Urban Earthquake Engineering: Foundation Characterization for Performance Based Design

W. D. Liam Finn, Anabuki Professor,
Kagawa University, Takamatsu, Japan

ABSTRACT: The major development in urban earthquake engineering in the last 10 years is the concept of performance based design. A crucial feature of this new design approach is recognition of the impact of flexibility of foundations on the seismic demand and seismic response of structures. The challenge for geotechnical engineers is to characterize the actions of the foundation-soil system effectively in a manner compatible with commercial software used in design practice. The state of practice for characterization of the actions of the foundations on structural response is reviewed and the effectiveness of various approximate approaches are evaluated.

1.0 INTRODUCTION

Traditionally the objectives of seismic design codes for structures were to protect life safety under the extreme events envisioned by the codes and to maintain serviceability under the smaller events with a greater probability of occurring during the life of the structure. During the 1985 Loma Prieta earthquake in California the life protection objective was met but the level of damage was considered high for such a short duration, moderate earthquake. The direct costs of repair or replacement of buildings and in many cases huge indirect losses due to business interruption motivated the progressive structural engineers of California to critically review the design concepts and propose the idea of performance based design.

The concept underlying performance based design is to design for an acceptable damage level specified by the owner. The range of performance options is illustrated in Fig.1. Note that along the capacity curve for the structure, potential performance options are defined in terms of global displacement. The effectiveness of a performance based design is assessed by an appropriate nonlinear analysis that establishes the demand on capacity in terms of global displacement. The location of the calculated performance point on the capacity curve relative to the desired performance point is a measure of how design meets the design criteria.

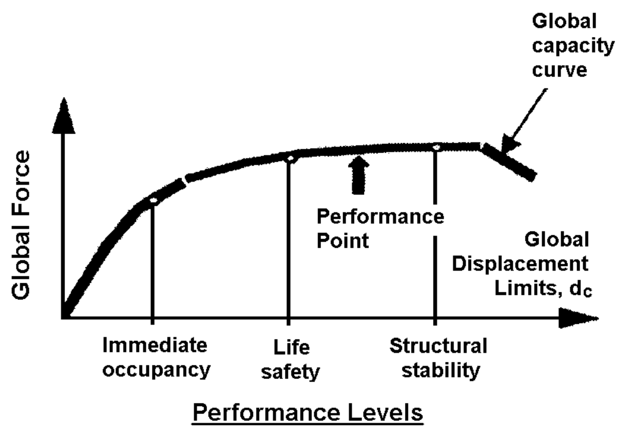

Fig. 1. Performance levels for seismic design (modified from (1))

A nonlinear pushover analysis is commonly advocated but it is recognized that in some cases a nonlinear dynamic analysis may be required. For an evaluation analysis to be valid, the structural model must include all elements of the building-foundation-soil system that significantly affect the seismic demand and response of the structure. Hitherto the seismic demand for code designed buildings was determined assuming that the structure rested on a rigid base. The actions of the foundations were completely ignored. The demands of performance based design make it imperative to include the effects of soils and foundations on seismic demand and structural response.

The first detailed examination of how to include the actions of the soils-foundation system in code design provisions was presented in a report by the Applied Technology Council (2) in which tentative provisions for seismic regulations were advanced. This procedure was based on the work of Veletsos and Wei (3). Veletsos et al (4) presented an updated review of these issues in a state of the art paper to the 9th World Conference on Earthquake Engineering in Tokyo. They noted that a proper analysis of the physical system of structure, foundation and foundation soils was necessary to get reliable estimates of structural demand and seismic response. They evaluated the relative contributions of kinematic and inertial interactions and concluded that inertial interaction had the greatest influence. These studies were based on analyses of a simple elastic structure on a rigid mat foundation welded to a homogeneous, elastic half space. Today the technology is available to consider flexible foundation elements and nonlinear soil response.

Investigations of structural performance during earthquake loading have confirmed the importance of treating the structure, foundation and foundation soils as a complete system. Wallace et al. (5) analyzed the seismic response of two 10-storey buildings designed and built in the 1970's. The buildings were instrumented to record strong motions. One building, in Northern California, was analyzed for the motions recorded during the 1984 Morgan Hill earthquake (Ms=6.2). The second building, in Southern California, was analyzed for the motions recorded during the Whittier earthquake (Ms=5.9). Good correlation was achieved between computed and recorded motions, when the flexibility of the foundation–soil system and cracked section properties were taken into account. Otherwise the correlation was poor.

The authors also evaluated the response of shear wall buildings in Chile that performed unexpectedly well during the 1985 Chilean earthquake. On the basis of conventional rigid base analysis these buildings had a ductility demand of 3 and should have suffered appreciable damage. However, when the effects of foundation flexibility were taken into account, the ductility demand dropped to 2. The beneficial effects of foundation flexibility were an important factor leading to the good performance.

Foundation flexibilities in the cases discussed above were all based on treating the soil response as elastic. More recent developments consider the nonlinear response of the soil to strong shaking. Furthermore allowing uplift during rocking and permitting yielding of the foundation soils is being advocated to reduce seismic demand on structures. These new advances are important developments for performance based design of new buildings but are even more important for developing cost effective retrofit strategies. Retrofitting to meet current code demand levels can be prohibitively expensive. The inclusion of foundation flexibility gives a more realistic picture of where retrofits are critical and can lead to lower seismic demand. These benefits result in more cost effective retrofits.

Clearly to evaluate a performance based design or the capacity of retrofitted building requires a realistic computational model of the structural system. This, in turn, requires a way of characterizing the actions of foundations and supporting soils on structural response that is compatible with available commercial computational software for structural analysis.

In this paper methods of characterizing both shallow and pile foundation will be presented.

2.0 CHARACTERIZATION OF SHALLOW FOUNDATIONS FOR STRUCTURAL MODELS

2.1 Spring models

The simplest way of modeling the force–displacement behavior of a shallow foundation to the seismic actions imposed on it by the structure is by means of discrete uncoupled elastic springs as shown in Fig. 2 (7) The footing is assumed to be a rigid plate, welded to a semi-infinite, homogeneous, elastic half-space. The stiffnesses of the translational and rotational springs are determined from published solutions. Surface stiffnesses for a rigid, circular plate on a semi-infinite homogeneous half space, published by Gazetas (6), are frequently used. Charts of shape and embedment factors to modify the surface stiffnesses of the circular plate for the effects of noncircular shape and depth of embedment are given in NEHRP (8).

