13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 5002

Characterizing Pile Foundations for Evaluation of Performance Based Seismic Design of Critical Lifeline Structures

W. D. Liam FINN [1]

SUMMARY

The actions of a pile foundation on a structure are represented in commercial structural software by stiffness coefficients. These are usually calculated by approximate methods that typically neglect one or more of the important factors that affect seismic response of a structure such as inertial interaction, kinematic interaction, seismic pore water pressures, soil nonlinearity, cross stiffness coupling and dynamic pile to pile interaction. A nonlinear 3-D analysis is used to calculate time histories of total stiffnesses and structural frequencies for a typical bridge during an earthquake to demonstrate some of the consequences of using various approximate methods.

INTRODUCTION

Performance based design of structures is design for specified, acceptable levels of damage. In order to deliver the expected performance at competitive cost, it is essential to be able to evaluate reliably the performance of a proposed design. The evaluation may be done using a nonlinear dynamic response analysis. The value of such an analysis depends on how well the computational model represents the structure-foundation-soil system. Therefore all significant factors that affect seismic demand on the structure should be included. Usually the effects of pile foundations on a structure are modeled by discrete, single valued springs that represent rotational and linear stiffnesses and any coupling between these springs is often ignored. The spring stiffnesses are frequently estimated using approximate, simplified methods of variable 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 non-linear soil response under strong earthquake shaking, 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

[1] Anabuki Professor, Kagawa University, Takamatsu, Japan: finn@eng.kagawa-u.ac.jp

and demonstrates the effects of kinematic and inertial interactions, pore water pressures and soil nonlinearity. A prime benefit of such an analysis, in addition to its 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.

The purpose of this paper is to present a comprehensive overview of the behavior of pile foundations during earthquakes in both liquefiable and non-liquefiable soils using non-linear dynamic effective stress continuum analysis. It is hoped that the overview may provide a better understanding of the actions of pile foundations on structures, and therefore a better appreciation of the limitations of approximate methods for the design of pile foundations and for the characterization of the actions of pile foundations on structures in computational models for structural response analysis.

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 inertial contribution from the superstructure. The analysis is done usually for a single pile and the group stiffnesses are evaluated using pile interaction factors, often derived from static analysis, or an empirical group reduction factor.

The most common approach to such an analysis is to use a Winkler spring computational model. A general Winkler model is shown in Fig. 1 which can be used for static or dynamic analysis. For static analysis, only the pile and the near field springs are used.

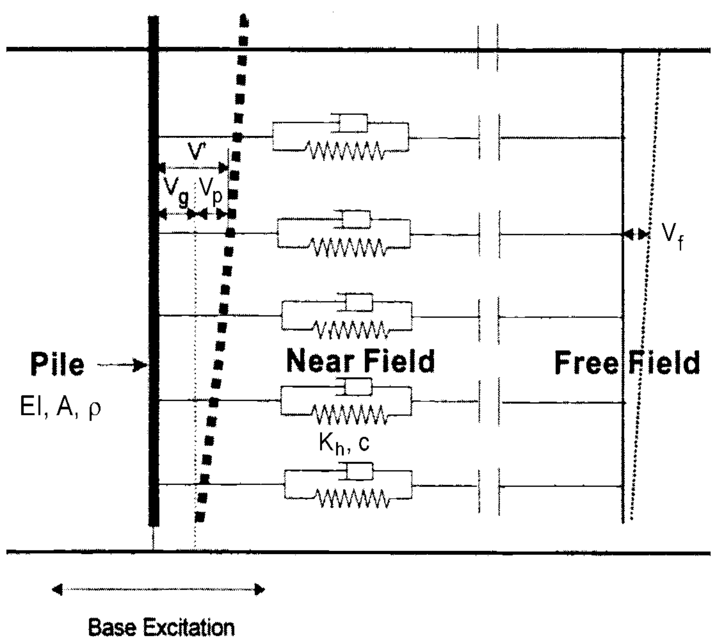

Fig. 1: Dynamic Winkler computational model for pile analysis.

The springs may be elastic or nonlinear. Some organizations, such as the American Petroleum Institute [1], 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 [2] suggest that the reliability of the Winkler (p-y) model may not be high.

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 are 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 in the superstructure. 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 very significant. 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 the static method requires a good understanding of how pile foundations behave during earthquakes.

A more realistic computational model that is still relatively simple to use is the dynamic Winkler model in Fig. 1[3]. The free field motions are computed using a 1-D program such as SHAKE [4] 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 [5] 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 [6-9]. 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 [6-8]. 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 [9].

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 [10] 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 [11, 12] 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, has been developed by Thavaraj and Finn [13] and validated by Finn et al [14] and Finn and Thavaraj [5] in cooperation with 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. 2. The pile is modeled using beam or volume elements. The program handles foundation rocking by alternating between the vertical and horizontal directions. When in the vertical mode, the program calculates the rocking stiffness from the current soil properties and returns to the horizontal mode with the updated rocking stiffnesses.

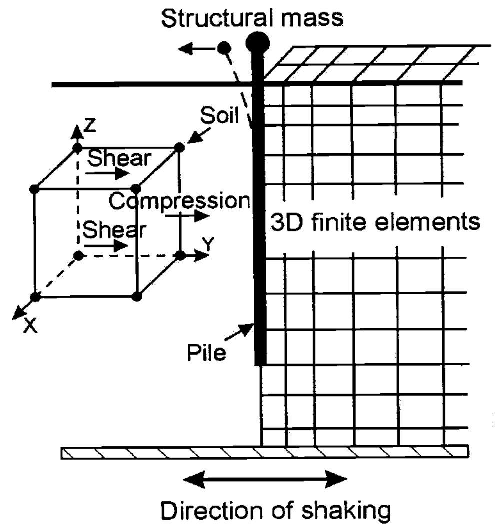

Fig. 2: Soil-pile model for analysis.

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 [15], and shown in Fig. 3, 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 is incorporated consistent with the shear strength of the soil and no tension is allowed to develop between the soil and the pile, allowing gapping to occur when appropriate.

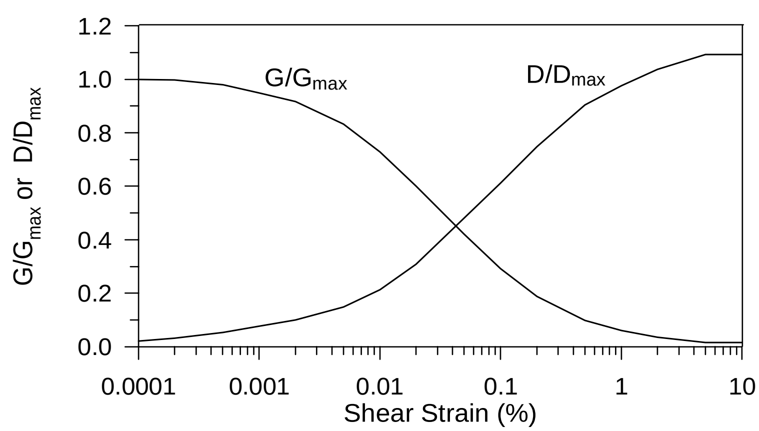

Fig. 3: Strain dependence of moduli and damping after Seed and Idriss [15].

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. The behavior of piles in liquefiable soils will be demonstrated by analyses of the seismic response of a 14-storey apartment building with columns supported on individual cast-in-deep-hole (CIDH) reinforced concrete piles.

SEISMIC RESPONSE ANALYSIS OF AASHTO (1983) CODE BRIDGE

A three span continuous box girder bridge structure shown in Fig. 4 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 Highway and Transportation Highway Officials, (AASHTO) [16]. The sectional and physical properties of the superstructure and the piers were taken from [16].

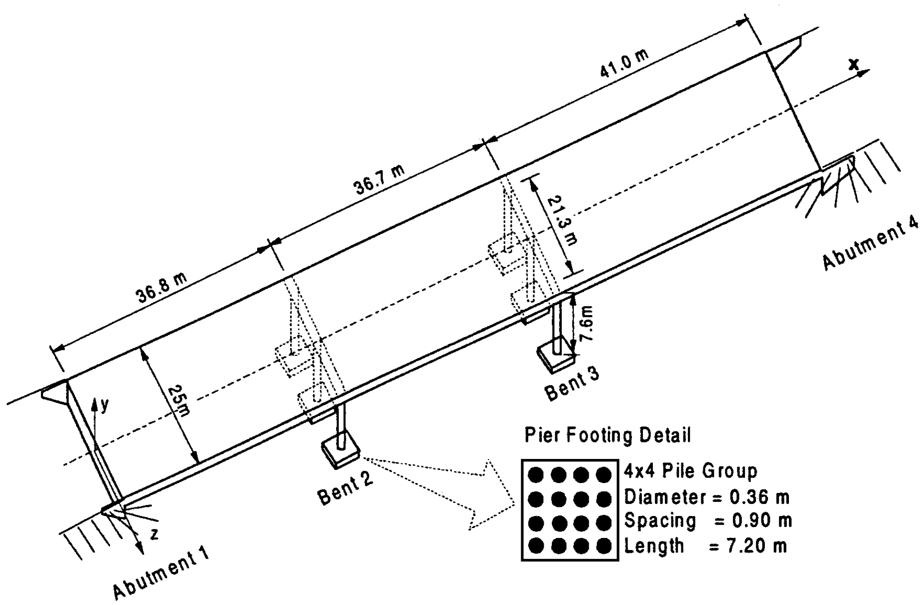

Fig. 4: Three span box girder bridge on pile foundations.

