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SEISMIC ANALYSIS OF PILE FOUNDATIONS: STATE-OF-ART

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ABSTRACT

Some of the new developments in the state of practice for the seismic design of pile foundations are presented. These developments are assessed against the findings of 3-D nonlinear dynamic analysis.

KEYWORDS

Bridge foundations; dynamic pile group stiffness; 3-D nonlinear seismic analysis; soil-pile interaction; seismic foundation-structure interaction

INTRODUCTION

The seismic response analysis of structures on pile foundations is a very complex process because of pile-soil-pile and foundation-structure interactions and the nonlinear response of soils to strong shaking. This is especially true of bridges with their multiple supports. This paper concentrates on bridge foundations, an area in which the state of the art has been rapidly changing as a result of foundation failures in recent earthquakes.

The most rigorous solutions for pile foundations are available for linear elastic response which approximates the behaviour under low levels of shaking. A solution often used in practice is the plane strain elastic layer solution for a single pile by Novak, incorporated in the program DYNA-3 (Novak et al., 1990). Pile group action is simulated by using dynamic interaction factors obtained by Kaynia and Kausel (1982) using elastic boundary element solutions. These factors are available only for small pile groups. For larger groups static interaction factors by Poulos and Davis (1980) are used beyond the range of the Kaynia and Kausel (1982) factors. Various approaches are used to adapt this procedure to cope with the nonlinear behaviour of soil. The simplest approach is to reduce the initial moduli by an arbitrary factor to simulate the softening of the soil under cyclic loading. Reduction factors of 2 to 3 have been used. A more realistic approach is to determine the free field effective moduli by equivalent linear analysis using the program SHAKE (Schnabel et al., 1972) and to use these moduli in the DYNA-3 elastic analyses.

Probably the most common way to model nonlinear response in practice is to use nonlinear p-y curves which relate the pressure of the soil against the pile to pile deflection. There are various procedures in the literature for constructing p-y curves for given site conditions. A widely used procedure is that described in the recommendations of the American Petroleum Institute (API, 1993) for the design of offshore structures. Group action may be modelled by using elastic interaction factors as in the Novak approach.

Commercial structural programs do not allow direct coupling between the structure and the nonlinear foundation elements. The restraining effects of the foundation on structural response are approximated by translational and rotational springs at the supports or more completely by stiffness matrices that represent all the direct and cross-coupled stiffnesses of the pile foundation. The determination of these elemental stiffnesses of the springs is routine for elastic conditions but for pile foundations modelled using p-y curves the stiffnesses must be selected at specific levels of displacement and rotation. It has been suggested that the lateral stiffness be evaluated at a lateral displacement of the pile cap of 25 mm. This procedure introduces a somewhat arbitrary element into the determination of pile foundation stiffnesses.

In the procedures outlined above, the pile foundation is analyzed without any supporting structure. Hence, the inertial interaction between structure and foundation is neglected. This interaction tends to drive the foundation to greater displacements which reduce the stiffness further.

The state transportation organization in California, USA, CALTRANS, has recently proposed a tentative procedure for comprehensive analysis of pile foundations based on the p-y curve approach which includes, in an approximate way, all the important factors controlling response (Abghari and Chai, 1995). The procedure is still being researched. The CALTRANS procedure is as follows.

- Each pile foundation is modelled by a single pile carrying its proportion of the superstructure and pile cap mass under static conditions. The superstructure mass is supported on a column in such a way that the fundamental period of the superstructure is modelled in the mode of vibration under consideration. This procedure ensures that some inertial interaction is included in the response of the pile foundation. Group action is represented by a group reduction factor which depends on pile spacing.

- The pile is modelled by finite elements using linear elastic or nonlinear elasto-plastic beam elements. The pile-soil interaction is modelled by p-y (lateral soil resistance), t-z (axial soil response), and Q-Z (tip resistance) curves which can be generated according to API (1993) recommendations or can be input directly.

- Structural damping and the hysteretic and radiation damping of the soil are modelled using Rayleigh damping.

- The free-field ground motions are calculated using nonlinear site response. These motions are then applied to the soil springs attached to the finite element nodes of the pile.

