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Seismic Response of Pile Foundations for Bridges

W.D.L. Finn[4], G. Wu[5] and T. Thavaraj[6]

ABSTRACT

A method is presented for the nonlinear dynamic effective stress analysis of pile groups including nonlinear stress-strain response of the foundation soils and pile-to-pile interaction. The method is used to analyze the behaviour of a large bridge foundation under strong earthquake shaking, and to demonstrate the importance of including the effects of inertial interaction from the mass of the superstructure.

INTRODUCTION

The seismic behaviour of bridges on pile foundations is a very complex process. To simplify the seismic structural analysis of bridges, discrete stiffnesses and dampers associated with the different degrees of freedom of the pile foundations are assigned to the bridge model. Comprehensive methods for seismic response analysis of pile foundations to establish the discrete stiffnesses and damping values for bridge models are based on linear elastic behaviour and use either boundary elements or finite element methods. Published graphs of the results are limited to relatively small pile groups because of the substantial computing time required. Novak (1991) gives an extensive review of these methods of analysis. The methods of analysis are exact for elastic isotropic conditions, but they do not take into account the nonlinear behaviour of soil under strong shaking. The reduction in soil stiffness and the increase in damping associated with strong shaking are sometimes modelled crudely in these analyses by making arbitrary reductions in shear moduli and arbitrary increases in viscous damping.

This situation is illustrated by the recent design guidelines of the State of Washington. They recommend two approaches: analyse the pile foundation by itself, using Novak's DYNA program (Novak, 1991) but reduce the initial soil moduli by 50% or (2) use p-y curves (non-linear springs) and dashpots to simulate the nonlinearity of the soil response. For design purposes, pile head stiffness, in the latter case, is evaluated at a displacement of 2.5 cm. A comparative study shows an order of magnitude difference in pile head stiffnesses from these two (apparently) equally acceptable approaches. A further problem with either (1) or (2) is that the inertial effects of the bridge are ignored. The additional displacements generated by the inertia of the superstructure reduce further the stiffnesses and damping ratios of the pile foundations.

A new approach to 3-D nonlinear seismic response analysis of pile groups which removes the limitations of current methods has been developed at the University of British Columbia (Wu, 1994; Finn and Wu, 1994). It is based on a simplified 3-D representation of the foundation soils which makes dynamic nonlinear effective stress analysis of pile groups in layered soils a feasible design tool. By relaxing some of the boundary conditions associated with a full 3-D analysis, the computing costs can be substantially reduced and the analysis can be done on the equivalent of a 486 PC.

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[4] Professor, Dept. of Civil Engineering, University of British Columbia, Vancouver, B.C., Canada V6T 1Z4
[5] AGRA Earth and Environmental Ltd., Burnaby, B.C. Canada V5C 5A9
[6] Graduate Student, Dept. of Civil Engineering, University of British Columbia, Vancouver, B.C., Canada V6T 1Z4

The simplified 3-D analysis gives the time-history of pile foundation stiffnesses and damping values during an earthquake and makes it possible to assess the importance of inertial interaction and the frequency dependence of foundation stiffnesses and damping ratios during strong earthquake shaking. This provides a basis for the selection of representative discrete values of stiffness and damping for bridge models.

Before describing these developments, the reduction in stiffness with ground displacements will be demonstrated from the measured response of a bridge in Northern California during the Cape Mendocino-Petrolia earthquake of 1992.

EFFECTIVE STIFFNESSES OF PILE FOUNDATIONS OF PAINTER STREET BRIDGE OVERPASS DURING CAPE MENDOCINO-PETROLIA EARTHQUAKE

The Painter Street Overpass located near Rio Dell in Northern California, is a two-span, prestressed concrete box-girder bridge that was constructed in 1973 to carry traffic over the four-lane US Highway 101. The bridge is 15.85 m wide and 80.79 m long (Fig. 1). The deck is a multi-cell box girder, 1.73 m thick and is supported on monolithic abutments at each end and a two-column bent that divides the bridge into two spans of unequal length. One of the spans is 44.51 m long and the other is 36.28 m long. The east and west abutments are supported by 14 and 16 piles, respectively. Longitudinal movement of the west abutment is allowed by means of a thermal expansion joint. Each column of the bent is 7.32 m high and supported by 20 concrete friction piles in a 4×5 group.