It is more common to use Winkler springs, shown also in Fig. 2. These eliminate the rotational spring and facilitate the study of foundation uplift. The vertical springs in the Winkler model must be selected to represent both the vertical and rotational stiffnesses, if both stiffnesses are to be included in the analysis at the same time. One method for doing this

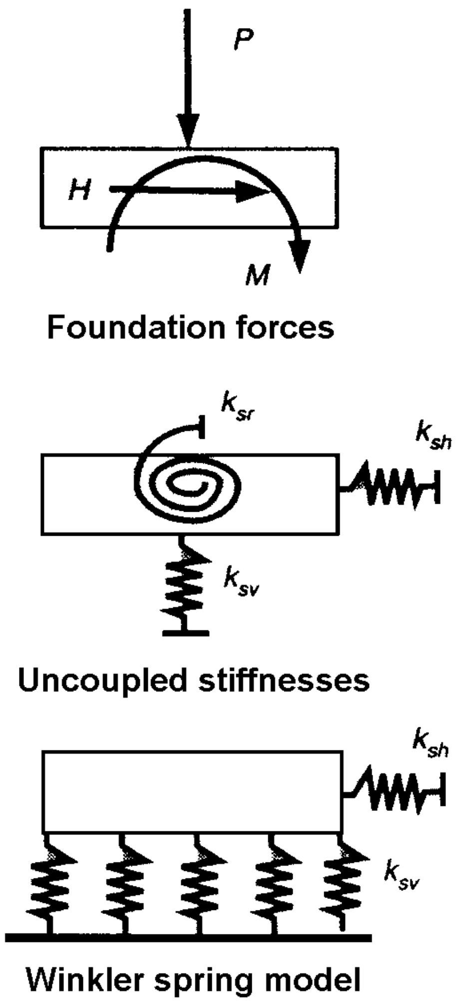

Fig. 2. Spring models for a spread footing (after (7))

is shown in Fig. 3 (8). The exterior stiffnesses in the B/6 wide end zones of the footing are assigned a stiffness of

$k_{end} = 6.83G/(1-\mu)B$ (1)

and, in the mid-section, of

$k_{mid} = 0.73G/(1-\mu)B$ (2)

Here k represents the stiffness/unit area, G is the shear modulus, μ is the Poisson ratio and B is the width of the footing. The distributed unit stiffnesses may be converted to individual spring stiffnesses, K_i, as shown in Fig.3, where K_i is given by

$K_i = L_i\, k\, B$ (3)

Here L_i is the length contributing to the stiffness of spring K_i.

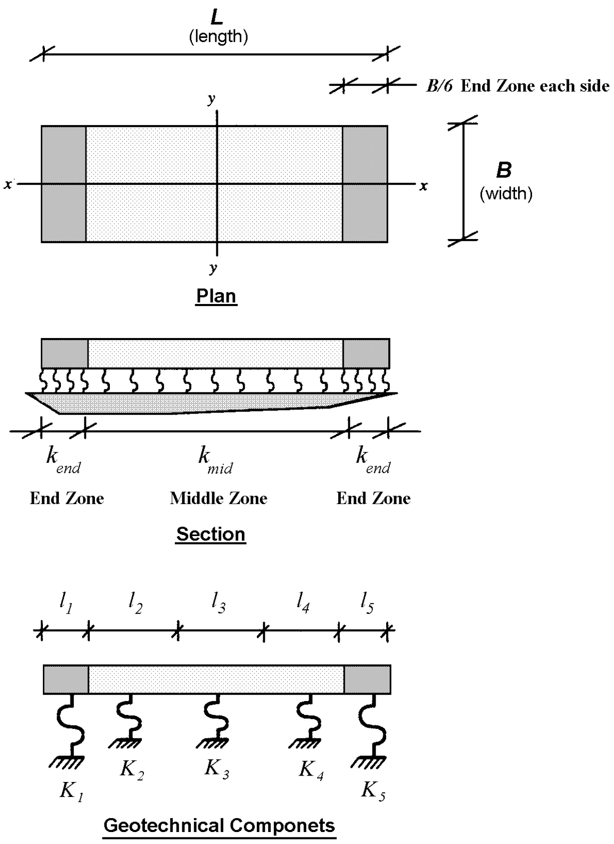

Fig. 3. Combined translation and rocking model for a footing (modified from (8))

In practice the modulus G is not constant with depth. Selection of a single representative value requires experience and a good knowledge of the dynamic response of surface footings. Little direct guidance is provided in the literature.

The response of soils to strong shaking is nonlinear, therefore the modulus used in the formulae to compute the translational and rotational stiffnesses should reflect the average effective moduli in the ground during shaking. An approximate way of modifying the effective, small strain elastic modulus, G_o, to the effective modulus, G, during to account for nonlinearity is to estimate the ratio, G/G_o, from Table 1 (adapted from (8)).

Table 1 Effective shear moduli G as a function of shaking intensity (8)

Modulus Ratio	Effective Peak Acceleration	
	0.10	0.70
G/G_o	0.50	0.20 *

* 1. Site specific values from a geotechnical site investigation may be used
2. Use linear interpolation for intermediate values

2.2 Rocking with Uplift and Yielding

The effect of incorporating rocking, without uplift, into the computational model of a structure is to increase the fundamental period of vibration. This leads to a reduction in seismic base shear for taller buildings in designs based on code response spectra because the fixed based periods of these structures are associated with the longer period region of the code spectrum where spectral amplitudes decay with increasing period.

It is very expensive to prevent uplift in taller building on fully occupied city lots. Typically it involves massive foundation slabs and/or soil anchors. Structural engineers consider that uplift can be allowed. Housner (9) was one of the first to study in detail the behavior of rocking structures with uplift. One important conclusion of his work was that the evaluation of the stability of rocking structures could not be based on the static application of the dynamic forces causing rocking.

Rocking can be evaluated using the Winkler model by allowing separation to occur between structure and the spring with the onset of tension. The analysis may be conducted keeping the springs elastic. Uplift response is highly geometrically nonlinear. It results in an increase of period and damping. Uplift may result in a reduction in demand but it does so at the expense of increased global displacements.

Idealized stress distributions that may occur under a rocking foundation are shown in Fig.4
The stress at the compressive edge is limited by the yield pressure q_u of the soil. Rocking with uplift and yield was investigated in detail by Bartlett (10).for a clay foundation. He developed the theory for the relationships between overturning moment and foundation rotation. A very detailed and lucid description of uplift with soil yielding and how it affects seismic demand and response is given by Martin and Lam (11). They discuss at length how to incorporate uplift with soil yield into seismic design procedures.

From Fig. 4 it may be seen that, when the ratio of applied pressure, q, from a concentric load, P, to the yield pressure, q_u, is q/q_u, < 0.5, uplift occurs before yielding takes place. Otherwise soil yielding occurs first. The moment capacity of the footing, M_c, when full yielding occurs is given by

$$M_c = PL\,(1-q/q_u)/2 \qquad (4)$$

Martin and Lam studied the impact of uplift with soil yielding on the seismic demand and retrofit strategy for a 8-storey building. They found that rocking and compressional yielding occurred early in the response and over two thirds of the deformation demand was absorbed by the foundation soils. This reduced the demand on the capacity of a shear wall so that the structure met life safety requirements. One surprising and significant finding from their study was that variations in stiffness and strength between 67% and 150% did not lead to a significant change in behavior. They concluded from this that the response was much more

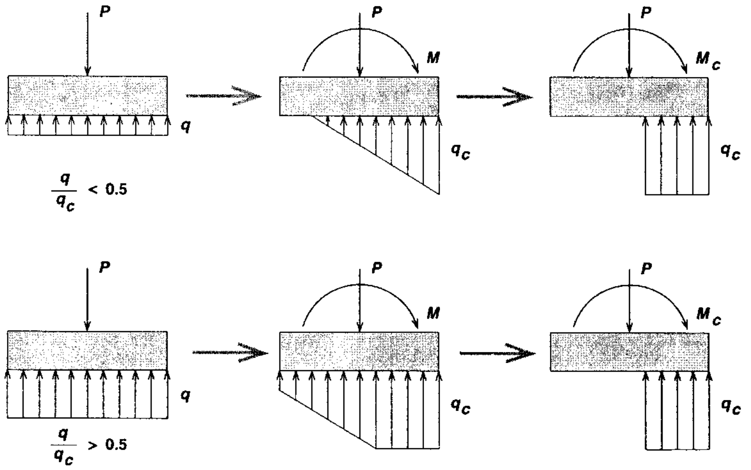

Fig. 4. Idealized stress distributions for rigid footings subjected to overturning moment (after (8))

sensitive to nonlinear rocking than to exact soil properties.