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. As will be shown later, the pile cap stiffnesses of this foundation are much greater than the corresponding column stiffnesses. The large disparity in stiffnesses was selected in order to facilitate a study of the effects of the ratio of column to foundation stiffnesses on the seismic response of the super-structure by increasing the column stiffnesses only.

The soil beneath the foundation is assumed to be a nonlinear hysteric continuum with unit weight, γ = 18 kNm^{-3} and Poisson's ratio, ν = 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 (Fig. 5). The variations of shear moduli and damping ratios with shear strain are those recommended by Seed and Idriss [15] for sand. The surface soil layer overlies a hard stratum at 10 m. For the PILE-3D finite element mesh, it was divided into

10 sub-layers of varying thicknesses. Sub-layer thicknesses decrease towards the surface where soil-pile interaction effects are stronger. 900 brick elements were used to model the soil around the piles and

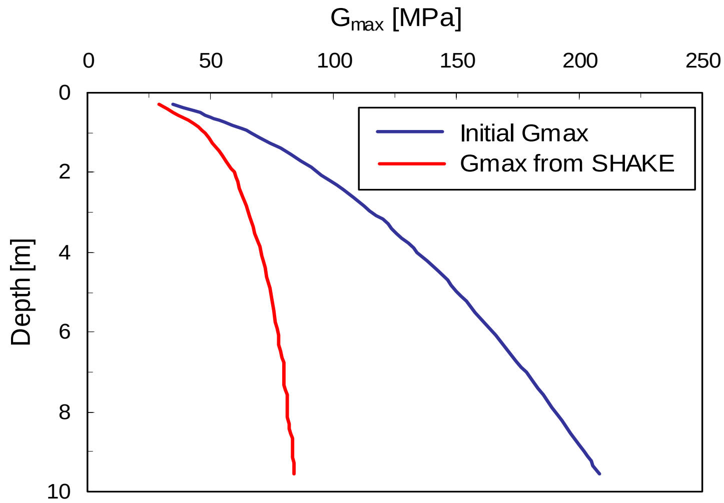

Fig. 5: Distribution of initial moduli at site of AASHTO (1983) bridge and of effective moduli from a SHAKE free-field analysis.

64 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.

PILE CAP STIFFNESSES

The pile cap stiffnesses of the pile foundation shown in Fig. 4 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 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 displacements. In this paper the focus will be on the stiffnesses only as these are the parameters of primary interest for current practice. However the effects of damping are always included in the analyses. 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 static loading tests, static analysis or by elastic formulae. 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. 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 6; rotational stiffness in Fig. 7. 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 the discrete single valued springs used in structural analysis programs to represent the effects of the foundation. In the absence of a complete analysis, probably a good approach to including the effects of soil nonlinearity on stiffness is to get the vertical distribution of effective moduli by a SHAKE [4] analysis of the free field and calculate the stiffnesses at the appropriate frequency using PILIMP with these moduli. The constant stiffnesses calculated in this way are shown also in Figs. 6 and 7. However these are kinematic stiffnesses. Later it is shown that inertial interaction by the superstructure may cause greater non-linear behavior leading to substantially reduced frequencies. A SHAKE analysis cannot capture this effect.

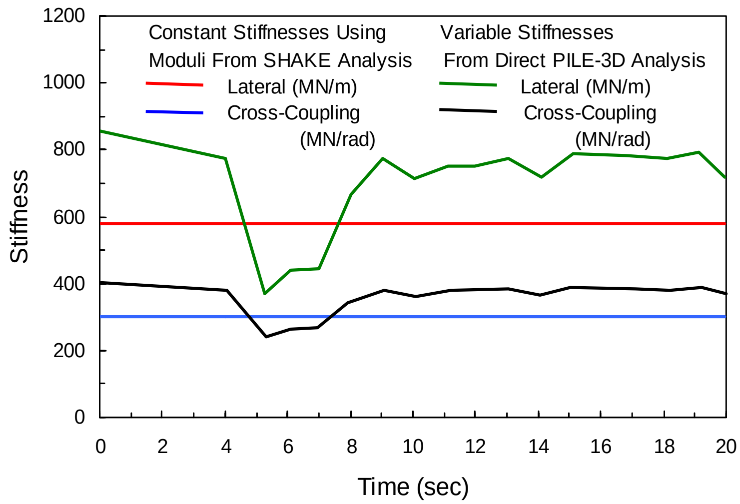

Fig. 6: Time history of lateral and cross-coupled stiffness under strong shaking.

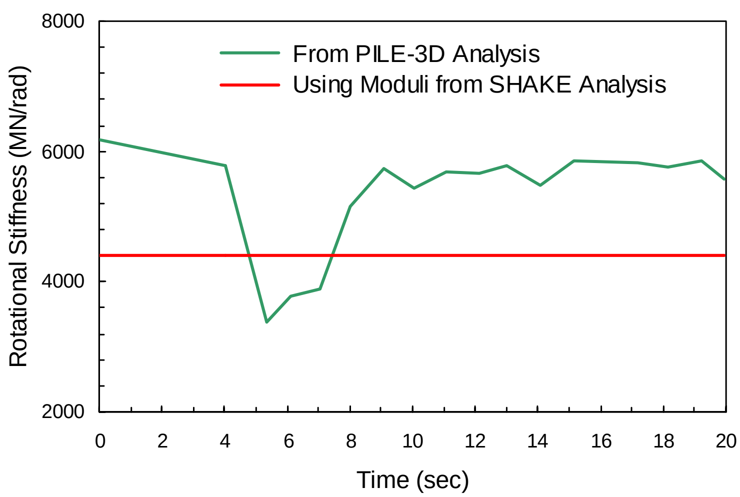

Fig. 7: Time history of rotational stiffness under strong shaking.

SEISMIC RESPONSE OF CODE BRIDGE TO TRANSVERSE EARTHQUAKE LOADING

Finite Element Model of the Bridge Structure

A three dimensional space frame model of the bridge is shown in Fig. 8. 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 [16]. 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.

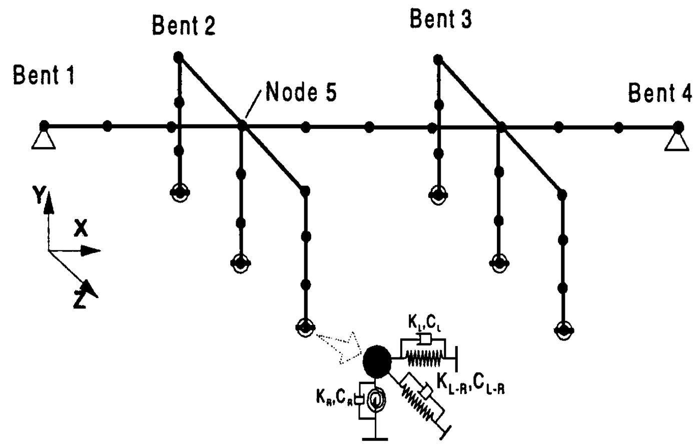

Fig. 8: Stick model of the bridge with the foundation springs and dashpots.

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 [17].

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

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 structure with a fixed base was found to be 3.18 Hz. This is the frequency quoted in the AASHTO-83 guide [16]. 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 4.42 Hz, a 24% reduction from the fixed base frequency. With kinematic strain dependent stiffnesses, the frequency reached to 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 was 4.18 Hz, a 28% change from the fixed base frequency. Fig. 9 shows the variation with time in fundamental transverse modal frequency for the different foundation conditions.