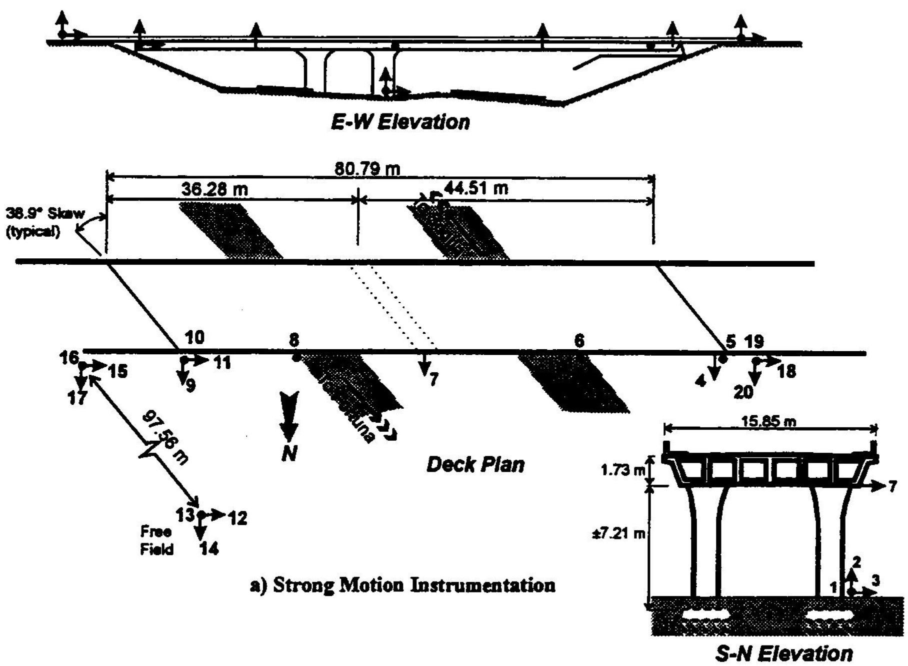

Fig. 1. Dimensions and instrumentation of Painter Street Overpass.

The bridge was instrumented in 1977 as part of a collaborative effort between the California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology and CALTRANS to record and study strong motion records from selected bridges in California. Twenty strong motion accelerometers were installed on and off the bridge as shown in Fig. 1. The instrumentation has recorded several earthquakes since its installation including the main shock of the Cape Mendocino-Petrolia earthquake of 25 April 1992. This earthquake had a Richter magnitude M_L = 6.9, and occurred 6.4 km from the bridge. It generated a free-field peak acceleration in the ground near the bridge, a_p = 0.54 g, and a peak acceleration in the bridge structure, a_p = 1.09 g. Despite the large structural accelerations, no significant structural damage has been observed at the bridge. The extent of damage has been limited to settlement of the backfills and minor spalling of the concrete.

Frequency analysis of bridge deck accelerations under earthquake ground motions indicated that the deck moved as a rigid body in horizontal translation and rotation. Using a modified version of the deck and force system used previously by Goel and Chopra (1995), as shown in Fig. 2, the variation of abutment stiffnesses during strong shaking were determined as a function of relative displacement between the abutment and the free-field. The variation of stiffness parallel to the face of the east abutment is shown in Fig. 3. It is clear that quite modest displacements bring about a large reduction in the stiffness. The stiffness during strong shaking corresponding to relative displacements greater than about 1 cm is about one-fourth (1/4) of the stiffness under the initial weak excitation by the earthquake. This reduction in stiffness is very similar to that determined by Goel and Chopra (1995).

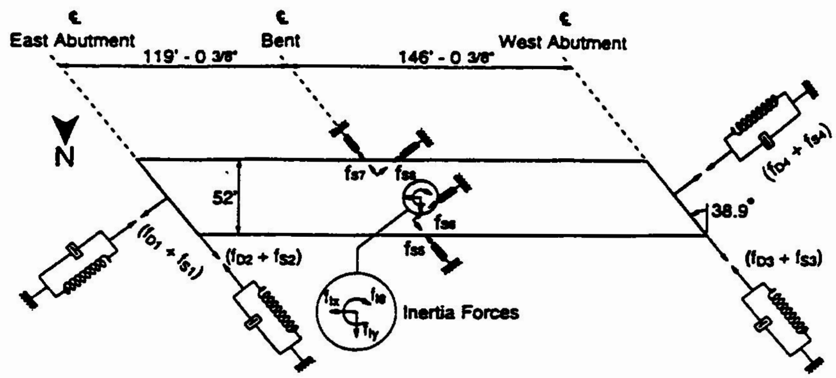

Fig. 2. Rigid body model and force system of Painter Street Overpass.

