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Soil and Valley Effects in Bridge Foundation Motion

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ABSTRACT

The paper refers to the seismic response of a bridge, founded on piles in a valley characterized by sharp impedance contrast with the underlying soil and by steep lateral boundaries. The bridge is a real-scale experiment as during many earthquake records have been obtained of: (a) the ground motions at the base and the surface of the valley, (b) the seismic response of a bridge pier, and (c) the seismic bending strains developed on its pile foundation. As one-dimensional analyses proved inadequate to capture the two-dimensional valley effects and to predict the recorded ground surface motions, two-dimensional seismic response analyses have been performed using the finite-element method along with the "effective seismic excitation" technique. The available records testify the successful prediction of the free-field motion. The model is then extended to incorporate the pile foundation and the superstructure. In addition, the successful estimation of the free-field field motion in the vicinity of the pile helps the implementation of an improved analytical model for computing the kinematic bending strain. Despite the simplifications in the numerical modeling and the limitations of the analytical solution, results are in agreement with the records. The importance of assessing realistically site specific ground motions for bridge foundation design is demonstrated.

INTRODUCTION

Although piles and pile groups are commonly used in bridge-foundation practice, their seismic distress during earthquakes seems to be not sufficiently understood. An explanation for this could be that a pile (or a pile group) constitutes one of the fundamental parts of a soil-pile-structure interaction system, the complex behavior of which under seismic loading is evident.

In general, the pile distress in a soil-pile-structure interaction system subjected to seismic excitation can be estimated by either (a) direct methods that treat the entire system as an entity, or (b) multi-step methods making use of the principle of superposition.

In the first case, the entire system is modeled and analyzed in a single step. Due to the complexity of the system, only numerical methods can be applied. Despite their ability to cope with irregular geometry and material in-homogeneity and non-linearity, numerical methods are not easy to implement.

On the other hand, as the seismic bending developed on piles is being determined not only by the oscillation of the superstructure, but by the seismic waves as well, the seismic response of a pile can be conceptually decomposed into an *inertial* and a *kinematic* part. The inertial part refers to the inertial loading imposed by the vibrating superstructure, while the kinematic part refers to the bending moments imposed on the pile due to the significant deformations developed on the surrounding soil during earthquake shaking. Thereby, the standard procedure for the seismic soil-pile-structure interaction analysis constitutes of the three consecutive steps:

a. *Ground Response Analysis* to obtain an estimate of the seismic environment to which the system will be subjected during the considered earthquake.

b. *Kinematic Pile Response Analysis* to obtain the response of the pilled foundation in the absence of the inertial forces and moments imposed by the superstructure.

c. *Inertial Soil-Structure Interaction Analysis* to obtain the dynamic response of the superstructure and the loads that this response imposes on the foundation.

For computational convenience, analysis of the inertial response is further subdivided into two consecutive independent analysis steps, as follows:

- computation of the dynamic impedances ("springs" and "dashpots") at the pile head (or the pile-group cap), associated with the swaying (R_x and R_y), rocking (R_{ry} and R_{rx}), and cross-swaying-rocking ($R_{x,ry}$ and $R_{y,rx}$) motion of the foundation, and

- analysis of the dynamic response of the superstructure supported on the "springs" and "dashpots" of the previous step, subjected to the kinematic pile-head motion.

For each of the above analysis steps several alternative formulations have been developed and published in the literature, including numerical and analytical solutions and methods (see Fan 1992; Gazetas & Mylonakis 1998).

In general, the inertial loading imposes on the piles bending moments that attenuate quickly with depth. On the contrary, kinematic loading causes on piles high bending moments at depth, especially in the presence of sharp stiffness discontinuities in the soil profile – a fact that has been verified by recent observations (Okamoto 1983; Mizuno 1987). It is worth noting that in piling engineering, seismically distressed piles were traditionally designed to withstand only the inertial forces (neglecting the kinematic ones), and it was not until recently that both inertial and kinematic pile bending have started being recognized in modern code provisions (EC-8, NEHRP-97).