The benefits of rocking and uplift come at the price of increased displacement. Most of the analyses of the effects of rocking have been conducted on structures with one dominant structural system. For structures with combined systems such as shear walls and moment resisting frames, the reduction in demand in one system may transfer demand to the other system.

3.0 CHARACTERIZATION OF PILE FOUNDATIONS

. A major weakness in some models is the inadequate representation of the effects of the foundations on the structure, especially of pile foundations. The actions of pile foundations are represented by discrete, single valued springs to model rotational and translational stiffnesses and any coupling between these springs is usually ignored. The spring stiffnesses are frequently estimated using approximate, simplified methods of unknown reliability. This is a natural consequence of the complexity of a full 3-D nonlinear dynamic analysis of pile foundations. Even for the elastic case, only a limited number of 3-D parametric studies have been published. These have focused mainly on providing dynamic interaction factors between piles in small groups or frequency dependent stiffnesses and damping for single piles.

A complete picture of the effects of the foundation on the structure during strong earthquake shaking requires taking simultaneously into account many factors such as soil non-linearity, seismically induced pore water pressures, kinematic interaction between piles and soil, inertial interaction of the superstructure with soil and piles and dynamic interaction between the piles themselves. All of these factors can be taken into account by a non-linear, effective stress, dynamic, continuum analysis. Such an analysis provides time histories of direct and coupled foundation stiffnesses and demonstrates the effects of kinematic and inertial interactions, the effects of pore water pressures and soil nonlinearity. One prime benefit of such analyses, in addition to their use in the context of a specific design, is that results of parametric studies provide the data base for evaluating the effectiveness of the various approximate methods in use.

A comprehensive overview of the behavior of pile foundations during earthquakes using non-linear dynamic effective stress continuum analysis is presented here. The presentation is limited to nonliquefiable soils. It is hoped that the overview will useful in providing a framework for exercising judgment and an understanding of the limitations of approximate methods that facilitates the selection of an appropriate method for a particular application.

4.0 METHODS OF ANALYSIS

The pile foundation-structure system vibrates during earthquake shaking as a coupled system. Logically it should be analyzed as a coupled system. However this type of analysis is generally not feasible in engineering practice. Many of the popular structural analysis programs do not include the pile foundation directly into a computational model. Therefore the pile head stiffnesses are typically calculated by analyzing the pile foundation without any mass contribution from the superstructure. The analysis is done usually for a single pile and the group stiffnesses are evaluated using pile interaction factors, often static factors, or a group reduction factor.

Seismic analysis of a pile foundation for design purposes is often conducted by applying the base shears and moments from a fixed base analysis of the structure to the pile head and using a static analysis to estimate moments, shears and displacements in the piles. The most common approach to such an analysis is to use a Winkler spring computational model. A general Winkler model is shown in Fig. 5 which can be used for static or dynamic analysis. For static analysis, only the pile and the near field springs are used.

The springs may be elastic or nonlinear. Some organizations, such as the American Petroleum Institute [12], gives specific guidance for the development of nonlinear load-deflection (p-y) curves as a function of soil properties to represent nonlinear springs. The API (p-y) curves, which are widely used in engineering practice, are based on data from static and slow cyclic loading tests in the field. Murchison and O'Neill [13] suggest that the reliability of the Winkler (p-y) model may not be high.

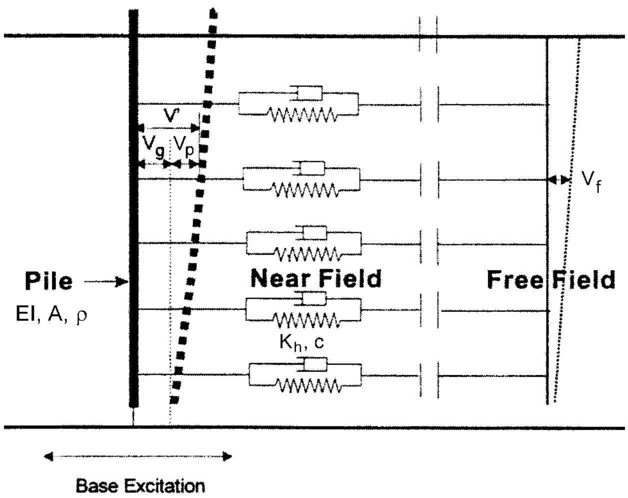

Fig. 5: Dynamic Winkler computational model for pile analysis.

The simple static analysis neglects many important factors that affect the seismic response of the structure-soil-foundation pile system. Inertial interaction between structure and foundation is neglected. This interaction increases the nonlinear behavior of the soil and reduces pile head stiffnesses. These effects increase the period of the system and change the spectral response and hence the base shears and moments. The kinematic moments are also neglected. These moments arise from the pressures generated against the pile to ensure that the seismic displacements of soil and pile are compatible at points of contact along the pile. These moments, which can be captured by a full dynamic analysis, can be significant in layered soils with soft or liquefiable layers, especially for large diameter piles. Finally the effects of high pore water pressures and liquefaction on the base moments and shears are treated very approximately. The effects of the neglected factors on pile design vary with the intensity of shaking, site conditions and the details of the pile foundation. As will be seen later, sometimes these factors are important and sometimes not. Intelligent use of such approximate methods requires a good understanding of how pile foundations behave during earthquakes. The prime objective of this paper is to provide such an understanding.

A more realistic computational model that is still relatively simple to use is the dynamic Winkler model in Fig. 5 [14]. The free field accelerations may be computed using a 1-D program such as SHAKE [15] and applied to the ends of the near field springs. This ensures that the kinematic interaction of the vibrating ground with the pile is taken into account approximately. The problem with this method is that the reliability of the p-y curves used in practice for dynamic analysis has not been established.

Finn and Thavaraj [16] have shown that a dynamic analysis version of the Winkler model using cyclic p-y curves may prove quite unreliable for seismic response analysis during strong shaking on the basis of centrifuge tests on model piles in dry sand. Several investigators have studied the applicability of the standard North American p-y curves to pile foundations in liquefiable soils and found them unsatisfactory also [17-20]. To take the effects of high pore water pressures into account, the p-y cures were degraded by multiplying the ordinates by a factor p, called the p-multiplier which ranged in value from 0.3 to 0.1 [17-19]. While it was possible to calibrate the p-y curves for a specific test [7], it was not possible to develop a general curve that could be used for all tests [20].

An alternative to the Winkler type computational model is to use a finite element continuum analysis based on the actual soil properties. Dynamic nonlinear finite element analysis in the time domain using the full 3-dimensional wave equations is not feasible for engineering practice at present because of the time needed for the computations. However, by relaxing some of the boundary conditions associated with a full 3-D analysis, Finn and Wu [21] and Wu and Finn (22,23) found it possible to get reliable solutions for nonlinear response of pile foundations with greatly reduced computational effort. The results are accurate for excitation due to horizontally polarized shear waves propagating vertically. Wu and Finn [22,23] give a full description of this method and of numerous validation studies. The method is incorporated in the computer program PILE-3D. An effective stress version of this program, PILE-3D-EFF that can generate and incorporate seismic pore water pressures, has been developed by Thavaraj and Finn [24] and validated by Finn et al [25] and Finn and Thavaraj [16], in cooperation the geotechnical group at the University of California at Davis.