Another recent development is a method for direct nonlinear seismic analysis for pile groups, PILE-3D, developed at the University of British Columbia (Wu and Finn, 1994, Finn and Wu, 1996). The program analyzes the foundation soils and pile foundations in a fully coupled manner, automatically taking into account pile-soil-pile interaction, gapping and yielding near the pile head. Some analyses conducted using PILE-3D will be presented to give some insight into the approximation suggested above.

THEORETICAL BASIS OF PILE-3D ANALYSIS

A brief summary of the PILE-3D method of analysis is given here. For details the reader is referred to Finn and Wu (1996) in the proceedings of this conference. Under vertically propagating shear waves the soil undergoes primarily shearing deformations on horizontal planes except in the area around the pile where extensive compressional deformations develop in the direction of shaking. These deformations also generate shearing stresses on vertical planes near the pile. Applying dynamic equilibrium in the direction of shaking, the governing equation for free vibration of the soil continuum is given by

$$\rho_s \frac{\partial^2 v}{\partial t^2} = G\frac{\partial^2 v}{\partial x^2} + \theta G \frac{\partial^2 v}{\partial y^2} + G\frac{\partial^2 v}{\partial z^2} \tag{1}$$

where G is the shear modulus, ρ_S is the mass density of soil, and θ is a coefficient related to Poisson's ratio of the soil. Piles are modelled using ordinary Eulerian beam theory. An 8-node brick element is used to represent soil, and a 2-node beam element is used to simulate the piles. The global dynamic equilibrium equation in matrix form is written as

$$[M]\{\ddot{v}\} + [C]\{\dot{v}\} + [K]\{v\} = -[M]\{I\} \cdot \ddot{v}_0(t) \tag{2}$$

in which $\ddot{v}_0(t)$ is the base acceleration, {I} is a unit column vector, and $\{\ddot{v}\}$, $\{\dot{v}\}$ and $\{v\}$ are the relative nodal acceleration, velocity and displacement, respectively.

The nonlinear hysteretic behaviour is modelled by using a continuous form of the equivalent linear method used in the SHAKE program (Schnabel et al., 1972). Additional features such as tension cut-off and shearing failure are incorporated in the program to simulate the possible gapping between soil and pile near the soil surface and yielding in the near field.

NONLINEAR SEISMIC ANALYSIS OF PILE FOUNDATION

PILE-3D is used to analyze the pile foundations of the bridge shown in Fig. 1. The bridge is used as an example in guide to the seismic design of bridges published by the American Association of State Highway and Transportation Officials (AASHTO, 1983). The profile of the foundation soil was selected from another bridge being studied by the authors. The profile is 10 m of stiff sand overlying bedrock. The shear modulus, G, varies parabolically with depth with a maximum value G = 213 MPa at 10 m. The variation of shear modulus and damping with seismic shear strain amplitude used in the analyses were