The existing analytical models for computing the kinematic moments on piles are quite handy, but, as they are based on the simplistic assumption of *uniform static stress field* (Dobry & O' Rourke 1983), they have certain limitations. As the actual stress field is *dynamic* and *non-uniform*, Mylonakis (1999) using wave-propagation theory developed an improved analytical model that is based on dynamic displacement fields. The success of such a model in estimating the developed bending strains (or bending moments) relies on the ability of the geotechnical earthquake engineer to estimate the level of stresses and strains developed on the surrounding soil under *free-field* conditions. That makes ground response analysis the essential first step in the seismic analysis of a bridge foundation.

The dynamic stress field developed on the soil is a function of the characteristics of the excitation at the base of the soil deposit and the *site conditions*. The term 'site conditions' is being used to describe both material and *geomorphic conditions*. Records and analyses (Aki 1988) have shown that – apart from soil-material conditions – the geomorphic conditions tend to alter the amplitude, the frequency content, and the duration of the ground motion, being thereby of particular importance in the seismic design of sensitive structures, such as bridges.

In geotechnical earthquake engineering it is a common practice to estimate the ground seismic response assuming parallel soil layers extended infinitely (one dimensional analysis), neglecting thereby the potential impact of geomorphic conditions. On the other hand, objective difficulties in classifying the large variety of geomorphic features makes it a formidable task to account for these effects in simplistic, code-type prescriptions. To cope with this, two- (or even three-) dimensional site-specific ground response analyses become essential.

The present work is involved with the seismic response of a road bridge in Japan, giving emphasis on the seismic behavior of its pile foundation. The bridge, known as Ohba-Ohashi, is founded in a soft alluvial valley. It is considered to be a real-scale experiment as the ground motion (at the base and the surface of the valley), the response of a bridge pier, and the bending strains developed on the pile foundation of the pier have been instrumentally recorded during many earthquakes. So, the available records may be used for the verification of any numerical or analytical model that may be implemented for the estimation of the free-field motion, the kinematic and inertial loading of piles, and/or the response of the superstructure.

Initially, as one-dimensional analyses proved inadequate to capture the geomorphic features of the valley and cannot thereby adequately explain the level of strong shaking at the ground surface, a two-dimensional ground response analysis is performed for the estimation of the free-field motion. The model is based on the finite-element method and incorporates the "effective seismic excitation" technique (Bielak & Christiano 1984; Loukakis 1988). The verification of the model has been performed by the successful reproduction of three recorded ground surface motions (using as input the recorded ground base motions).

The finite-element model is then extended to incorporate, in a simple but realistic way, firstly the pile foundation and then the pile-superstructure system. The numerical model developed is able to reproduce accurately both the recorded kinematic and inertial strains developed on the piles, and the acceleration time-histories recorded on the superstructure.

Finally, given the free-field stresses and strains developed in the vicinity of the pile, the analytical model developed by Mylonakis (1999) is implemented. The model takes into account the dynamic displacement field, incorporating realistically the dynamic characteristics of the excitation, as well as the geometry, inertia, and damping of the soil. The kinematic bending strains predicted are in a very good agreement with the corresponding recorded ones, proving the efficiency of the simple analytical model.

THE BRIDGE FOUNDATION AND THE VALLEY

Pile foundation

The bridge is about 600 meters long and 11 meters wide. It is supported by seventeen piers and its girder is continuous from pier P5 to pier P8. Piers P5, P7, and P8 are equipped with moveable bearings, while pier P6 is of the fixed-shoe type. Figure 1 sketches the plan view and cross section of the bridge between pier P5 and pier P8, and the arrangement of the accelerometers. Of interest in this study is pier P6, which is supported by a pile group consisting of (8 × 8 =) 64 steel piles, 32 of which are batter, as shown in Figs 2 & 3. The piles have a ring cross-section, and the following dimensions: length = 22 m, diameter = 0.60 m, wall thickness = 9 mm (for the vertical piles) and 12 mm (for the batter piles). The strain gauges are installed along one vertical and one batter pile at four depths, each of which has four measuring points along circumference.