A drop in stiffness of this amount can have a significant effect on the fundamental period of the bridge as demonstrated by Ventura et al. (1995), from frequency analysis of the records from ambient and strong motion data. It is clearly desirable to have a reliable method for the direct estimation of stiffness and damping under strong shaking.

SIMPLIFIED 3-D SEISMIC ANALYSIS OF PILE FOUNDATIONS

The basic assumptions of the simplified 3D analysis are illustrated in Fig. 4. Under vertically propagating shear waves the soil undergoes primarily shearing deformations in xOy plane except in the area near the pile

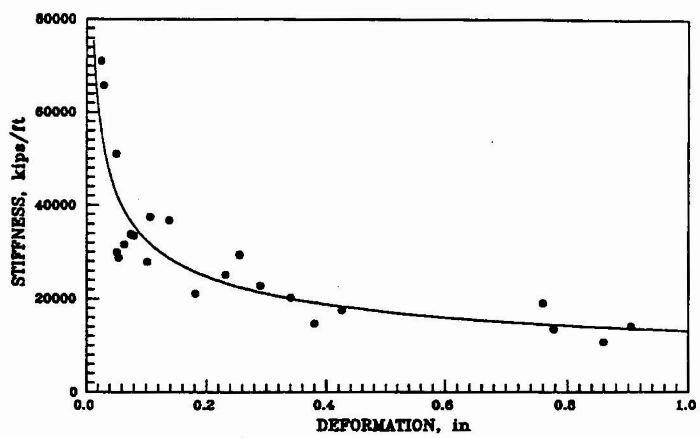

Fig. 3. Variation of lateral stiffness parallel to the east abutment with displacement.

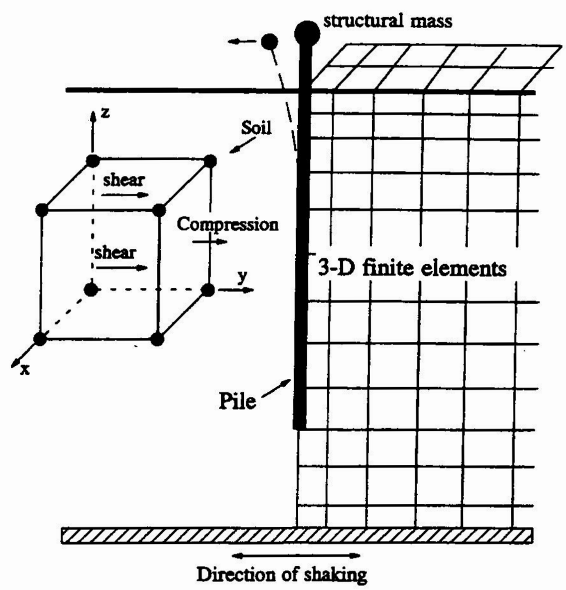

Fig. 4. Quasi-3D model of pile-soil response.

where extensive compressional deformations develop in the direction of shaking. The compressional deformations also generate shearing deformations in yOz plane. Therefore, the assumptions are made that dynamic response is governed by the shear waves in the xOy and yOz planes and compressional waves in the direction of shaking, Y.

Deformations in the vertical direction and normal to the direction of shaking are neglected. Comparisons with full 3D elastic solutions confirm that these deformations are relatively unimportant for horizontal shaking. Applying dynamic equilibrium in Y-direction, the dynamic governing equation of the soil continuum in free vibration is written as

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

where G is the shear modulus, v is the displacement in the direction of shaking, 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. Bending of the piles occurs only in the yOz plane. Dynamic soil-pile-structure interaction is maintained by enforcing displacement compatibility between the pile and soils.

A finite element code PILE3D (Wu and Finn, 1994) was developed to incorporate the dynamic soil-pile-structure interaction theory described previously. An 8-node brick element is used to represent soil and a 2-node beam element is used to simulate the piles, as shown in Fig. 4. 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.