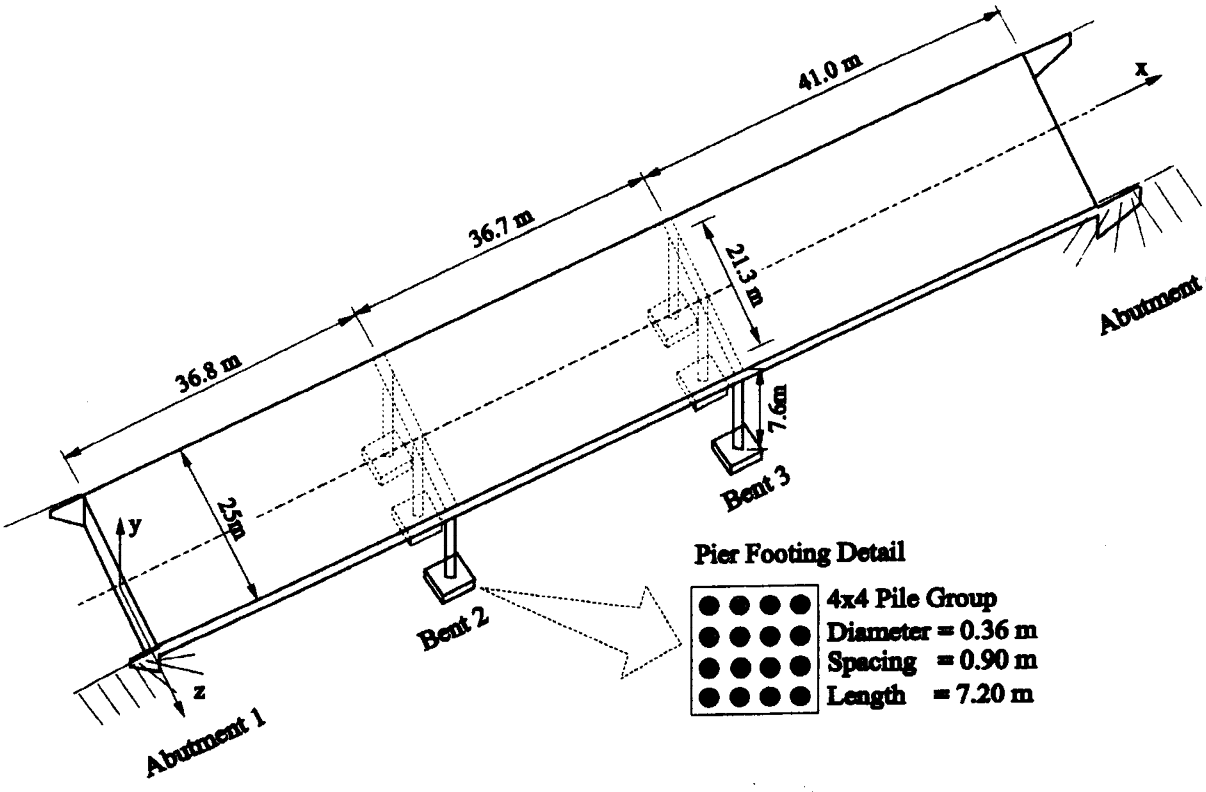

Fig. 1. Bridge and pile foundations used in PILE-3D analyses.

those recommended for sand by Seed and Idriss (1970). The input motion for the study were the N-S free-field accelerations recorded at CSMIP Station No. 89320 at Rio Dell, California during the April 25, 1995 Cape Mendocino-Petrolia earthquake.

Effect of Pile Foundation on Fundamental Frequency of Bridge

A 3-D structural analysis was used to determine the fundamental frequencies of the bridge in the different modes of vibration. The computed transverse frequency, assuming the bridge was founded on rigid supports, was found to be 3.18 Hz which agrees with the frequency cited in AASHTO (1983).

The effects of foundation compliance on the dynamic characteristics of the superstructure were analyzed, assuming elastic response, using the initial moduli of the site. The fundamental period of the bridge in the transverse direction reduced slightly to 3.08 Hz. In this case, the vertical columns had a stiffness corresponding to 7% of the lateral stiffness of the pile foundation.

The initial analysis was then repeated but assuming that the vertical columns had a stiffness equal to 50% of the initial foundation stiffness. For this condition, the fixed base frequency became 5.82 Hz reflecting the increased stiffness of the superstructure and the frequency including the compliance of the foundation was 4.43 Hz. These results show that the effect of foundation compliance on the superstructure is dependent on the relative stiffnesses of the foundations and the supporting columns. It also suggests, as noted by CALTRANS, that foundation interaction effects are likely to be significant only in relatively soft foundations.

The effects of the earthquake on the soil properties in the free-field was next taken into account. A SHAKE (Schnabel et al., 1972) analysis was conducted to find a vertical distribution of moduli and damping ratios compatible with the effective strain levels (65% maximum strain in each layer) developed during the earthquake. These reduced moduli led to a somewhat softer foundation and a slightly lower fundamental frequency of 4.18 Hz. The results of these elastic analyses are shown in Fig. 2.