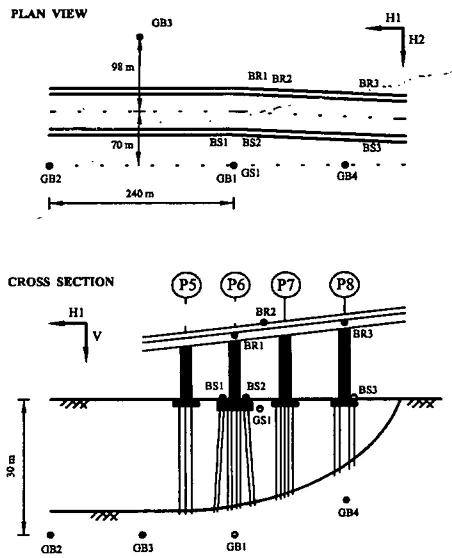

Fig. 1. *Plan view and cross section of the bridge between pier P5 and P8 (sketch only).*

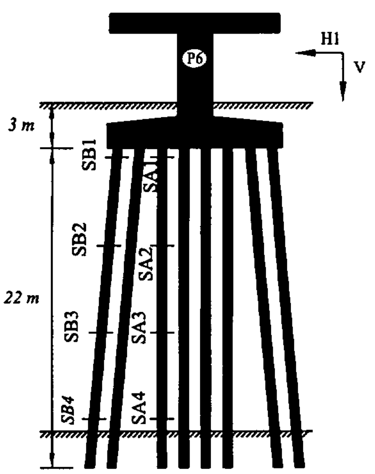

Fig. 2. *Pier P6 with the location of the strain gauges.*

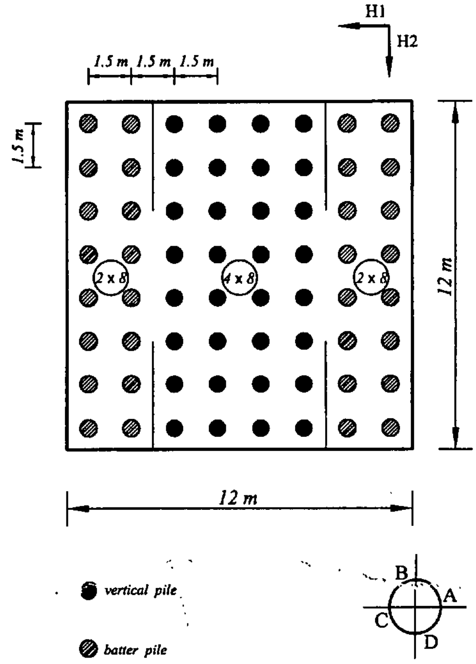

Fig. 3. *Arrangement of the piles supporting pier P6.*

Geotechnical data

The soil profile obtained from a borehole near pier P6 is shown in Fig. 4. The top layers that piles penetrate consist of extremely soft alluvial strata of humus and silt. Despite the soil improvement performed before the bridge construction, the standard penetration test values N_{SPT} were almost null, while the shear wave velocity measured by down-hole tests was ranging between 50 to 100 m/s. The depth of the soft soil layers is between 22 and 25 meters. The underlying substratum of diluvial deposits consists of stiff clay and fine sand, and it has much higher bearing capacity, with shear wave velocity being 400 m/s and N_{SPT} values over 50. The ground water table is almost one meter below the ground surface, while the water content of the top layers exceeds 100%. It worth noting that the top layers are characterized by large to extremely large plasticity index *PI*, being thereby far more elastic than the standard clays (Vucetic & Dobry 1991).