Direct step-by-step integration using the Wilson-θ method is employed in PILE3D to solve the equations of motion in Eq. (2). The non-linear hysteretic behaviour of soil is modelled by using an equivalent linear method in which properties were varied continuously as a function of soil strain. 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.

Seismic centrifuge tests of single piles and pile groups were carried out on the California Institute of Technology Centrifuge by B. Gohl (1991). A centrifuge acceleration of 60 g was used in the tests. Horizontal input accelerations, with a peak acceleration of 0.158 g, were used to study the effects of strong shaking. Details of these tests may be found in a paper by Finn and Gohl (1987). The PILE3D analysis was used to analyze the centrifuge tests. Comparison of computed and measured pile head accelerations and the distrtibutions of peak bending moments along the piles showed that PILE3D can simulate quite well the dynamic response of piles under strong shaking. Details of these verification studies can be found in Finn and Wu (1994).

FOUNDATION STIFFNESSES OF THE PAINTER STREET BRIDGE BENT USING PILE3D

The lateral and rocking stiffnesses of the 20-pile foundation of the bridge bent were evaluated for two conditions, elastic and nonlinear response, in order to demonstrate the importance of nonlinear effects. The elastic stiffness of a single pile was also determined to allow an estimate of the effect of pile-to-pile interaction on stiffness.

The elastic stiffnesses of the pile foundation are not affected by the inertia of the superstructure because the moduli are not dependent on displacements. The stiffnesses of the real soil under strong shaking are affected by the inertia of the superstructure because the stiffnesses are functions of displacement as shown in Fig. 3. Therefore, the proportion of the inertial mass of the deck structure adopted by Makris et al. (1994) in their analysis was included in the nonlinear dynamic response of the bridge bent.

The model of the foundation soils was based on data from field tests conducted by the Lawrence Livermore Laboratory in the U.S., and was supplied by Heuze (1994). The shear modulus of the soil varied parabolically with depth with an average value of 100 MPa in the surficial soil layer.

The model of the bent and the 20-pile foundation was subjected to the accelerations of the main shock of the Cape Mendocino-Petrolia earthquake. The time-histories of lateral and rocking stiffnesses during the earthquake are shown in Fig. 5 and Fig. 6, respectively. What discrete spring stiffnessnes should be selected to represent adequately the complex variations in lateral and rocking stiffnesses shown in Figs. 5 and 6? This is obviously a matter of judgement. The authors' choices are shown in Column 4 of Table 1. The selected stiffnesses are biased towards the values associated with the time of strongest shaking.

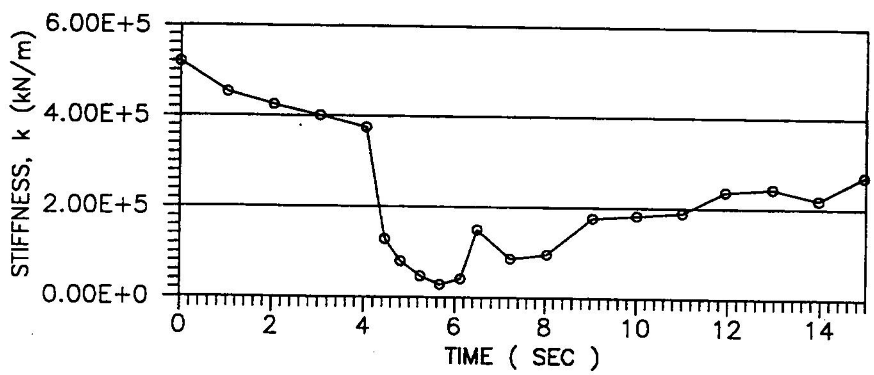

Fig. 5. Time variation of lateral stiffness during strong shaking.

The corresponding elastic stiffnesses of the pile foundation computed using initial moduli are shown in Column 3. The strong shaking of the nonlinear foundation soils has reduced these initial stiffnesses by factors of 3.5 for lateral stiffness, and by 3.2 for rocking stiffness. These reductions are of the same order as those deduced from the rigid body analysis of the entire bridge system shown in Fig. 3. The vertical and cross-coupling stiffnesses for elastic and nonlinear conditions are also given in Table 1. The elastic stiffnesses for a single pile are given in Column 2, Table 1. The elastic stiffnesses of the pile foundation are only about 30% of the stiffnesses corresponding to 20 times the stiffness of a single pile. This huge reduction is due to pile-to-pile interaction.