Seismic response analysis is usually conducted assuming that the input motions are horizontally polarized shear waves propagating vertically. The PILE-3D model retains only those parameters that have been shown to be important in such analysis. These parameters are the shear stresses on vertical and horizontal planes and the normal stresses in the direction of shaking. The soil is modeled by 3-D finite elements as shown in Fig. 6. The pile is modeled using beam or volume elements.

The pile is assumed to remain elastic, though cracked section moduli are used for concrete piles, when displacements exceed specified threshold values. This assumption is in keeping with the philosophy that the structural elements of the foundation should not yield. This requirement cannot always be met. If the pile shaft is projected upwards prismatically to act as a column, then any yielding is likely to occur in the buried portion of the shaft.

The constitutive soil model is equivalent linear with strain dependent shear modulus and damping. The strain dependence relations developed by Seed and Idriss [26] were used in the analyses described later. The equations of motion are formulated in the time domain. This allows the modulus and damping to be updated continually during earthquake shaking to maintain compatibility with shear strain level for the duration of analysis. A yield condition

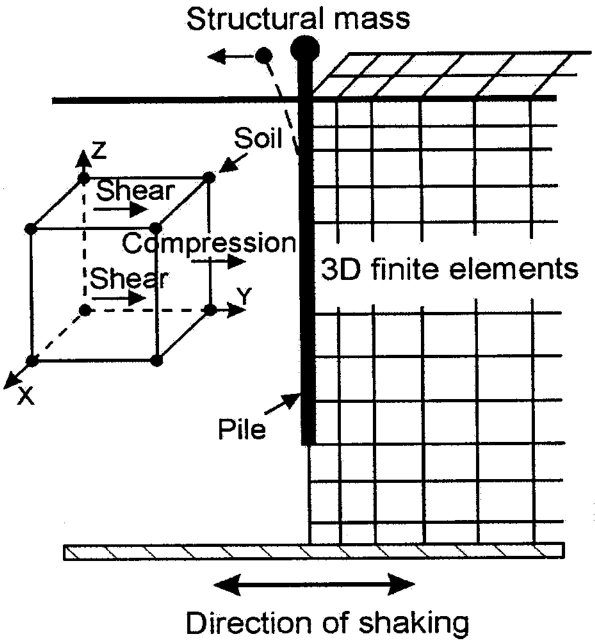

Fig. 6: Soil-pile model for analysis.

is incorporated consistent with the shear strength of the soil and no tension is allowed to develop between the soil and the pile.

A comprehensive picture of the behavior of pile foundations during earthquakes and how pile foundations affect structural response will be developed by detailed analyses of specific practical examples. The behavior of pile foundations in non-liquefiable soils will be examined in the context of the seismic response of a bridge on pile foundations.

5.0 SEISMIC RESPONSE ANALYSIS OF AASHTO (1983) CODE BRIDGE

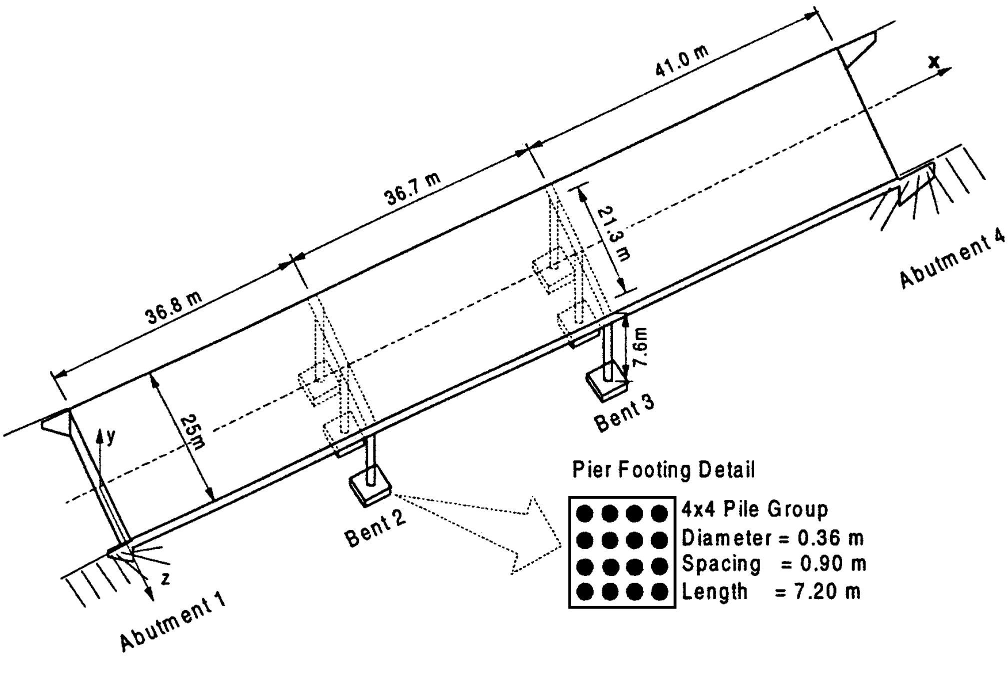

Fig. 7: Three span box girder bridge on pile foundations (after (27)).

A three span continuous box girder bridge structure, shown in Fig. 7, was chosen for the numerical studies of pile foundations in non-liquefiable ground. A rigid base version of this bridge is used as an example in the guide to the seismic design of bridges published by the American Association of State and Transportation Highway Officials [27]. The sectional and physical properties of the superstructure and the piers were taken from [27].

Each pier is supported on a group of sixteen (4×4) concrete piles. The diameter and length of each pile are 0.36 m and 7.2 m respectively. The piles are spaced at 0.90 m, center to center. The Young's modulus and mass density of the piles are E = 22,000 MPa and ρ = 2.6 Mg m^{-3} respectively.

The soil beneath the foundation is assumed to be a nonlinear, hysteretic continuum with unit weight, γ = 18 kNm^{-3} and Poisson's ratio, μ = 0.35. The low strain shear modulus of the soil varies as the square root of the depth with values of zero at the surface and 213 MPa at 10 m depth. The variations of shear moduli and damping ratios with shear strain are those recommended by Seed and Idriss [26] for sand. The surface soil layer overlies a hard stratum at 10 m. For the Pile-3D finite element mesh, the foundation soil was divided into 10 sub-layers of varying thicknesses. The thickness decreased towards the surface where soil-pile interaction effects are stronger. Brick elements were used to model the soil around the piles and beam elements were used to model the piles.

The input acceleration record used in the study was the first 20 seconds of the N-S component of the free field accelerations recorded at CSMIP Station No.89320 at Rio Dell, California during the April 25, 1992 Cape Mendocino Earthquake. The power spectral density of this acceleration record shows that the predominant frequency of the record is approximately 2.2 Hz.