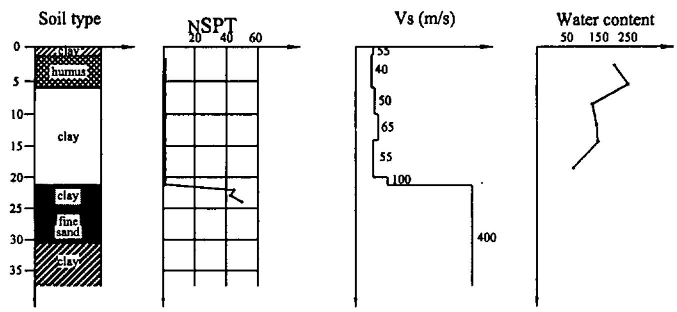

Fig. 4. *Soil profile characteristics under pier P6.*

RECORDED MOTIONS & STRAINS

The earthquake observations were carried out by the Institute of Technology of Shimizu Corporation, Japan. From April 1981 to April 1985 fourteen earthquakes were recorded. Five accelerometers had been installed on the valley, six on the bridge, and eight strain gauges were installed on the pile foundation. Three of the earthquakes (A, B, and C) were near-distant events and their time histories are being used in the analyses. Among the 14 recorded earthquakes, the one that gave the highest ground surface acceleration (0.114g) was the Kanagawa – Yamanashi – Kenzakai earthquake (earthquake C), with magnitude M_{JMA} = 6, and epicentral distance R = 42 km. For the ground response analyses the three of them have been used, while only the earthquake that gave the higher acceleration levels (earthquake C) has been used for the estimation of the bending strains and the response of the superstructure. The free-field motion has been adequately recorded with accelerometers installed at the ground surface (e.g. GS1 near pier P6), and at the base of the superficial deposits (e.g. GB1, GB2, GB3, GB4). The recorded acceleration time-histories at the base of the profile (GB1) and at the ground surface (GS1) during earthquake C are shown in Fig. 5(a), while Fig. 5(b) illustrates the corresponding elastic response spectra.

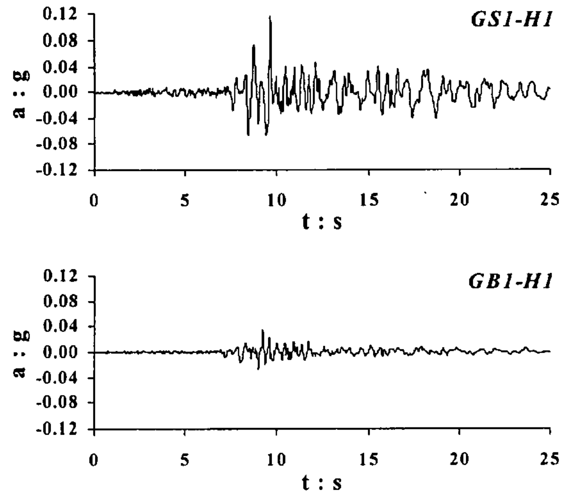

Fig. 5(a). Acceleration time-histories recorded during earthquake C at the base (GB1) and the surface (GS1) of the valley for the direction H1.

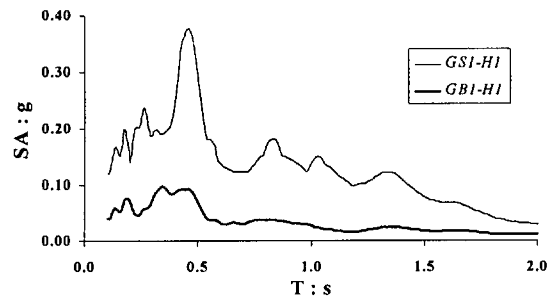

Fig. 5(b). Elastic response spectra (5% damping) of the records of earthquake C.

Three accelerometers (BS1, BS2, and BS3) have been installed on the pile caps, two (BR1, BR3) are on the bridge piers P6 and P8, respectively, while an extra one (BR2) is located on the girder, between the piers P6 and P7. Figure 6 shows the acceleration time-history recorded on the superstructure (accelerometer BR2) during earthquake C.

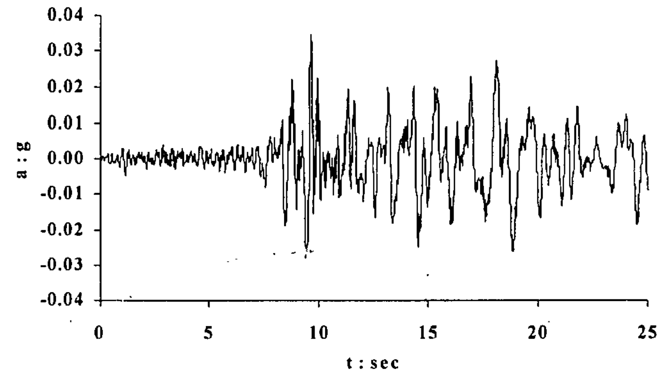

Fig. 6. Acceleration time-history recorded on the bridge gutter (accelerometer BR2) during earthquake C.