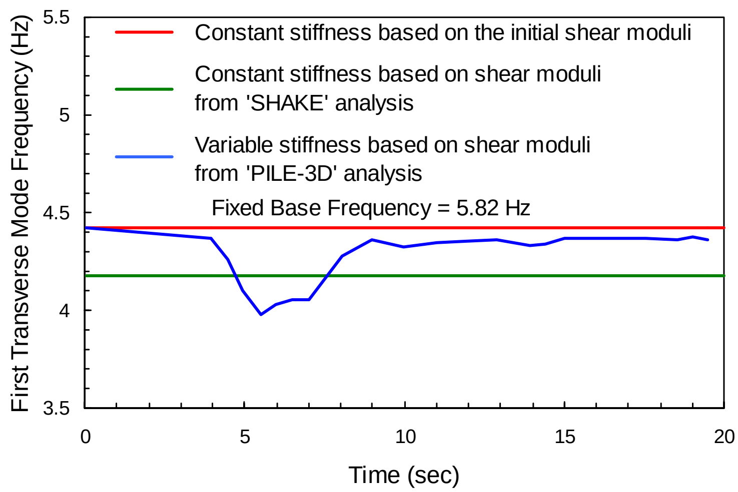

Fig. 9: Time history of the frequency of first transverse mode.

The response of the bridge deck at Bent 2 (Node No. 5 in Fig. 8) 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. 10. 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. An elastic analysis of the displacement response of the deck at Bent 2 was carried out also using stiffnesses calculated by PILIMP using the low strain initial moduli of the site. The elastic response underestimates the displacements based on the kinematic stiffnesses but is a big improvement over the fixed base solution.

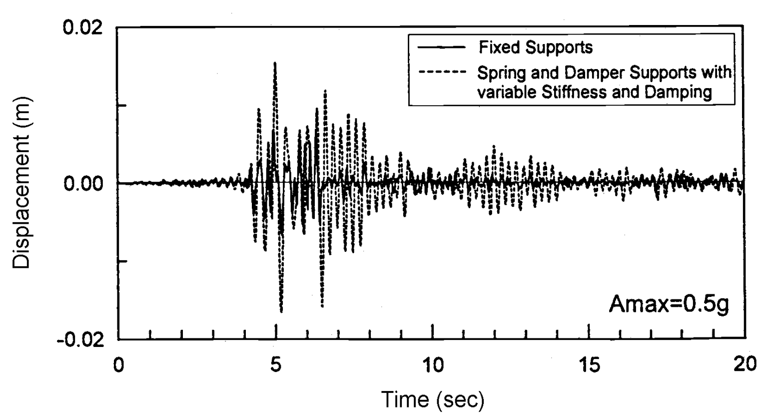

Fig. 10: Effects of foundation flexibility on deck displacements.

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 fully a coupled analysis is not possible with current commercial structural software. Even if it were, it would not be feasible in practice because it would require 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. 11. 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 mode of interest of the fixed base bridge structure.

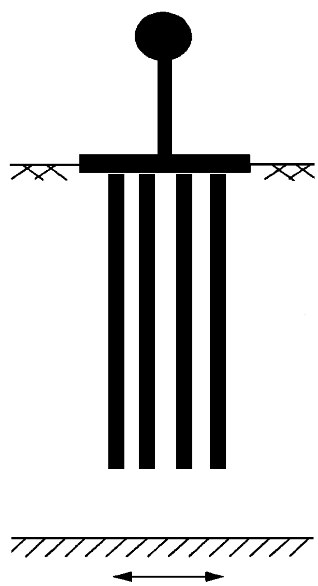

Fig. 11: Pile foundation with structure.

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 PILE-3D 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. 12. 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.

When inertial interaction is included, the lateral stiffness reached a minimum of 188 MN/m which is 22% of the initial value. When the inertial interaction was not included, the minimum was 400 MN/m. Clearly in this case, inertial interaction has a major effect on foundation stiffness. The effects of inertial interaction may be underestimated somewhat in this case, because the column stiffness of the AASHTO code bridge was increased from 7% to 50% of the foundation stiffness without any increase in superstructure mass. Such a stiffness ratio would normally be associated with a heavier super-structure.

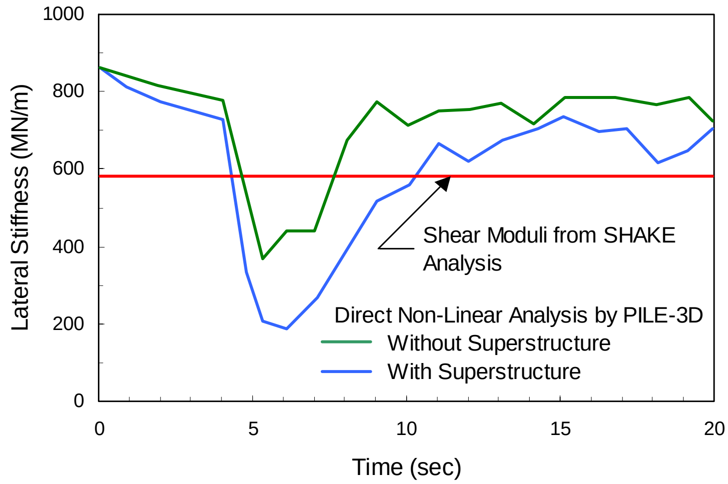

Fig. 12: Effects of inertial interaction on lateral pile cap stiffness.

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. 13. 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. 14 and 15 show the effects of superstructure interaction on the time histories of acceleration and displacement respectively. When the superstructure interaction effect is included it leads to greater accelerations and displacements. There is a major increase of 70% in the peak displacement.

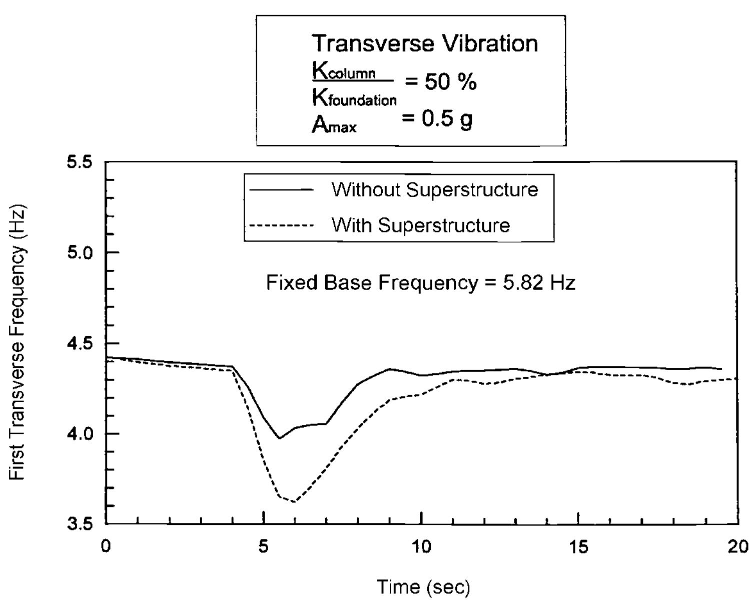

Fig. 13: Effects of inertial interaction on foundation frequency.

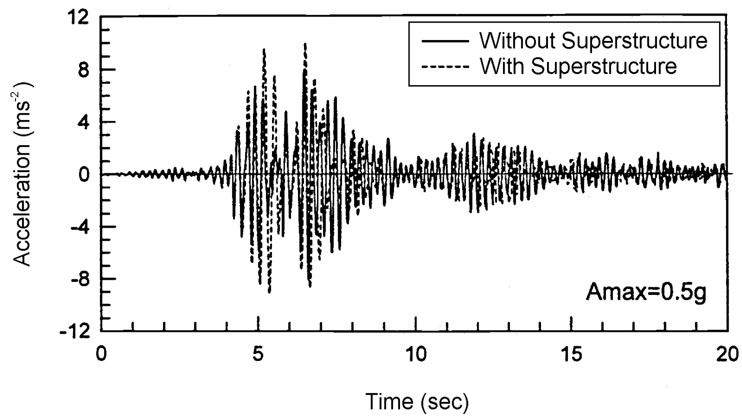

Fig. 14: Effect of superstructure interaction on deck acceleration.

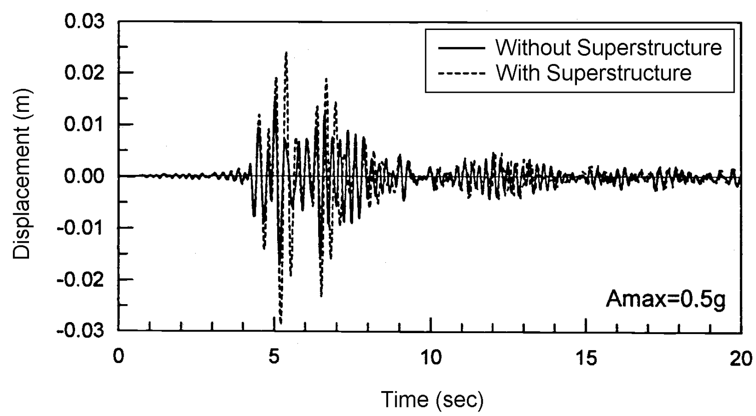

Fig. 15: Effect of superstructure interaction on deck displacement.