6.0 PILE CAP STIFFNESSES

The pile cap stiffnesses of the pile foundation shown in Fig. 7 will be determined for two different ratios of the column/foundation stiffness ratio, 7% and 50%. A PILE 3-D analysis is conducted first and the spatially varying time histories of modulus and damping are stored. Then an associated program PILIMP calculates the time histories of dynamic pile head impedances using the stored data. The dynamic complex impedances are calculated at any desired frequency by applying a harmonic force of the same frequency to the pile head and calculating the generalized forces for unit generalized displacements. In this paper, discussion will focus on the stiffnesses, the real parts of the complex impedances, as these are the parameters of primary interest for current practice. The stiffnesses are calculated first without taking into account inertial interaction between the superstructure and the pile foundation. This is the usual condition in which stiffness is estimated either by elastic formulae, static loading tests, or static analysis. The stiffnesses are calculated also taking the inertial effects of the superstructure into account. In this latter case, both kinematic and inertial interactions are taken into account at the same time. Since the entire pile group is being analyzed, pile-soil-pile interaction is automatically taken into account under both linear and non-linear conditions. Therefore the usual difficult problem of what interaction factors to use or what group factor to apply is avoided.

Time histories of lateral and cross coupling stiffnesses are shown in Figure 8; rotational stiffness in Fig. 9. These stiffnesses, resulting from kinematic interaction only, were calculated for the predominant frequency of the input motions, f = 2.2 Hz. It is clearly not an easy matter to select a single representative stiffness to characterize a discrete single valued spring to be used in structural analysis programs to represent the effects of the foundation. In the absence of a nonlinear analysis, probably a good approach to including the effects of soil nonlinearity on stiffness is determine the vertical distribution of effective moduli using a

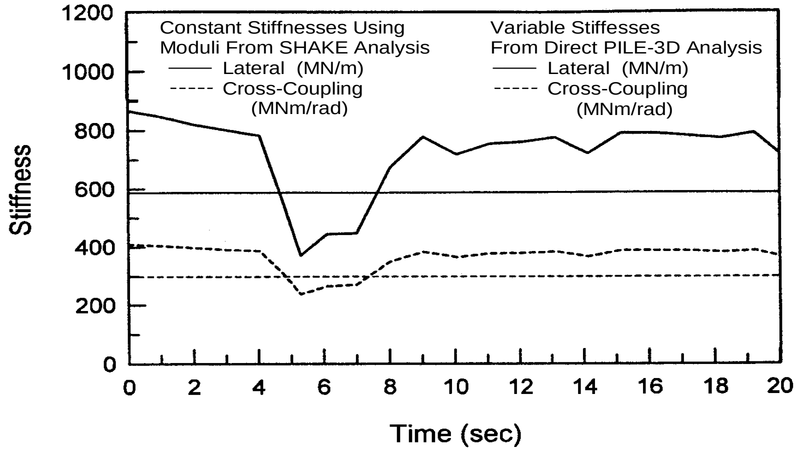

Fig. 8: Time history of lateral and cross-coupled stiffness under strong shaking.

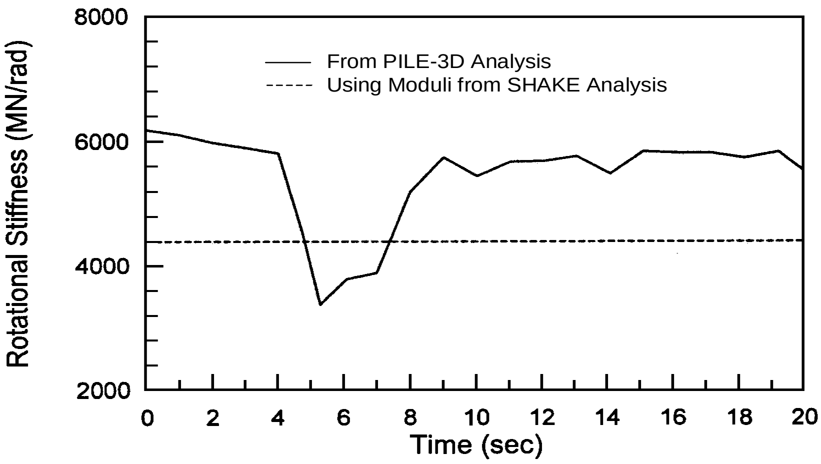

Fig. 9: Time history of rotational stiffness under strong shaking.

SHAKE [15] analysis of the free field and calculate the stiffnesses at the appropriate frequency using these moduli. The constant stiffnesses calculated in this way are shown also in Figs. [8] and [9]. These are kinematic stiffnesses. .

7.0 RESPONSE OF CODE BRIDGE TO TRANSVERSE EARTHQUAKE LOADING

7.1 Finite Element Model of the Bridge Structure

A three dimensional space frame model of the bridge is shown in Fig. 10. At the abutments, the deck is free to translate in the longitudinal direction but restrained in the transverse and vertical directions. Rotation of the deck is allowed about all three axes. The space frame members are modeled using 2-noded 3-D beam elements with twelve degrees of freedom, six degrees at each end. The bridge deck was modeled using 13 beam elements and each pier was modeled by 3 beam elements. The cap beam that connects the tops of adjacent piers was modeled using a single beam element. The sectional and physical properties of the deck and the piers are those provided in the AASHTO guide [27]. The pier foundation is modeled

using a set of time-dependent nonlinear springs and dashpots that simulate exactly the time histories of stiffnesses and damping from the PILE-3D analyses.
The response of the bridge structure was analyzed for different foundation conditions to study the influence of various approximations to foundation stiffnesses and damping using the computer program BRIDGE-NL [28].

The free field acceleration was used as the input acceleration and the peak acceleration was set to 0.5g.

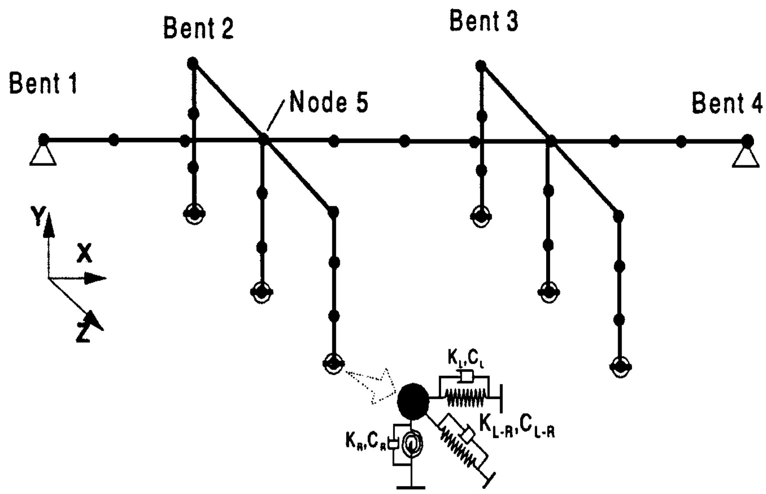

Fig. 10: Stick model of the bridge with the foundation springs and dashpots.

7.2 Foundation Conditions for Analyses

The seismic response of the bridge to transverse earthquake loading was analyzed for the four different foundation conditions listed below.
1. Rigid foundation and fixed base condition is assumed
2. Flexible foundation with elastic stiffness and damping
3. Flexible foundation with kinematic time dependent stiffness and damping
4. Flexible foundation with stiffness and damping based on the 'SHAKE' effective moduli.