The pile distress was traced by strain gauges that recorded the bending strains at the two directions (H1 and H2). Four instruments (SA1, SA2, SA3, and SA4) were installed along one of the vertical piles of pier P6, while four more instruments (SB1, SB2, SB3, and SB4) were placed along one of the batter piles. Figure 2 is indicative of the location of the strain gauges, while Fig. 7 shows the distribution of maximum bending strain recorded on a vertical pile of pier P6 during earthquake C.

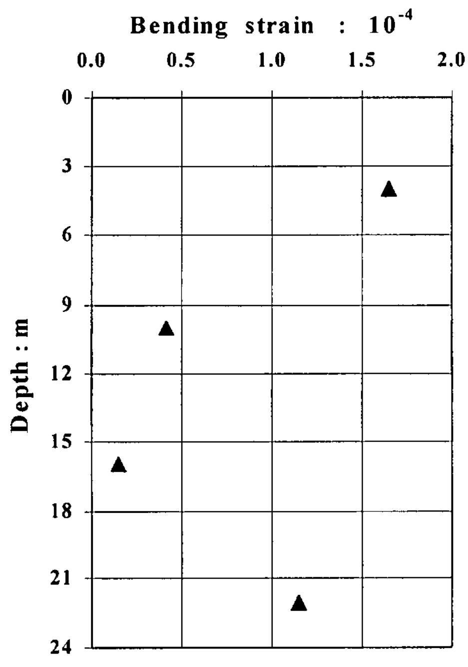

Fig. 7. Maximum bending strains recorded on one of the vertical piles of pier P6 during earthquake C.

GROUND RESPONSE ANALYSIS

As the valley is characterized by steep lateral boundaries (Fig. 8(a)), one dimensional analyses proved insufficient to estimate the recorded free-field motion (Fan 1992). To this end, two-dimensional finite-element analyses were performed for the simulation of the seismic response of the valley. As shown in Fig. 8(b), the geometry and the soil properties of the valley were simplified, assuming a trapezoidal shape and a mean low-strain shear wave velocity of the soil stratum equal to 60 m/s. In the same figure it is shown the point of interest P1, which is located on the surface and coincides with the location of receiver GS1. All the analyses are based on the assumption of linear visco-elastic behavior of the soil, which is quite acceptable for earthquakes that produce relatively low peak values of horizontal accelerations and/or for clayey deposits with very high plasticity index (as is the case here), since these soils develop non-linearity only at higher deformation levels (Vucetic & Dobry 1991).

The finite-element mesh generation (Fig. 9) has been produced by the automatic mesh generator NeGe (1992), capable of handling material and geometry irregularities. The mesh consists in general of 6-noded triangular elements, while 4-noded quadrilateral elements have been used where the piles will be later placed. The size of all the elements has been tailored to the wavelength of the propagating waves.

The approach for the finite element analysis using ABAQUS was based on the "effective seismic excitation" technique developed by Bielak & Christiano (1984) and implemented by Loukakis (1988). With this approach, the problem of seismic response of a two-dimensional valley is transformed into an equivalent one, in which the source is located in the interior of the domain of computation. The advantage of the technique is that the artificial boundary is needed only to absorb the scattered energy of the system, while the seismic excitation is introduced directly within the region of interest. In addition, the artificial boundary may be placed as close to the examined region as the accuracy of the boundary for absorbing outgoing waves permits, as no approximation is involved in the specification of the free field motion. This option permits the discretization of a limited area of the surrounding 'rock', minimizing thus substantially the computational cost of the analysis.

By trial-and-error it was found that a material damping of the upper soil layer of the order of 3 % gives the best results in all three cases. In ABAQUS material damping is of Rayleigh type, which means that the damping ratio is frequency dependent (Asimaki 1999).