The results of the analyses for four different foundation conditions are summarized in the acceleration and displacement spectra for transverse vibrations of the bridge, shown in Figs. 16 and 17 respectively. The displacement spectra clearly show the importance of including inertial interaction, when calculating foundation stiffnesses in this case. The fixed base model for estimating response is inadequate. As the ratio of super-structural stiffness to foundation stiffness is reduced, the effect of inertial interaction on system frequency is reduced and kinematic stiffnesses become adequate. Only for low stiffness ratios is the fixed base model adequate. The role of the stiffness ratio in controlling system frequencies is demonstrated in the next section.

For the example bridge, when effective moduli from a SHAKE analysis of the free field are used in an elastic analysis to obtain a discrete, foundation stiffness for each degree of freedom, the corresponding system frequencies lead to acceleration and displacement responses very close to the responses from a PILE-3D nonlinear analysis. This is true when the complete pile foundation is included in the analysis. It or may not be true, if the effective moduli are used to get the stiffness of a single pile and the stiffness of the pile group is developed from this with the help of empirical factors for group effects. This result suggests that kinematic stiffnesses may be obtained taking non-linear soil effects into account, by an elastic structural program that can model the pile group foundation, if the effective moduli from a SHAKE analysis are used. This needs to be verified by a few more case histories.

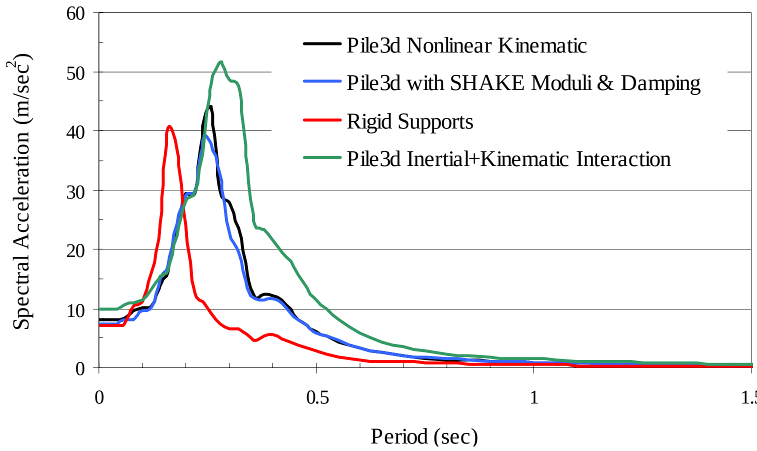

Fig. 16: Spectral accelerations of AASHTO bridge for four different foundation conditions.

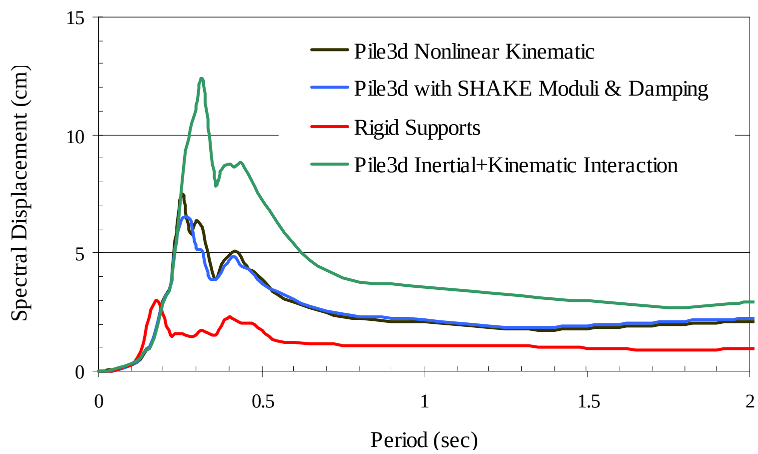

Fig. 17: Spectral displacements of AASHTO bridge for four different approximations to foundation conditions.

PILE CAP STIFFNESSES AND SYSTEM FREQUENCIES

This study has shown that the different approximations made in the evaluation of pile cap stiffnesses can lead to large differences in the estimated pile cap stiffness matrix. It does not follow, however, that there will necessarily be correspondingly large differences in the seismic responses of the structure. Structural response depends on the system frequencies that result from the coupling of foundation and structural stiffnesses. The impact that pile cap stiffnesses have on system frequencies depends on the relative stiffnesses of the superstructure and the pile foundation. This effect can be estimated by the period shift in the first mode frequency.

The extent of the period change depends on the non-dimensional period ratio, T_P/T_F, where T_P is the system period for a fixed base and T_F is the system period for a flexible base. The ratio depends on many factors characterizing the structural system but a dominant factor is the ratio of superstructural stiffness to foundation stiffness in the direction of the degree of freedom of interest. In the lateral mode of translation the ratio is K^S_P/K^F_L, where K^S_P is the superstructural lateral stiffness and K^F_L is the lateral stiffness of the pile foundation.

A parametric study was conducted on a simple two span bridge to define the dependence of period shift on relative superstructure/ pile cap stiffness. The bridge is shown in Fig. 18. It has a single column bent supported by pile group foundations and two seated type abutments. The bridge is similar to one studied by Lam and Martin [18] and Imbsen and Penzien [19]. A computational model of the bridge incorporating translational and rotational stiffnesses of the pile foundations is shown Fig. 19. Two different pile groups were used to support the central pier, a (2x2) and a (3x3). Three different soil conditions were considered; in one case the shear modulus varied parabolically with depth and in the other two cases the moduli were uniform but different. Three different pile/soil modulus ratios, E_p/E_s, were used. The foundation parameters for the study are shown in Table 1. The calculated foundation stiffnesses for the six cases are shown in Table 2. The mass, height and diameter of the pier were also varied. The parameters characterizing the pier and the superstructure are given in Table 3.

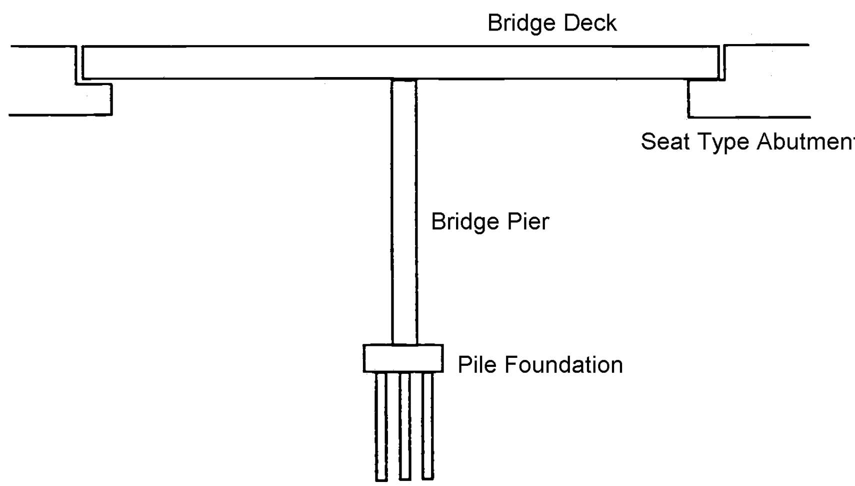

Fig. 18: Two span bridge used in parametric study

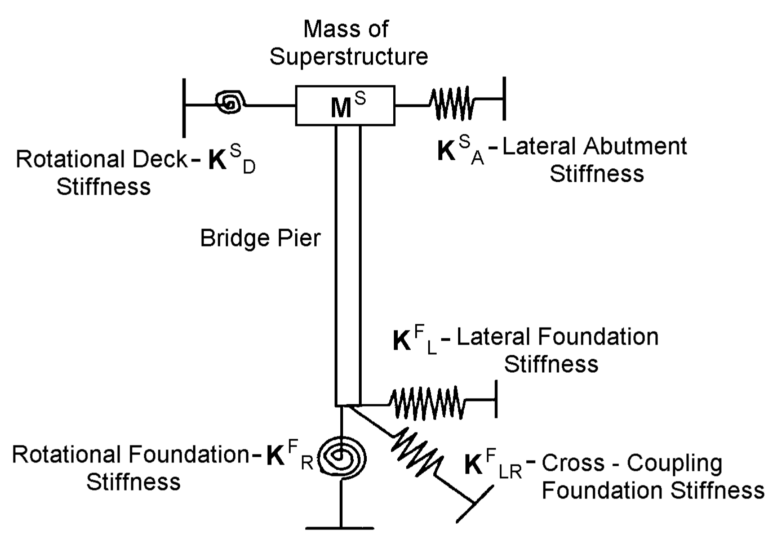

Fig. 19: Computational bridge model for parametric study.