The fundamental transverse mode frequency of the computational model of the bridge with a fixed base was found to be 3.18 Hz. This is the frequency quoted in the AASHTO-83 guide [27]. This agreement in fundamental frequencies indicates an acceptable structural model. In this analysis, the lateral stiffness of the bridge pier is only 7% of the foundation stiffness. For this extremely low stiffness ratio, the columns control the fundamental frequency of the bridge and the influence of the foundation is negligible. Results from analyses in which the column/foundation stiffness ratio is 50% will be presented here. The stiffness ratio was raised by increasing the stiffness of the piers only, with no changes to the super-structure. Normally much stiffer piers would imply a heavier superstructure and therefore higher inertial forces.

For a 50% stiffness ratio, the fixed base fundamental frequency of the bridge is 5.82 Hz. When the stiffnesses associated with low strain initial moduli are used, the fundamental frequency is 4.42 Hz, a 24% reduction from the fixed base frequency. With kinematic strain dependent stiffnesses, the frequency reached a minimum value of 3.97 Hz during strong shaking, a 32% reduction from the fixed base frequency. When the foundation stiffnesses are based on effective shear moduli from a 'SHAKE' analysis of the free field, the frequency is 4.18 Hz, a 28% change from the fixed base frequency. Fig. 11 shows the variation with time in fundamental transverse modal frequency for the different foundation conditions.

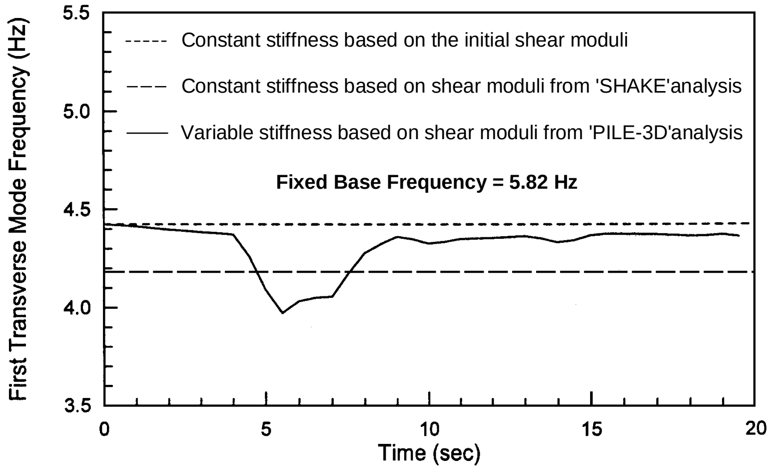

Fig. 11: Time history of the frequency of first transverse mode.

The response of the bridge deck at Bent 2 (Node No. 5 in Fig. 10) was computed for two cases: the fixed base case and a flexible foundation with kinematic time dependent stiffnesses. The effect of including the foundation flexibility is shown in Fig. 12. There is a dramatic change in the deck displacement during the strong shaking, when the foundation flexibility is included in the model. The peak displacement increased from 7mm to 17mm.

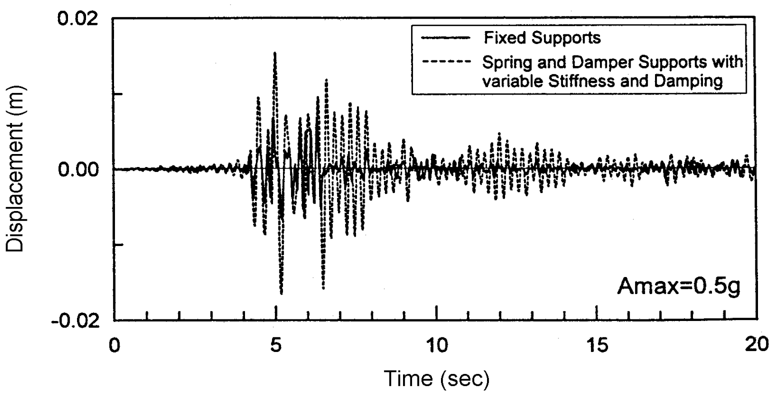

Fig. 12: Effects of foundation flexibility on deck displacement.

8.0 INERTIAL INTERACTION OF STRUCTURE AND PILE

The time dependent stiffnesses used in the analyses described above were computed without taking the inertial interaction of superstructure and foundation into account. The primary effect of this interaction is to increase the lateral pile displacements and cause greater strains in the soil. This in turn leads to smaller moduli and increased damping. The preferred method of capturing the effect of superstructure interaction is to consider the bridge structure and the foundation as a fully coupled system in the finite element analysis. However, such a fully coupled analysis is not possible with current commercial structural software. Even if it were, it would be not be feasible in practice because it would require requires enormous amounts of computational storage and time.

An approximate way of including the effect of superstructure interaction is to use the model shown in Fig. 13. In this model, the superstructure is represented by a single degree of freedom (SDOF) system. The mass of the SDOF system is assumed to be the portion of the superstructure mass carried by the foundation. The stiffness of the SDOF system is selected so that the system has the period of the fixed base bridge structure in the mode of interest.

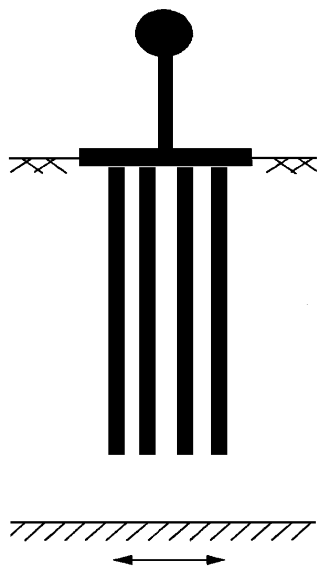

Fig. 13: Pile foundation with structure.

This approximate approach will be demonstrated by the analysis of the center pier at Bent 2. The fundamental transverse mode frequency of the fixed base model was found earlier to be 5.82Hz. The static portion of the mass carried by the center pier is 370 Mg. The superstructure can be represented by a SDOF system having a mass of 370 Mg at the same height as the pier top and frequency 5.82Hz. The corresponding stiffness of the SDOF system is 495 MN/m.

A coupled soil-pile-structure interaction analysis can be carried out using PILE3-D by incorporating the SDOF model into the finite element model of the pile foundation. The pile foundation stiffnesses derived from this finite element model incorporate the effects of both inertial and kinematic interactions and are called total stiffnesses. The time histories of stiffnesses with and without the superstructure are shown in Fig. 14. The reduction in lateral stiffness is greater throughout the shaking when the inertial interaction is included. There is a similar reduction in the rotational and cross-coupling stiffnesses.

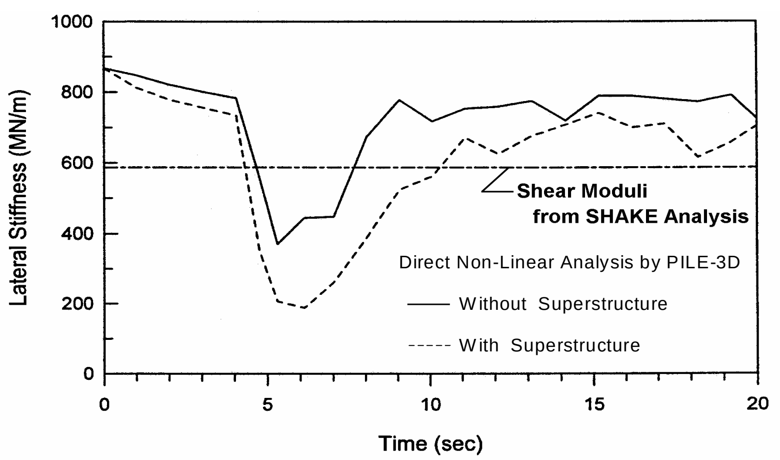

Fig. 14: Effects of inertial interaction on lateral pile cap stiffness.