To demonstrate the effect of inertial interaction, a single pile was analyzed, with and without a structural mass at the pile head. The variations in lateral stiffness, with and without inertial interaction, are shown in Fig. 7. In this case, the inertial interaction has a major effect on the stiffness of the pile. It is clear that inertial interaction should be considered when evaluating pile foundation stiffnesses under strong earthquake shaking.

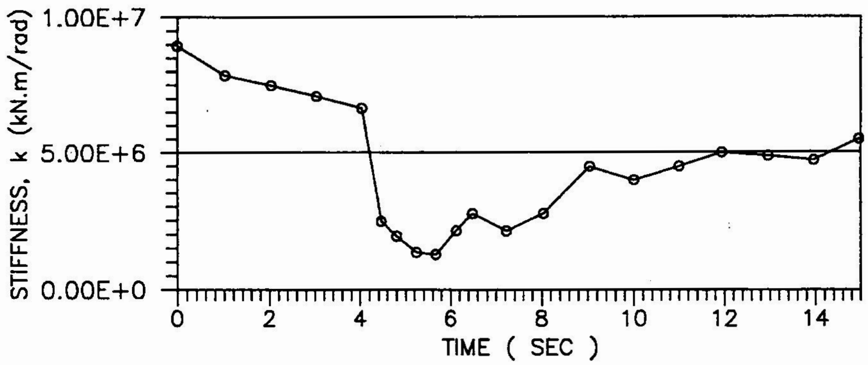

Fig. 6. Time variation of rocking stiffness during strong shaking.

Table 1. Dynamic Stiffnesses of a Single Pile and the 20-Pile Foundation Group

Type of Stiffness		Single Pile (elastic response)	20-Pile Group (elastic response)	20-Pile Group (strong shaking)
Lateral	k_{uu}	77.7 MN/m	520 MN/m	150 MN/m
Cross-Coupling	$k_{u\theta}$	36.7 MN/rad	330 MN/rad	170 MN/rad
Rocking	k_{RR}	--	8400 MN.m/rad	2600 MN.m/rad
Vertical	k_{zz}	376 MN/m	700 MN/m	230 MN/m

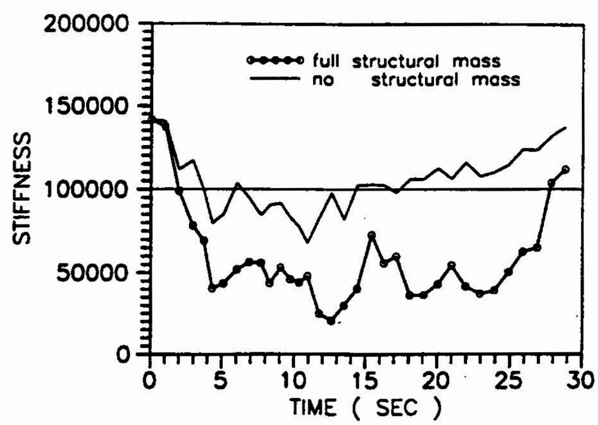

Fig. 7. The effect of the inertia of structural mass on foundation stiffness under strong shaking.

CONCLUSIONS

Analysis of the response of the Painter Street Overpass in Rio Dell, northern California to the 1992 Cape Mendocino-Petrolia earthquake, $M_L = 6.9$, using recorded strong motion data demonstrated clearly the dependence of the stiffness of the pile supported abutments on the dynamic displacements. The stiffnesses during strong shaking were only 1/4 of the stiffnesses estimated from the low strain moduli derived from shear wave velocity measurements. This reduction in stiffness can have a significant effect on the mode shapes characteristic periods of the bridge. It is imperative that these stiffnesses and associated damping should be determined reliably.

A validated method for nonlinear dynamic analysis of pile foundations based on a simplified 3D model of the half space called PILE3D has been presented which can calculate the time-histories of stiffness (and damping) of a pile foundation during an earthquake. The analysis can also include the effects of superstructure inertia on both stiffness (and damping). The time variation of these parameters for a bridge foundation for a given design earthquake allows a more realistic selection of the representative discrete stiffnesses and damping ratios required by structural analysis programs than the rather arbitrary procedures often used in practice.

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