Table 1: Foundation parameters.

Case	No of Piles	G_{max} Profile	Siffness Ratio, E_p/E_s	Spacing Ratio, S/D	Depth Ratio, L/D	Density Ratio, ρ_p/ρ_s
1	(2 x 2)	Uniform	100	5	15	0.7
2	(2 x 2)	Uniform	1000	5	15	0.7
3	(2 x 2)	Parabolic	312	5	15	0.7
4	(3 x 3)	Uniform	100	4	15	0.7
5	(3 x 3)	Uniform	1000	4	15	0.7
6	(3 x 3)	Parabolic	312	4	15	0.7

Table 2: Superstructure and pier parameters.

Parameter	Values
Mass of superstructure, M (Mg)	500, 750, 1000
Young's modulus of pier, E (MPa)	25000
Density of pier, ρ (Mg/m^3)	2.5
Height of pier, H (m)	5, 10, 15, 20
Diameter of pier, D (m)	1.25, 1.50, 1.75

Table 3: Calculated pile cap stiffnesses.

Case	Lateral Stiffness (MN/m)	Cross-Coupling Stiffness (MNm/rad)	Rotational Stiffness (MN/rad)
1	704	364	2440
2	507	118	1244
3	273	99	996
4	1075	578	7375
5	858	177	4169
6	507	178	3412

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, $K^S{}_P/K^F{}_L$.

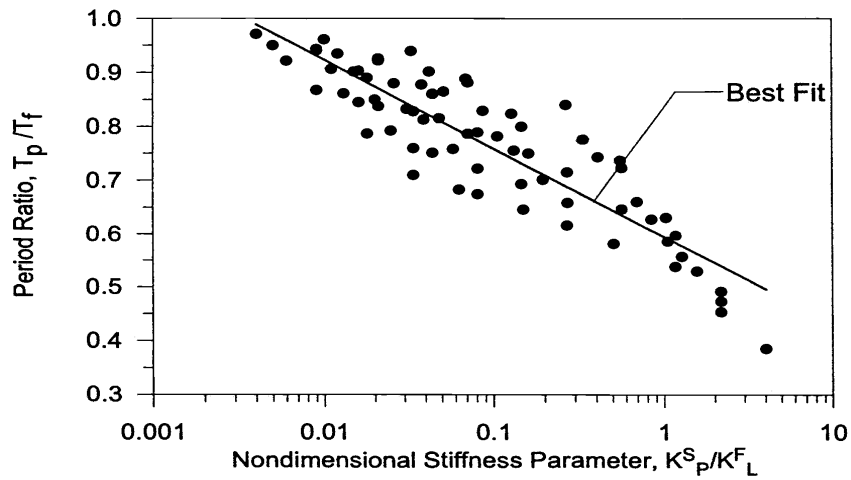

Fig. 20: Period shift for bridge-foundation system as a function of relative superstructure – foundation lateral stiffness.

ROLE OF AMBIENT VIBRATION MEASUREMENTS

There are two phases in the calculation of pile foundation stiffnesses, calculating the stiffnesses at the initiation of shaking and tracking the evolution of these stiffnesses under the varying intensities of earthquake shaking during the earthquake. Evolution requires a constitutive soil model that has the demonstrated capability to model nonlinear soil behavior satisfactorily for the problem under consideration. The initial stiffnesses depend only on the initial distribution of moduli in the ground before the earthquake and are calculated for elastic response. Since they represent the starting point on the evolutionary path, it is important to get the best estimate of their distribution as possible.

Site investigations to characterize foundation soils vary widely in the technology used and level of detail in defining the in situ soil properties. Therefore, it is desirable, at least for critical structures, to check independently the estimates of initial stiffnesses. Ambient vibration testing provides a cost effective way to do this. The utility of the method will be demonstrated by an example from engineering practice.

The Queensborough Bridge over the Fraser River in Vancouver, Canada, is over 800m long with three major structures. The North Approach is 220m long, the South Approach is 490m and the Main Span over the Fraser River is 204m. The bridge was built in 1960. It is considered to be a critical structure and was retrofitted in the mid-90's. The Main Span is shown in Fig. 21. Abutment and river piers are shown in Fig. 22, together with the finite element pier model in which the initial foundation stiffnesses are represented by two sets of springs. The initial foundation stiffnesses were calculated for pier S_1, using soil data from a site investigation conducted at pier S_1. Since soil data was not available for piers S_2, N_1 and N_2, the translational and rotational stiffnesses of S_1 were scaled in proportion to the area and second area moment of inertia of the bases of these piers to provide estimates of their stiffnesses.

A series of ambient vibration studies (AVS) of the Main Span were conducted by Ventura et al (18, 19) before the retrofit work started. Mode shapes and modal frequencies were determined from the ambient vibration data. The experimental and analytical first three frequencies of transverse vibration are shown in Fig. 23. The sensitivity of these analytical frequencies to variations in the initial moduli is also shown in Fig 23. where variations in the moduli are expressed as multiples of the initial values. For an accurate prediction of the first transverse mode, the initial moduli need to be doubled. Surprisingly the frequency based on the initial moduli is quite good; it is90% of the measured frequency. For the higher frequencies the soil properties needed adjustment. The relative insensitivity of the frequencies to soil moduli at higher stiffnesses arises from the fact that the system frequencies depend not on foundation stiffnesses but on the combined stiffnesses of the foundation and super- structure, as discussed earlier.

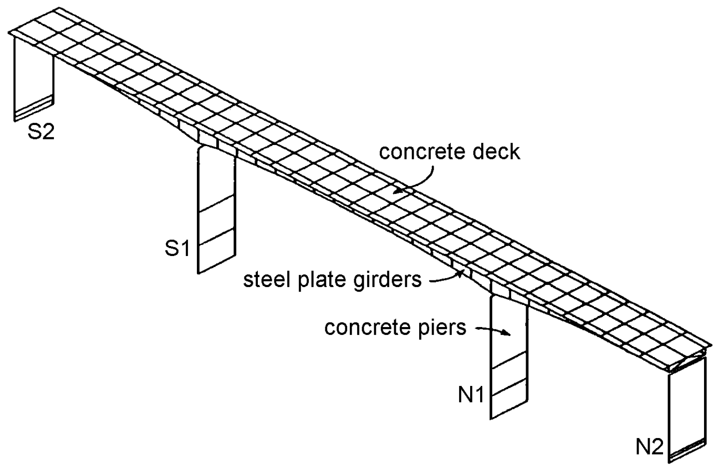

Fig. 21: Main span of the Queensborough Bridge showing the river (S_1, N_1) and the abutment piers (S_2, N_2) (18).

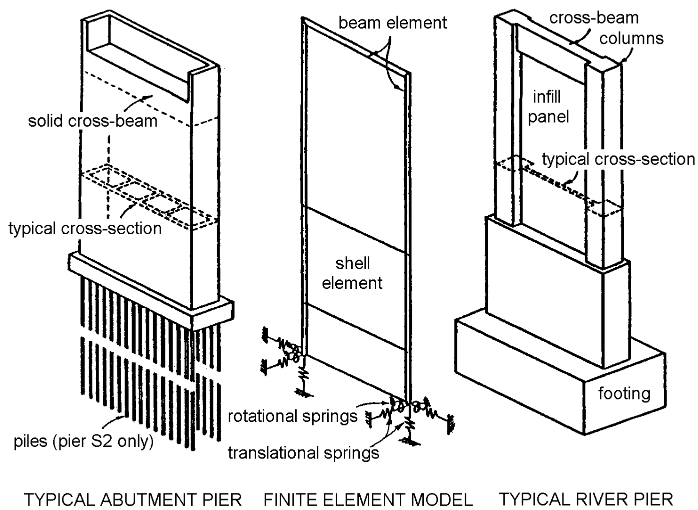

Fig. 22: Typical piers and a finite element pier model of Queensborough Bridge (18).

The use of ambient vibration data to adjust the initial stiffnesses of the foundations is based on the tacit assumption that the structural modeling needs no adjustment.

Ambient vibration can also provide valuable data on potential displacement patterns of a bridge. Additional lateral vibration measurements were taken at selected piers to verify the frequencies of the lateral modes of vibration. The transverse mode shape between piers S19 and S23 in the South Approach is shown in Fig. 24. Two piers showed significant translation during the AVS. The pile cap of pier S21 had a lateral translation that was 40% of the translation of the deck. The pile cap is supported by twenty five, 14m long, untreated timber piles that penetrate 14m of peat to bed on fine sand. The low translational stiffness of the pile cap is consistent with the low stiffness of the peat. The lateral motion at pier S21 is shown in Fig. 24 and the coupling of the different modes of displacement is evident.