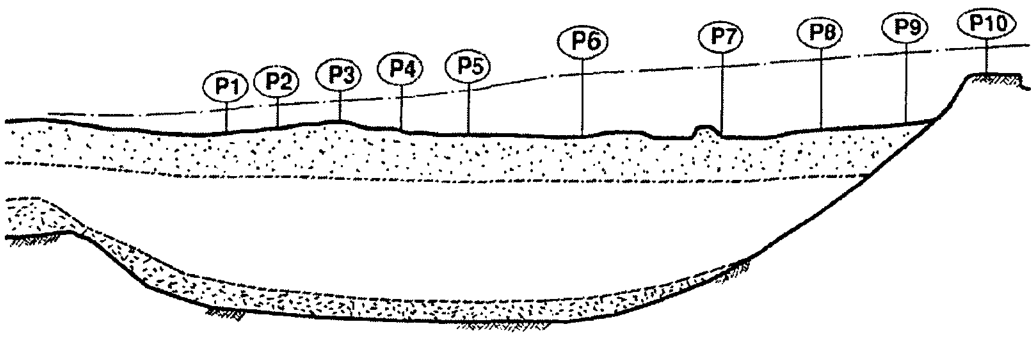

Fig. 8(a). *Longitudinal section of the valley. The vertical scale is exaggerated.*

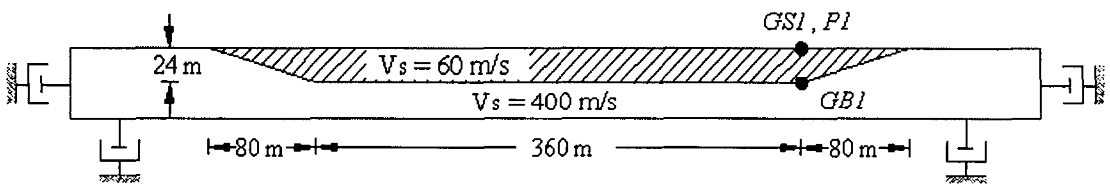

Fig. 8(b). *Idealized geometry of the valley.*

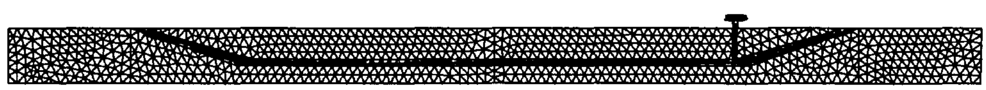

Fig. 9. *Finite-element discretization.*

To verify the model, each of the recorded ground base (GB1) acceleration time histories of the three different earthquakes (A, B, C) was applied as input excitation, and the recorded acceleration time-histories at the ground surface (GS1) were obtained. As there were no records available on the surface of the valley, other than GS1, the satisfactory comparison between records and analyses at this location offers a first validation for the model and the method of analysis. Figs 10(a) & 10(b) depicts the results obtained for point P1 in the case of earthquake C. As it will be seen in the sequel, the free-field strains estimated at the vicinity of the soil layer interface will be the input for the analytical estimation of the kinematic distress of piles.

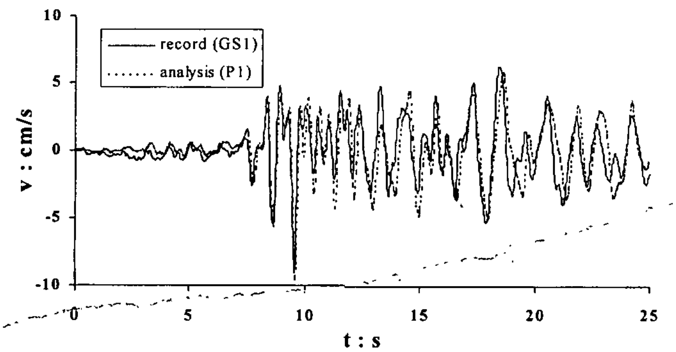

Fig. 10(a). Velocity time-histories for earthquake C: comparison of the record (GS1) with numerical results (point P1).

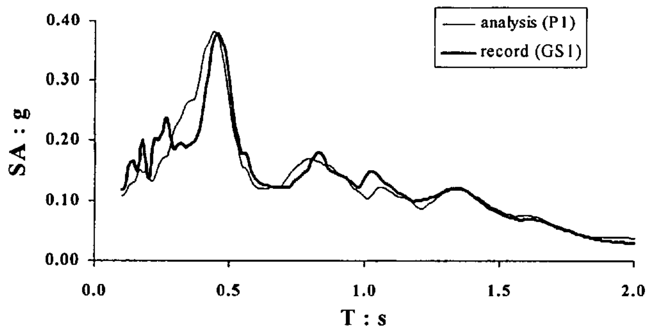

Fig. 10(b). Response spectra for earthquake C: comparison of the record (GS1) with numerical results (P1).