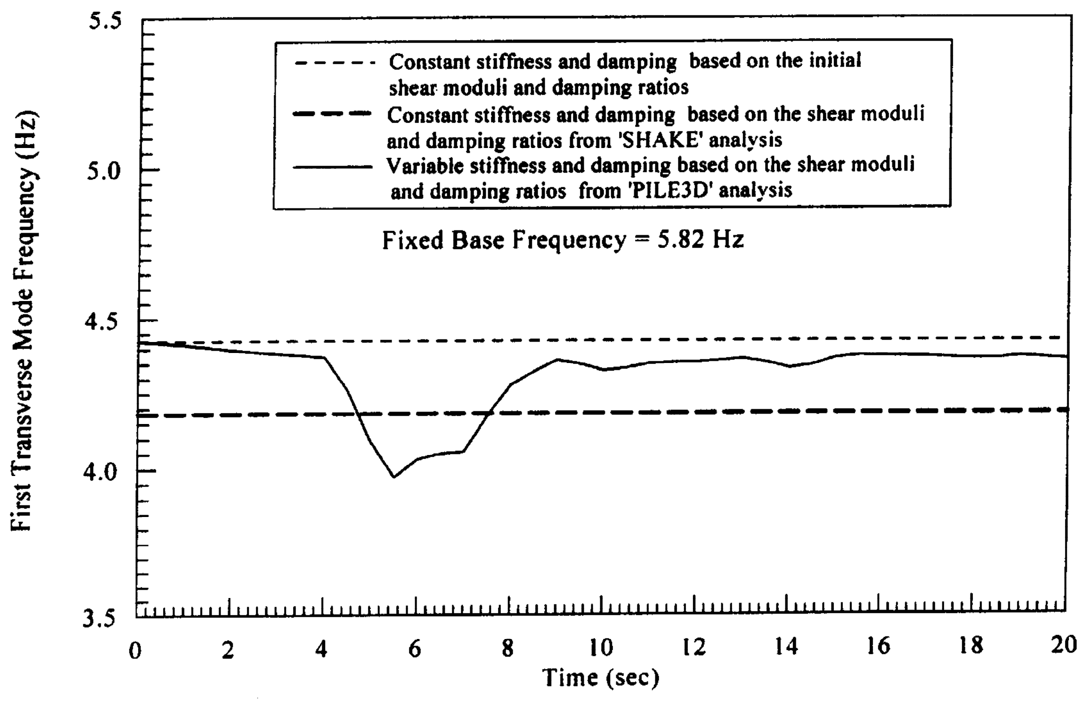

Fig. 2. The effect of foundation compliance on transverse mode frequency for column pile foundation stiffness ratio of 0.5.

Finally, a nonlinear analysis was conducted using PILE-3D. For the nonlinear analysis, it is necessary to include the effects of the inertial interaction between superstructure and foundation. The appropriate inertial mass was selected using the procedure recommended by CALTRANS above. The nonlinear analysis gives the time-histories of variations in modulus and damping and consequently the fundamental period of the bridge can be evaluated as a function of time during the earthquake. The time-history of the fundamental transverse frequency is shown in Fig. 2. together with the periods based on initial and SHAKE moduli and damping ratios. It is interesting to note the consequences of conducting the analysis with constant moduli and damping values, even the values from the SHAKE analyses which reflect some of the effects of the earthquake. For most of the duration of shaking, the estimated frequency from the SHAKE analysis is too low, and during the period of stronger shaking, the frequency is too high. At this relatively stiff site these differences may not be very significant, but for much softer sites, for example some of the sites in Kobe during the 1995 earthquake, the differences would be very significant.

Lateral Stiffness of Pile Foundation

The lateral stiffness of the pile foundation was evaluated using PILE-3D for the two cases when the supporting column stiffness is 7% and 50% of the foundation stiffness. Because of space limitations only the transverse stiffness is considered. The time variation of lateral stiffness is shown in Fig. 3. It is clear that there is a very large reduction in lateral stiffness during the period of strong shaking. The initial stiffness is 870 MN/m and during the period of strongest shaking, the stiffness reduces to 190 MN/m, a reduction to 22% of the initial stiffness when the supporting columns are stiff. When the supporting columns are relatively soft compared to the pile foundation, the reduction in lateral stiffness of the pile foundation is less. In this case, the lateral stiffness of the pile foundation is reduced from 870 MN/m to about 360 MN/m. This is a reduction to 41% of the original value. The differences in loss of lateral stiffness is due to the greater inertial interaction between the superstructure and the foundation for the stiffer bridge. When the supporting columns are relatively soft compared to the foundation stiffness, there is very little inertial interaction. This may be seen in Fig. 3, which also shows the time-history of lateral