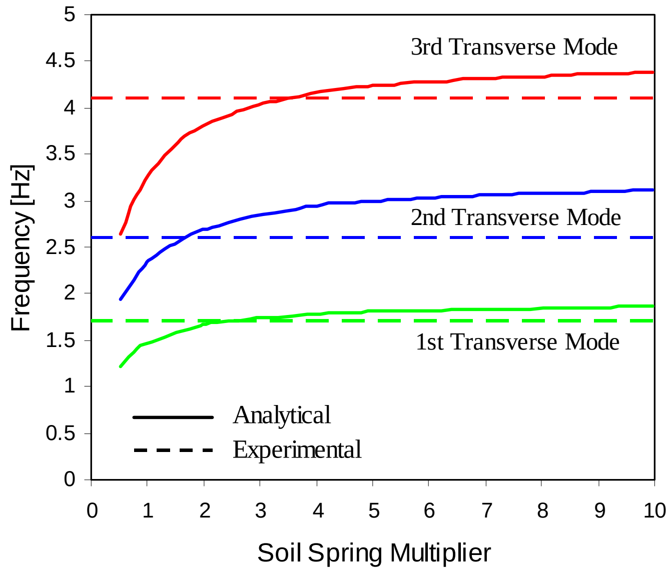

Fig. 23: Sensitivity of computed transverse frequencies to soil spring stiffness (19).

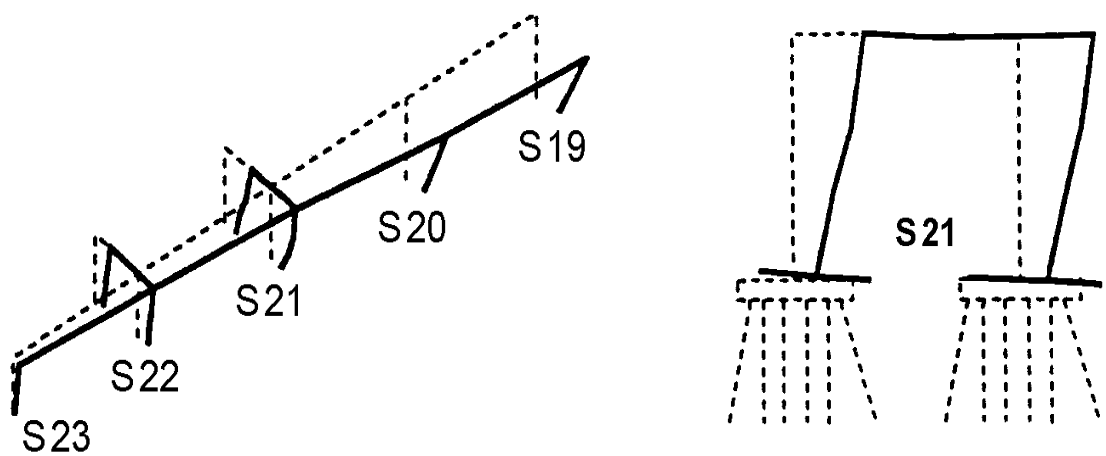

Fig. 24: Transverse mode shape between piers S19 and S 23 and displacements at S21 (19).

PILES IN LIQEFIABLE SOILS

Pile cap stiffnesses for foundations in liquefiable soils can also be calculated as described above. The additional information required is data on the liquefaction resistance of the soil. This data can be obtained from in situ tests or from laboratory tests, if undisturbed samples can be recovered. The resistance data is used to calibrate the pore water generation model in PILE-3D. During shaking the pore water pressures are generated continuously and the shear moduli are adjusted to be compatible with the current state of effective stress. The modulus varies roughly as the square root of the effective stress, so when pore water pressures reach 50%, the reduction in modulus is 70%. Up to this level of pore water pressure, the time-history of stiffness will not be very different from that for soil without the excess pore water pressures. However beyond this level stiffness degrades rapidly and can reach the theoretical minimum of the stiffness of the pile foundation itself with no contribution to stiffness from the liquefied soil.

In this section the types of analyses described earlier will not be presented. Instead of focusing upwards on the structure, attention will be directed downwards to the effects of seismic actions on the foundation itself and to a study the displacements and bending moments in the piles.

Many studies have been conducted on the seismic behavior of piles in liquefiable soils to establish a basis for reliable design [5-14, 20-26]. In this section, some of the findings from a major research project supported by the construction company, Anabuki Komuten, with headquarters in Takamatsu, Japan, will be described. The main focus of the study is on the seismic response of large diameter cast-in–place concrete piles. The Anabuki Company uses these piles to support their buildings in reclaimed land. Each column is supported by one pile.

A typical pile installation is shown in Fig. 25 and an idealized computational model for analysis is shown in Fig. 26. The upper 10m are expected to liquefy during the design earthquake. The mass mounted on the pile in Fig. 26 represents the portion of the building mass carried by the pile. It is mounted on the pile head by a flexible support. The flexibility is selected so that the fixed base period of the mass-support system is 1.4s which is the fixed based fundamental period of the prototype structure. The purpose in placing the mass on the pile is to model approximately the inertial interaction between the super-structure and the pile foundation. The nonlinearity of the soil and the effects of seismic pore water pressures are taken into account continuously during analysis.

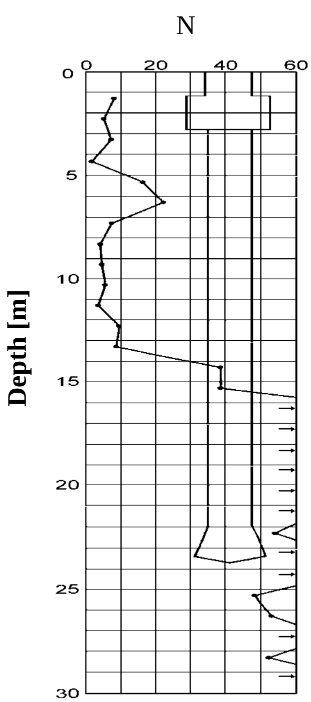

Fig. 25: Site in reclaimed land.

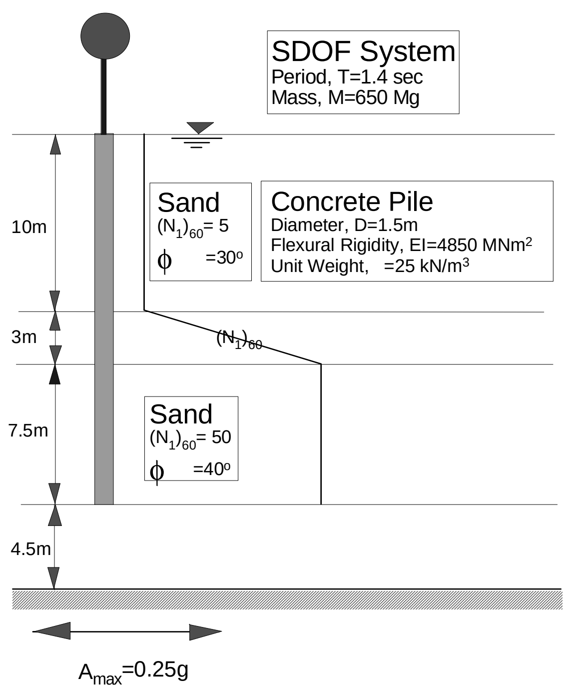

Fig. 26: Model of soil-pile- structure system.

The peak acceleration of the input acceleration record is 0.25g and is amplified to 0.4g at the surface. Dynamic effective stress analyses of this system were conducted for two conditions: including both inertial and kinematic interactions and including kinematic interactions only. The latter analyses did not include the mass of the superstructure. Data from these two kinds of analyses are compared to evaluate the significance of kinematic interaction. The finite element mesh is shown in Fig. 27. Because of symmetry only half the region, including half of the pile, need be meshed. Note the finer mesh surrounding the pile itself.

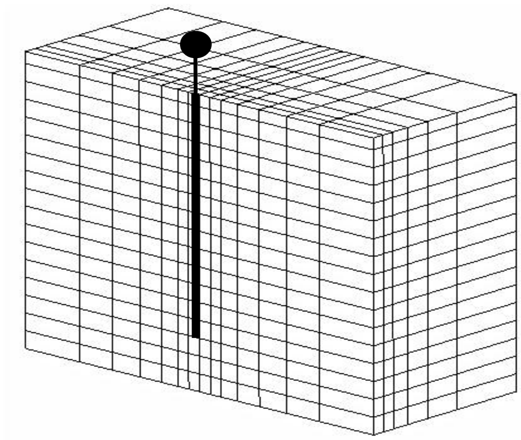

Fig. 27: Finite element mesh for pile analysis taking symmetry into account.