NUMERICAL SIMULATION OF THE SOIL-PILE-STRUCTURE INTERACTION SYSTEM

In order to estimate numerically the kinematicaly imposed bending moments developed on the piles, the finite-element model was then extended by incorporating, in a simple but realistic way, the pile foundation. The geometry and the material properties of the soil were kept exactly the same with the ones used in the ground response analyses. As it was impossible to perform a three-dimensional finite-element model (that could possibly take into account the entire pile group), the new model was based on the following simplistic assumption: plane-strain conditions were considered, and to this end an "equivalent diaphragm" was used. The diaphragm is characterized by longitudinal stiffness $E_P I_P$, equal to the one that characterizes the piles per current meter (in the transversal direction). The 4-noded quadrilateral elements used for the modeling of the "equivalent diaphragm" are equipped with incompatible modes as the enhancement of incompatible modes in the lower-order quadrilateral continuum elements improves their bending behavior. The maximum kinematic bending strains developed close to the pile tip during earthquake C are in consistence with the recorded bending moments at depth.

The finite-element model is then extended one step further, as the bridge pier and the corresponding mass of the girder were incorporated as an additional single-degree-of-freedom system. The entire soil-pile-structure interaction system is then analyzed. In Fig. 11 the maximum bending strains predicted from the numerical simulation are being compared with the recorded bending strains. In Fig. 12 the acceleration time-histories predicted for the superstructure are being compared with the recorded time-histories (BR2) for earthquake C.

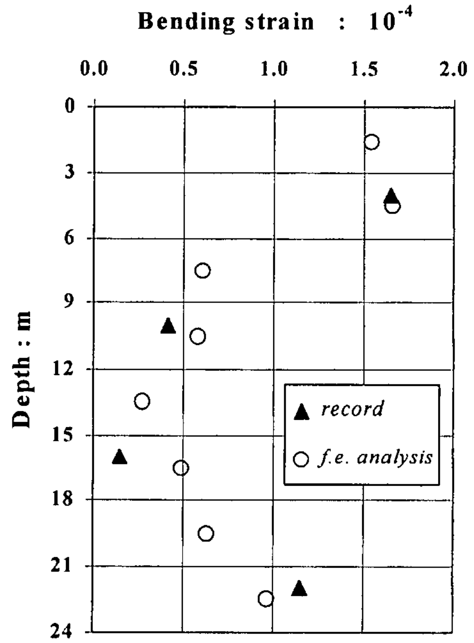

Fig. 11. Computed maximum bending strains developed on piles for earthquake C.

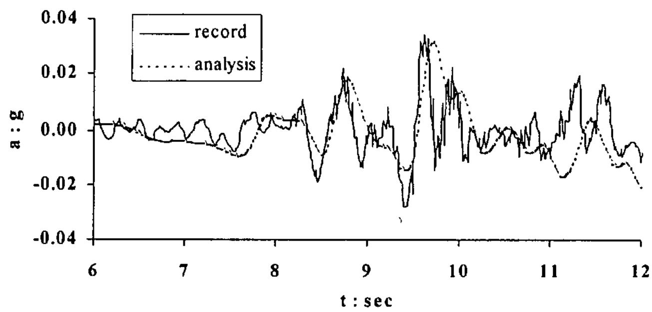

Fig. 12. Acceleration time-history predicted for the superstructure in comparison with the record.

ANALYTICAL MODEL FOR THE KINEMATIC DISTRESS OF PILES

Model proposed by Mylonakis (1999)

As it was noted, kinematicaly imposed bending moments of piles tend to be amplified in the vicinity of interfaces of soils characterized by high impedance contrast. Contrary to the inertial induced moments, the kinematic moments may be high and may lead to damage. Quite recently, Mylonakis (1999) developed an improved closed-form analytical solution for the prediction of the kinematic bending strain developed on piles, for a two-layer soil profile. His model, based on the wave-propagation theory, takes into account the dynamic displacement field, incorporating thereby the dynamic characteristics of the excitation as well as the geometry, inertia, and damping of the soil. A simplified analysis procedure is proposed for a preliminary assessment of kinematic pile bending moments. The procedure involves the following five steps:

Step 1: Perform a free-field ground response analysis to estimate the peak shear strain, γ_1, at the soil layer interface.