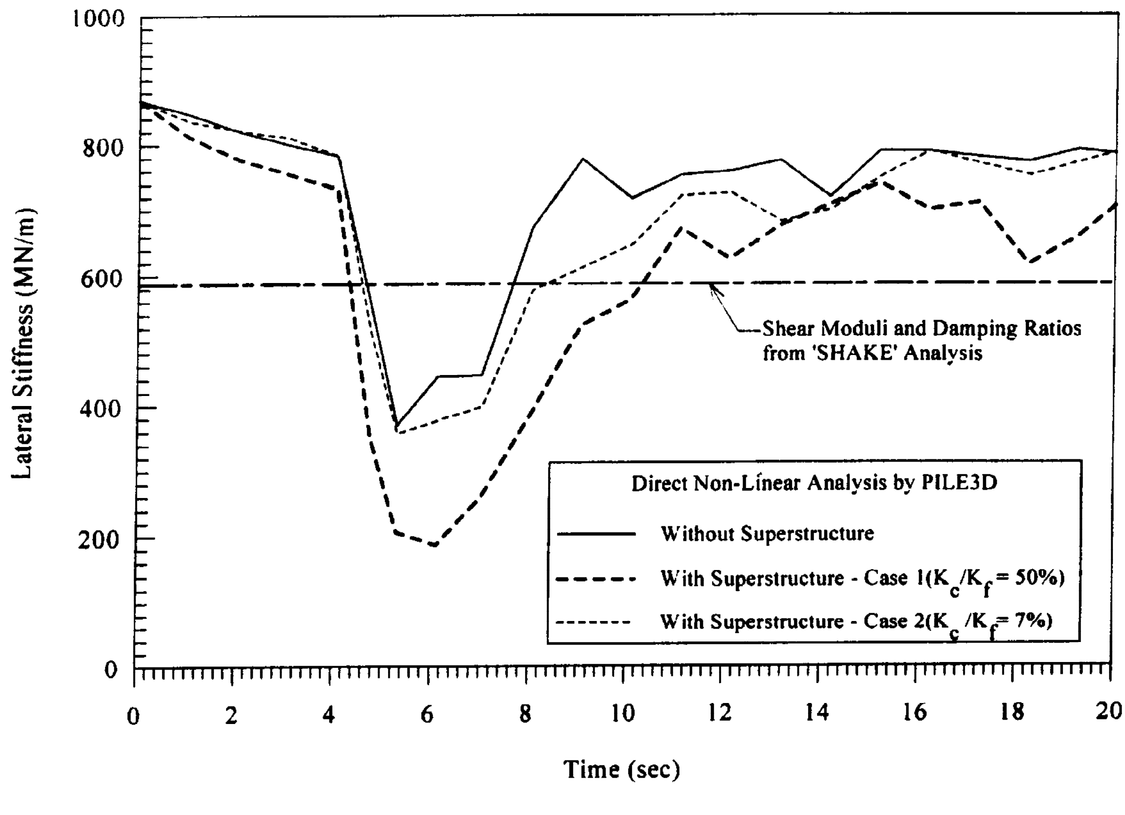

Fig. 3. Time histories of lateral pile foundation stiffness for different conditions.

stiffness of the foundation computed without taking the inertial mass of the superstructure into account. The lateral stiffnesses for no inertial interaction are very similar to the lateral stiffnesses with inertial interaction when the supporting columns are relatively soft compared to the pile foundation.

The constant lateral stiffness computed using the effective 'SHAKE' moduli in an elastic analysis is shown for comparison in Fig. 3. The variation of the nonlinear PILE-3D lateral stiffness and damping ratios about the constant 'SHAKE' stiffness is substantial and indicates how difficult it may be in critical cases to select a single value that will effectively reflect the action of the foundation. This can be a major problem when accurate assessments of the potential displacements are critical as in the case of tall bridges, for example, the tall harbour bridges which dropped decks during the Kobe earthquake.

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