RESULTS OF ANALYSES

Analyses with inertial interaction

Pile displacements and moments for the 14 storey building, at the instant of maximum pile head displacement, are shown in Fig. 28 and Fig. 29 respectively. Approximately the top 10 m liquefy or develop very high pore water pressures during earthquake shaking. Results are shown for two conditions; the pile head is fixed against rotation and the pile head is free to rotate. The displacements are more than twice as large when the pile head is free to rotate. The maximum moment occurs at the pile head, when the pile head is fixed against rotation, but significant moment also occurs at the boundary between the softer and stiffer soils. When the pile head is not fixed against rotation, the maximum moment occurs at the boundary between the stiffer and softer soils. This moment is approximately equal to the pile head moment, when the pile head is fixed against rotation. The results show that when designing piles or evaluating pile foundations in potentially liquefiable soils for earthquake loading, it is important to make a realistic assessment of pile head restraint against rotation and to be aware of the potential for large moments at the interfaces between soft and hard layers.

At some sites a thick surface layer of non-liquefiable soil may lie over the liquefaction zone. A stiff upper layer is incorporated into the original site of the 14 storey building. Deflections and moments for this case, at the instant of maximum pile head displacement, are shown in Fig. 30 and Fig. 31, respectively. As before, the results are shown for two pile head conditions, no pile head rotation and the pile head is free to rotate.

The moments at the pile head and at the interface between the soft and stiff soils have increased by 30%, compared to the case without the upper layer. When the pile is fixed against rotation the moments at the pile head and the interfaces between layers are about the same. The behavior of the upper layer is clarified further in the next section which presents results from kinematic analyses.

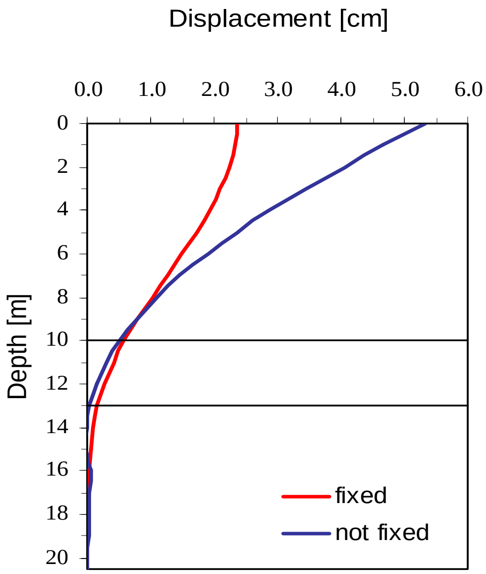

Fig. 28: Pile deflections at maximum pile head displacement.

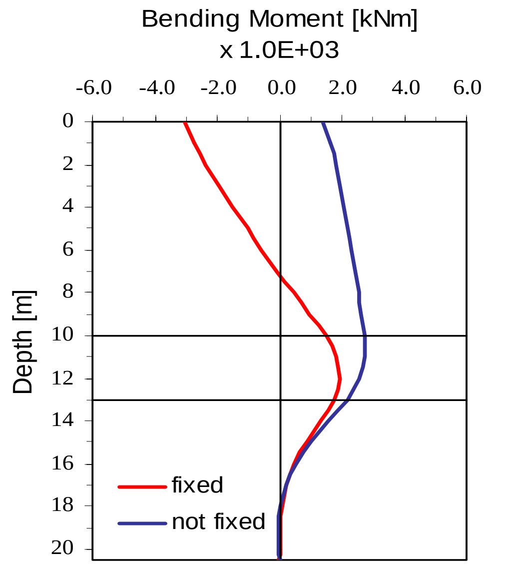

Fig. 29: Pile moments at maximum pile head displacement.

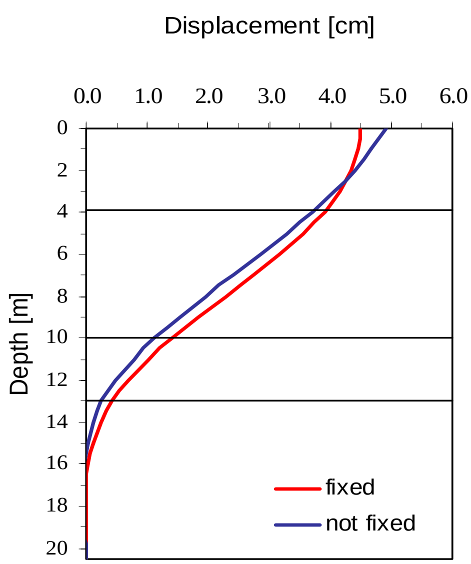

Fig. 30: Pile deflections at maximum pile head displacement.

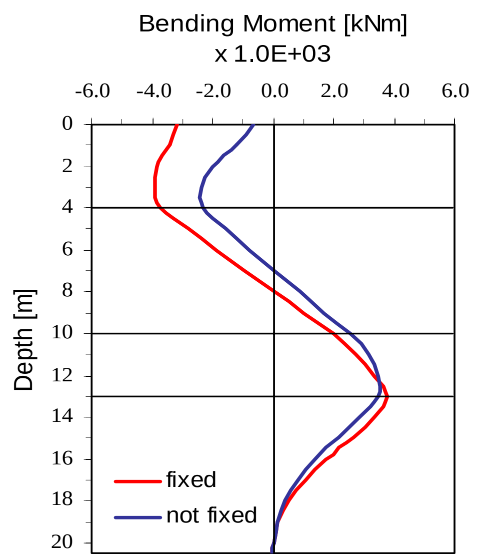

Fig. 31: Pile moments at maximum pile head displacement.

KINEMATIC ANALYSIS

Kinematic analyses were conducted on the 1.5 m diameter pile to assess the importance of kinematic interaction in this case. Analyses were conducted with and without the stiff surface layer and, in each case, the pile head was considered either fixed against rotation or not. The kinematic analyses were conducted by removing the super-structural mass in Fig. 26.

The pile and free field displacements at the instant of maximum pile head displacement are shown in Fig.32 for the case when there is a stiff surface layer. It is evident that the stiff surface layer is moving as a rigid body at the time of maximum pile head displacement which occurs after the incidence of liquefaction. At this time it also appears to be driving the pile, so that the pile and surface layer undergo about the same displacements. Consequently when the stiff surface layer is present, the kinematic pile head moments shown in Fig. 33 are about the same as the moments, when both inertial and kinematic interactions are included (Fig. 31). This indicates that, in this case, the kinematic moments caused be the internal displacements in the foundation soils dominate the moment response of the foundation.

The moments and shears in pile foundations of buildings are often determined by applying the code base shear and moment from a fixed based analysis of the structure to the pile head and conducting a static analysis using a Winkler beam model, to determine the shears and moments in the pile. In this case, such a procedure would not account for the large kinematic moments caused by the internal distribution of seismic displacements in the foundation soils. Clearly analyses that neglect kinematic effects may in some situations underestimate significantly design moments and shearing forces in foundation piles.

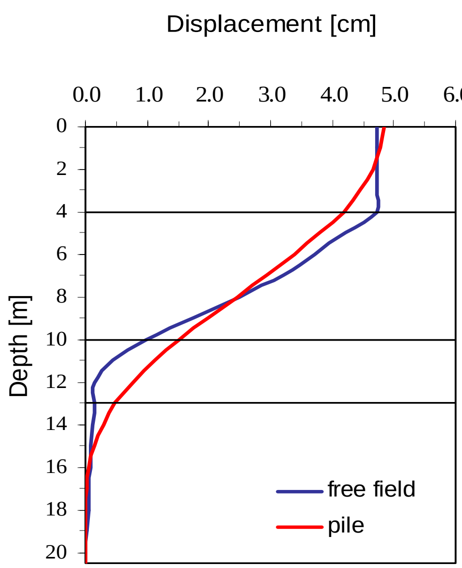

Fig. 32: Displacements of pile and free field at maximum pile head displacement.

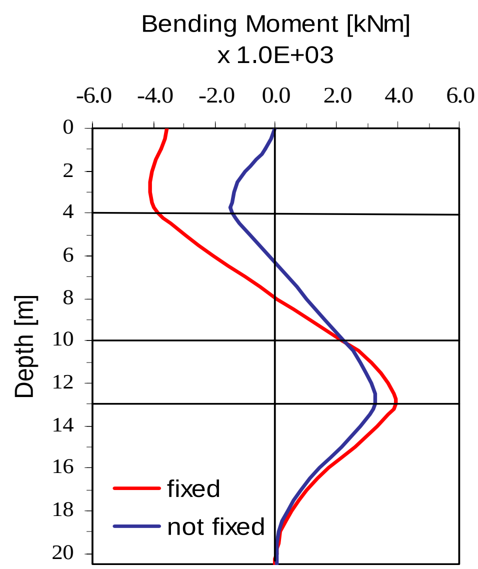

Fig. 33: Kinematic moments at maximum pile head displacement

CLOSING REMARKS

This paper explores the reliability of some approximate methods for estimating and representing the rotational and translational stiffnesses of pile foundations in the computational structural model of a superstructure. The study is focused on a 4x4 pile group supporting a bridge pier. The stiffnesses are incorporated in the computational model by single value springs with appropriate stiffness factors.