When inertial interaction is included, the lateral stiffness reached a minimum of 188 MN/m which is 78% lower than the initial value. This minimum was 20% lower than the minimum that was attained, when the inertial interaction was not included. This analysis probably underestimates the effects of inertial interaction because the column stiffness of the AASHTO code bridge was increased from 7% to 50% without any increase in superstructure mass. Such a stiffness ratio would normally be associated with a heavier super-structure.

An eigenvalue analysis of the complete bridge structure was carried out, using the total foundation stiffnesses. The variation in first mode transverse frequency with time is shown in Fig. 15. This figure also shows the frequency variation for the case in which the inertial interaction was not considered. The frequency reached a minimum of 3.62Hz, when the inertial interaction was included and 3.97Hz, when the interaction was ignored. Figs. 16 and 17 show the effects of superstructure interaction on the time histories of acceleration and displacement respectively. For this particular case, when the superstructure interaction effect is included, it leads to greater acceleration and displacement. The increase in peak displacement is approximately 72%, a major increase.

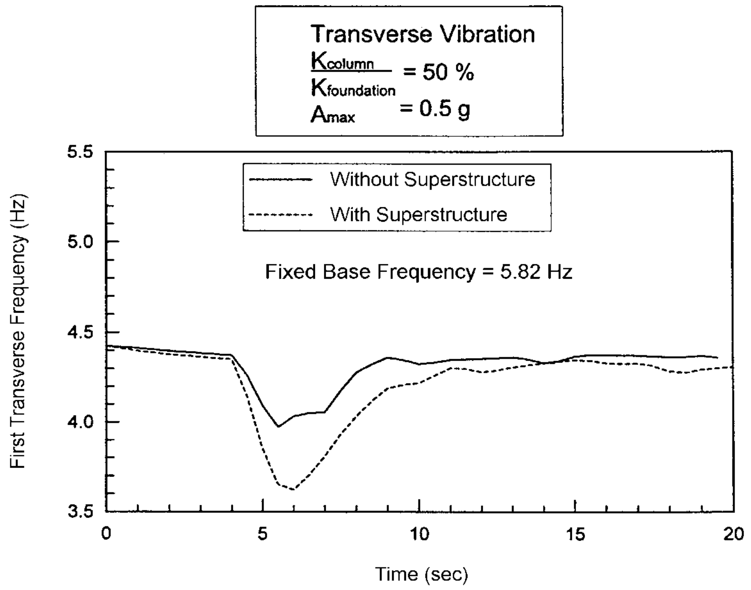

Fig. 15: Effects of inertial interaction on foundation frequency.

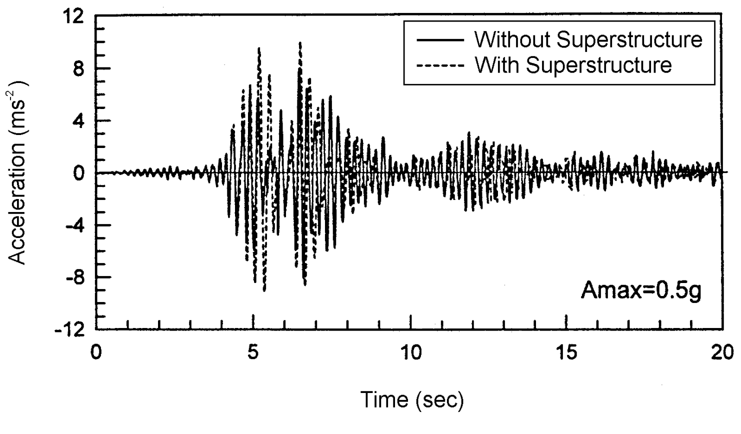

Fig. 16: Effect of superstructure interaction on deck acceleration.

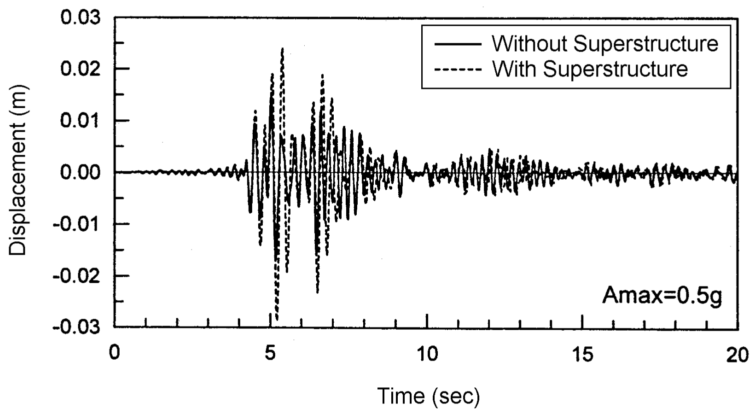

Fig. 17: Effect of superstructure interaction on deck displacement.

The results of the analyses for the four different foundation conditions are summarized in the acceleration and displacement spectra for transverse vibration of the bridge, shown in Figs. 18 and 19 respectively. The fixed base model for estimating response is clearly inadequate in this case. If the effects of inertial interaction are neglected, and only kinematic stiffness are taken into account, the seismic response obtained using effective moduli from a SHAKE analysis of the free field gives almost identical results to the PILE-3D response. However when the inertial interaction is included, there is significant difference in response.

9.0 PILE CAP STIFFNESSES AND SYSTEM FREQUENCIES

This study has shown that different approximations to foundation conditions of a bridge and in the evaluation of pile cap stiffnesses can make large differences in the estimated pile cap stiffness matrix. These differences will affect the mode frequencies of the bridge foundation system. It is these system frequencies that control response. The impact that pile cap stiffnesses have on system frequencies depends on the relative stiffnesses of the superstructure supports and the pile foundation. This effect can be estimated by the period shift in the first mode frequency. A parametric study was conducted to define the dependence of period shift on relative superstructure/ pile cap stiffness. The results are shown in Fig. 20, where the non-dimensional period ratio, T_P/T_F, is plotted against the non-dimensional stiffness ratio, $K^S{}_P/K^F{}_L$. In these ratios, T_P is the system period for a fixed base,

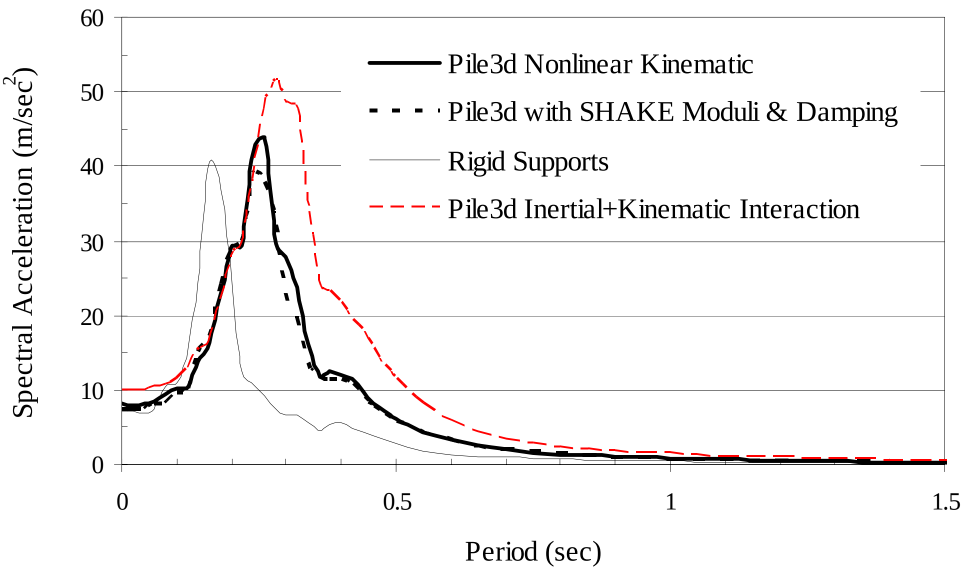

Fig. 18. Spectral accelerations of AASHTO bridge for four different approximations to foundation conditions.