Step 2: Determine the relative stiffness between the two soil layers, G_2/G_1, the pile-soil stiffness contrast, E_p/E_1, and the pile embedment ratio, h_1/d). Strain-compatible soil moduli (computed in Step 1) can be used to this end.

Step 3: Determine the spring coefficient k_1 using, for instance, the following equation:

$k_1 = \delta E_1$ where

$$\delta \approx \frac{3}{1-\nu^2}\left(\frac{E_p}{E_1}\right)^{-1/8}\left(\frac{L}{d}\right)^{1/8}\left(\frac{h_1}{h_2}\right)^{1/12}\left(\frac{G_2}{G_1}\right)^{-1/30}$$

where: E_1 = Young's modulus of layer 1, L = pile length, h_1 & h_2 = thickness of layers 1 and 2, respectively.

For relatively long piles (L/d about 40) and two soil layers of approximately equal thickness ($h_1 = h_2$), the previous equation takes the simpler form:

$$\delta \approx 6\left(\frac{E_p}{E_1}\right)^{-1/8}$$

Step 4: Using the parameters obtained in Steps 2 and 3, determine the strain transfer ratio from the following equation:

$$\frac{\varepsilon_p}{\gamma_1} = \frac{(c^2 - c + 1)\left\{\left[3\left(\frac{k_1}{E_p}\right)^{1/4}\left(\frac{h_1}{d}\right) - 1\right]c(c-1) - 1\right\}}{2c^4\left(\frac{h_1}{d}\right)}$$

where

$$c = \left(\frac{G_2}{G_1}\right)^{1/4}$$

Step 5: Based on steps 1, and 4, determine the peak kinematic bending strain and the corresponding pile bending moment at the interface.

Implementation for the Ohba-Ohashi bridge case

Step 1: According to the two-dimensional ground response analysis described in Section 4, the shear strains γ_1 developed during earthquake C close to the soil layer interface is of the order of 3.5×10^{-4}.

Step 2: The relative stiffness between the two soil layers of the valley is $\frac{G_2}{G_1} = \frac{288{,}000\ kPa}{5{,}400\ kPa} \approx 53$,

where $G = \rho V_S^2$.

So, $c = \left(\frac{G_2}{G_1}\right)^{1/4} \approx 2.7$

For one of the vertical piles of the Ohba-Ohashi bridge, the pile-soil stiffness contrast is

$\frac{E_p}{E_1} = \frac{22{,}941{,}438\ kPa}{16{,}200\ kPa} \approx 1400$, while the pile embedment ratio $\frac{h_1}{d} \approx 40$.

Step 3: Using $\delta \approx 6\left(\frac{E_p}{E_1}\right)^{-1/8} = 6 \times (1400)^{-1/8} = 2.43$

$k_1 = \delta E_1 = 2.43 \times 16{,}200 \approx 40{,}000$

Step 4: So the strain transfer ratio can be computed: $\frac{\varepsilon_p}{\gamma_1} = 0.14$

Step 5: Combining the results of Step 1 with the ones of Step 4: $\varepsilon_p \approx 1 \times 10^{-4}$, which is in accordance with the recorded kinematic strain for earthquake C (see Fig. 7).

CONCLUSIONS

The seismic response of a road bridge founded on piles has been examined. The importance of assessing realistically site specific ground motions for bridge foundation design is demonstrated.

The bridge is a real-scale experiment as during many earthquake records have been obtained of: (a) the ground motions at the base and the surface of the valley, (b) the seismic response of a bridge pier, and (c) the seismic bending strains developed on its pile foundation.

Ground response analyses using the finite-element method capture the two-dimensional valley effects and reproduce successfully the recorded ground surface motions.

Then, the kinematic bending strains recorded on the pile base can be estimated by either an extension of the finite-element model, or a simple analytical approach that takes into account the dynamic displacement field of the valley.

The presence of the superstructure, as it was expected, gives rise to inertial bending strains close to the pile cap.

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