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 comparing time histories of pile cap stiffnesses during strong earthquake shaking with the single valued spring stiffnesses based on effective free field moduli from a SHAKE analysis.. The consequences of using various approximations to pile cap stiffnesses are investigated by examining their effects on the 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 in some cases.

The pile cap stiffnesses affect the seismic response of the superstructure by their effect on system frequencies. These frequencies depend not directly on the values of the foundation stiffness coefficients but on the ratio of superstructure stiffness to pile cap stiffness in a specific mode. If the stiffness ratio is very low, the structure behaves as if on a fixed base. If the ratio is high, then a significant change in period from the fixed base period occurs in the system period and consequently in the seismic response. Parametric studies were conducted to develop an approximate relationship between stiffness ratio and period change for the guidance of designers.

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.

The behavior of piles in liquefied ground was studied in the context of large diameter CDIH reinforced concrete piles. These piles are often used to support buildings in reclaimed land in Japan and as combined foundation –piers for bridges worldwide. Analyses show that large bending moments develop in critical areas such as at the pile head, when it is fixed against rotation, and the boundary between liquefied and non-liquefied layers. The analyses also demonstrate that if a stiff surface layer overlies the liquefied zone, then the moment and deflection demands on the pile may be substantially increased over the case when the stiff upper layer is not present.

Restraint against pile head rotation has a significant effect on the response of piles in liquefied soils, when the surface layer liquefies. The pile cap displacements may be up to three times larger, if the restraint is low compared to full fixity. Reliable estimates of pile head fixity are necessary for estimating pile head rotations and displacements.

The keys to good design are reliable estimates of environmental loads, realistic assessments of pile head fixity and the use of methods of analysis that can take into account adequately all the factors that control significantly the response of the pile-soil-structure system to strong shaking in a specific design situation. Not all factors are important all the time but an informed background is essential in making decisions about what can be ignored safely.

ACKNOWLEDGEMENTS

The author wishes to acknowledge his great debt to his former PhD student, Dr T. Thavaraj, Klohn Crippen Consultants of Richmond, BC, Canada. The data on pile cap stiffnesses was developed by Dr. Thavaraj as part of his PhD studies. The research project on the seismic design and analysis of pile foundations at Kagawa University is funded by Anabuki Komuten, Takamatsu, Japan. The support of the company is gratefully acknowledged. Noboru Fujita, Research Assistant, carried out the analyses on the large diameter piles and his help was invaluable in the preparation of this paper.

REFERENCES

1. API, “Recommended practice for planning, designing, and constructing fixed offshore platforms.” API RP 2A, American Petroleum Institute, 1995.
2. Murchison JM, O’Neill MW. “An evaluation of p-y relationships in cohesionless soils.” Proc. ASCE Sym. Analysis and Design of Pile Foundations, ASCE National Convention, San Francisco, California, J. R. Meyer, Editor, pp. 174-191, 1984.
3. Thavaraj T, Finn WD Liam, “A program for dynamic analysis of a single pile using a Winkler model and p-y curves.” Anabuki Chair of Foundation Geodynamics, Kagawa University, Japan, 2000.
4. Schnabel PB, Lysmer J, Seed HB. “SHAKE: A computer program for earthquake response analysis of horizontally layered sites.” Earthquake Engineering Research Center, University of California, Berkeley, CA, Report No. EERC72-12, 1972.
5. Finn WD Liam, Thavaraj T. “Deep foundations in liquefiable soils: Case histories, centrifuge tests and methods of analysis.” CD-ROM Proceedings, 4th Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, 2001.
6. Liu L, Dobry R. “Effect of liquefaction on lateral response of piles by centrifuge tests.” Nat.Ctr for Earthquake Engrg. Res. (NCEER) Bull., 9 (1), pp. 7-11.
7. Weaver TJ, Ashford SA, Rollins KM. “Development of p-y curves for a 0.6 m diameter CISS pile in liquefied sand.” CDROM Proc. 6th CALTRANS Seismic Research Workshop, Sacramento, CA, 2001.
8. Wilson DW, Boulanger RW, Kutter BL. “Observed lateral resistance of liquefying sand.” J. of Geotech. And Geoenvir, Engrg., ASCE, 126 (10), pp. 898-906.
9. Brandenburg SJ, Singh P, Boulanger RW, Kutter BL. “Behavior of piles in laterally spreading ground during earthquakes.” CDROM Proc. 6th CALTRANS Seismic Research Workshop, Sacramento, CA, 2001.
10. Finn WD Liam, Wu G, “Recent developments in the static and dynamic analysis of pile groups.” Proc. Annual Sym. Vancouver Geotechnical Society. Vancouver, B. C. Canada , 1994, pp. 1-24.
11. Wu G, Finn WD Liam, “Dynamic elastic analysis of pile foundations using the finite element method in the frequency domain.” Canadian Geotechnical Journal, (34), 34-43, 1997.
12. Wu G, Finn WD Liam, “Dynamic nonlinear analysis of pile foundations using the finite element method in the time domain.” Canadian Geotechnical Journal, (34), 144-152, 1997.
13. Thavaraj T, Finn WD Liam, “PILE-3D-EFF: A program for nonlinear dynamic effective stress analysis of pile foundations.” Anabuki Chair of Foundation Geodynamics, Kagawa University, Japan, 1999.
14. Finn WD Liam, Thavaraj T, Wilson DW, Boulanger RW, Kutter BL. “Seismic analysis of piles and pile groups in liquefiable sand.” Proc. 7th Intern. Sym. Numerical Models in Geomechanics, NUMOG VI, Graz, Austria, pp. 287-292, 1999.
15. Seed HB, Idriss IM. “Soil moduli and damping factors for dynamic response analysis.” Earthquake Engineering Research Center, University of California, Berkeley, CA, Report No. EERC70-10, 1970.
16. AASHTO, “Guide specifications for seismic design of highway bridges.” American Association of State Highway and Transportation Officials, Washington, DC, USA, 1983.
17. Thavaraj T, Liam Finn WD. “A program for dynamic analysis of bridges incorporating foundation springs with specified time histories of stiffnesses.” Anabuki Chair of Foundation Geodynamics, Kagawa University, Japan, 2001.
18. Ventura CE, Felber AJ, Prion HGL, “Dynamic Evaluation of a Medium Span Bridge by Modal Testing.” Proceedings of the 4th International Conference on Short and Medium Span Bridges, Halifax, Nova Scotia, August 8-11, 1994, pp. 545-556.
19. Ventura CE, Felber AJ, Stiemer SF. “Dynamic Characteristics of Bridges by Experimental Investigations of Ambient Vibrations - Queensborough Bridge.” Proceedings, 5thUS NationalConference on Earthquake Engineering, Chicago, Illinois, July 1994, Vol. II, pp. 733-742.

20. Iai S, Tobita T. "Analysis of soil-pile interaction," CDROM Proc. 8th US-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Liquefaction, Tokyo, Japan, 2002.

21. Sato T, Matsumara T, Zhang F, Moon Y, Uzuoka R. "3-Dimensional simulation of pile-ground system during liquefaction and following ground flow process," CDROM Proc. 8th US-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Liquefaction, Tokyo, Japan, 2002.

22. Kagawa T, Minowa C. "Effects of liquefaction on soil-structure-interaction," Proc. 10th European Conf. on Earthquake Engrg, Duma (ed.), Balkema, Rotterdam, pp. 617-622, 1995.

23. Cubrinovski M, Ishihara K, Furukuwazono K. "Analysis of full-scale tests on piles in deposits subjected to liquefaction," in Earthquake Geotechnical Engineering, Balkema, Rotterdam, pp. 567-572,1999.

24. Ishihara K. " Effects of spacing on performance of pile groups undergoing latyeral spreading," CDROM Proc. 8th US-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Liquefaction, Tokyo, Japan, 2002.

25. Yasuda S, Ishihara K, Orense R, Ikeda M, Tamura S. "Large scale shaking table tests on pile foundations in liquefied ground," in Proc. 12WCEE 2000, Paper 1474, Auckland, New Zealand, pp. 8, 2000.

26. Kobayashi K, Yoshida N, Yao S. "Pile-Soil Modelling in liquefied sand layer in aseismic design method," in Earthquake Engineering, 10th World Conference, Balkema, Rotterdam, pp.1743-1748, 1992.