T_F is the system period for a flexible base, $K^S{}_P$ is the superstructural lateral stiffness and $K^F{}_L$ is the lateral stiffness of the pile foundation.

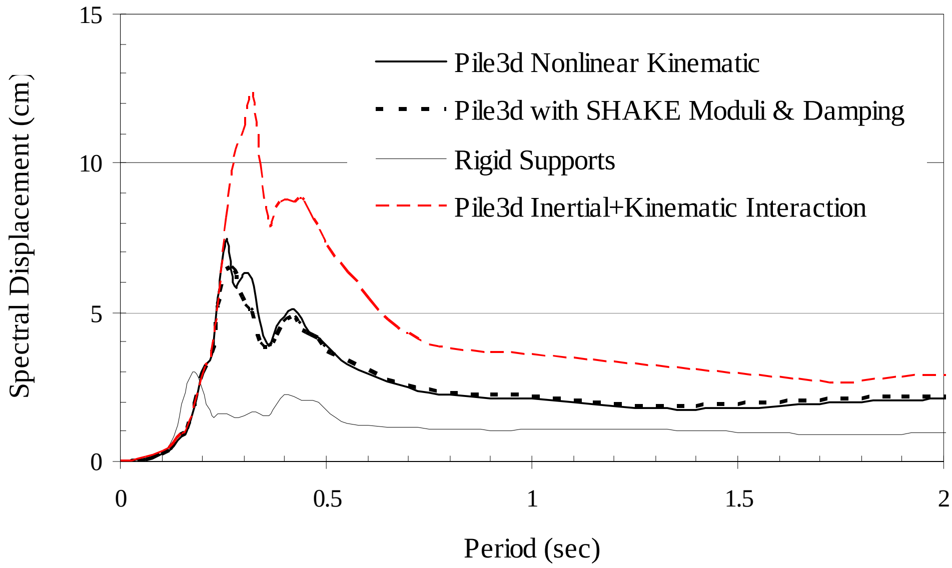

Fig. 19. Spectral displacements of AASHTO bridge for four different approximations to foundation conditions.

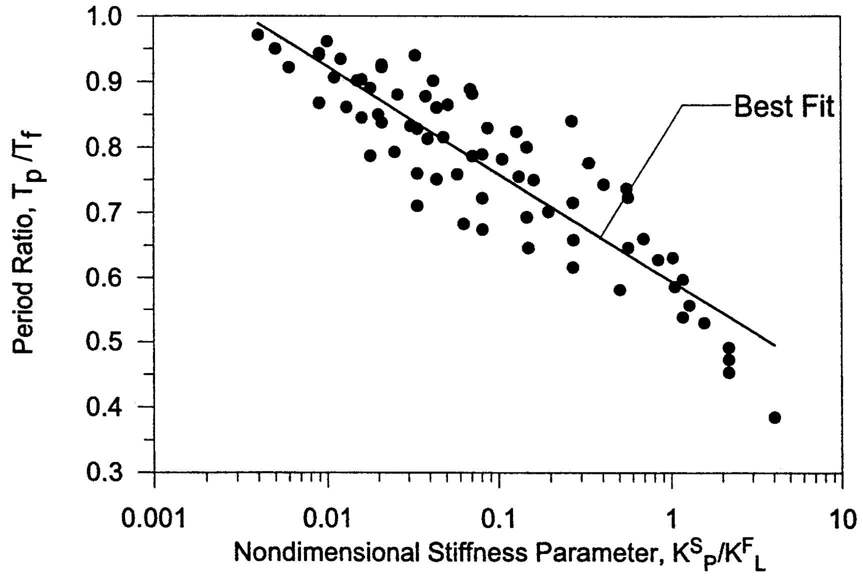

Fig. 20. Period shift for bridge-foundation system as a function of relative superstructure – foundation lateral stiffness.

10.0 CLOSING REMARKS

Performance based design demands an appropriate computational model of the structure in order to check whether the design meets the performance criterion. This requirement is a challenge for geotechnical engineers to provide effective models of the actions of foundations on the structure during shaking. A constraint on such modeling is that the models must be compatible with the commercial software used in structural design practice.

The paper describes the modeling of shallow foundations as recommended by NEHRP (1997). NEHRP recommends Winkler spring models. The stiffnesses of these models are based for the most part on stiffness formulations related to rigid footing. Large mat footings may have flexibilities that make the rigidity assumption invalid.

The new development with a major impact on the modeling of shallow foundations is the acceptance of footing uplift, with or without soil yielding. To accommodate uplift and soil yielding, the Winkler springs must not be allowed to develop tension and must have compression stress limits that match the yield pressure of the soil. Reaching the yield capacity moment of a foundation is not synonymous with failure because of the ductility capacity of soil. The real indices of failure are the increased displacements and their effects on global stability and on seismic demand.

The reliable modeling of pile foundations in a manner suitable for design requires simplified methods but there is very little hard numerical data from past earthquakes to validate current methods. Also because the fully coupled analysis of pile groups is difficult and time consuming with commercial software not many analytical investigations of pile foundations have been conducted for realistic soil properties.

The reliability of approximate methods for representing the rotational and translational stiffnesses of pile foundations in the computational structural model of a superstructure are investigated in the paper, using a pseudo-3-D nonlinear continuum soil model. The study is focused on a 4x4 pile group supporting a bridge pier.

Some of the assumptions of the approximate methods in use for evaluating foundation stiffnesses were incorporated into 3-D nonlinear analyses of the foundations and the foundations-bridge system. Most of the approximate methods in use are based on single pile analysis and further assumptions must be made to establish the group response. They often neglect both the kinematic interaction between pile and foundation soils and inertial interaction between superstructure and foundations. The problems in selecting appropriate single valued springs to represent the actions of pile foundations on a superstructure are illustrated by time histories of pile cap stiffnesses during strong earthquake shaking. The consequences of using various approximations to pile cap stiffnesses are investigated by examining their effects on the first modal frequencies of the pile foundation - bridge system and on the accelerations and displacements of the deck. The effects of ignoring inertial or kinematic interactions, as some methods do, are also evaluated. The effects of inertial interaction can be very significant in reducing the stiffness factors and hence the frequency of the pile foundation–bridge system.
Parametric studies are continuing in order to provide a larger data base for a comprehensive evaluation of the many approximate methods in use for the evaluation of pile stiffnesses.

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