stiffness. Wilson's equation for abutment stiffness presented earlier can be used for evaluation of the passive resistance component of cap stiffnesses. The spread footing procedure for an unembedded (surface) footing can be used for evaluation of the cap stiffness for soil reactions at the cap base. The pile footing example problem shown in Figure 6b is used to illustrate the relative stiffness of the pile group versus the pile cap. The stiffness contribution from piles have been developed earlier (See Table 1). The pile-cap stiffnesses including: (1) the lateral passive resistance component developed from Wilson's method (Equation 1) and (2) the 6 degrees-of-freedom soil reaction at the base of the cap (from spread footing Equations 2) are presented in the following table for comparison. A shear modulus of 7.2×10^5 psf or 34.5 Mpa (a conservative, or low value for compacted backfills in most construction practice) and Poisson ratio of 0.35 were used in the pile-cap stiffness evaluation presented in Table 2.

From Table 2, it can be concluded that, the vertical pile stiffness dominates the response of the vertical translational and the rocking rotational stiffnesses of the overall foundation. The influence of the pile cap is relatively minor for these two modes. However, the lateral footing stiffness is quite high relative to the lateral pile stiffness and dominates the lateral and the torsional rotation stiffness terms.

Table 1. Pile Stiffness Solution

Stiffness Coefficient	Single Pile	Pile Group
Lateral Translation $k_{11} = k_{22}$, (kip/in)	42	$9 \times 42 = 378$
Vertical Translation k_{33}, (kip/in)	1,200	$9 \times 1{,}200 = 10{,}800$
Rocking Rotation $k_{44}=k_{55}$ (in-kip/rad)	193,000	$N R_p + \sum_{n=1}^{N} K_{\delta n} S_n^2$ (Eq. 5) $= 1.74 \times 10^6 + 1.66 \times 10^7$ $= 1.83 \times 10^7$
Torsional Rotation k_{66}, (in-kip/rad)	0	$4\text{x}42\text{x}48^2 + 4\text{x}42\text{x}(48^2+48^2)$ $= 1.16 \times 10^6$
Cross-Coupling $k_{15}=k_{51}=-k_{24}=-k_{42}$, (kip)	-2,250	$9 \times -2{,}250 = -20{,}250$

See Figure 6 for definition of stiffness coefficients and example problem. Note that the pile size is different from that used in earlier discussion on Caltrans standard 16-inch CIDH pile.

Table 2. Comparison of Pile vs. Cap Stiffness

Stiffness Component	From Piles	From Pile Cap: Base of Footing	From Pile Cap: Passive Pressure Vert. Face
Lateral Translation (k/in)	378	1,833	1,167
Vertical Translation (k/in)	10,800	2,333	N/A
Rocking Rotation (in-k/rad)	1.8×10^7	5.2×10^6	N/A
Torsional Rotation (in-k/rad)	1.2×10^6	1.2×10^7	N/A
Cross Coupling Between (k) Lateral Trans. and Rocking Rotation	2.0×10^4	0.0	N/A

See Figure 6b for configuration and Table 1 for pile stiffnesses.

Notes: 1 k/in = 1.75 kN/cm; 1 in-k/rad = 11.3 cm-kN/rad; 1 k = 4.45 kN

Significant engineering judgement is required on the use of the above estimated stiffness coefficients for a pile footing. Considerations regarding interaction between piles and the footing and other factors such as the contact condition between the cap bottom and soil in the presence of the piles introduce uncertainties on the use of the footing base stiffnesses (the second column Table 2). However, it would be prudent to add the pile-cap lateral passive soil resistance (the third column) to the pile stiffness in design practice. Caltrans have adopted similar view point in their design practice. Ignoring the footing base is considered conservative. It can be observed that even if the bottom tractional stiffness is ignored, the lateral passive resistance of the pile cap would dominate the lateral stiffness of the pile group.

CONCLUSIONS AND RECOMMENDATIONS

Procedures and accompanying design charts to facilitate practical solutions of abutment and foundation stiffnesses for dynamic response analyses of typical highway bridges have been developed and presented. The presented procedures are relatively simple and emphasis has been placed on hand and graphical solution methods for easy application. An in-house Earth Mechanics project to computerize the presented procedures is presently being undertaken.

In view of uncertainties in ground motion which could lead to differential foundation movements and other geotechnical concerns (e.g. ground stability and liquefaction problems), ductility design has significant technical merit for earthquake resistance. Allowance for ductility tends to lead to a design more tolerant to foundation movements. However, ductility design requires realistic evaluation of the magnitude of displacements and deformations and design provisions (e.g. allowance of minimum seating widths as recommended by Caltrans) to accommodate the displacements.

Incorporation of foundation and abutment stiffness in design and retrofit analyses of highway bridges leads to an improved solution of the overall seismic load level and the distribution of the overall load among various bents and abutments. More importantly, it leads to better estimates of displacements. However, uncertainties in ground motion and other geotechnical concerns warrant an even more prudent approach to provide for potential bridge displacements. This has led to the specification of minimum seat widths by Caltrans and bridge designers should be cognizant of this issue. Provision of a sufficient seat width represents excellent earthquake design practice and can be accommodated economically for most new designs. The role of foundation stiffness becomes more important in retrofit situations especially when a more realistic analysis approach is warranted to reduce the level of conservatism associated with uncertainties in analytical procedures.

The above procedures on foundation stiffness can also be applied for temperature loading evaluations. Significant reduction (relaxation) in the structural stresses can usually be realized if the foundations are allowed to deform. Therefore introduction of foundation stiffnesses in bridge analysis would usually lead to a more economical design.

There are other geotechnical considerations, especially those related to ground stability that have not been addressed in this paper. Cooperation between structural and geotechnical engineers is strongly recommended to address such issues, especially for poor soil conditions. In addition, a number of sensitivity issues have been discussed in this paper. Although an attempt has been made to ensure our example problems reflect real typical situations, one must be careful in extrapolating the presented discussions and results to other design conditions.

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