01 Jun 1993 - 06 Jun 1993

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Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, SOA No. 2

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SYNOPSIS: The paper outlines the recorded response to numerous earthquakes of the pile foundation, the supporting soil, and the superstructure of the main pier of a road bridge. The records include free-field accelerograms at the ground surface and the base of the alluvial deposit, accelerograms on the footing and the superstructure, and the bending and axial strain histories at several depths along two of the sixty-four piles. Recently developed methods of seismic analysis are used in interpreting the recorded data. Extensive comparisons are made between theory and measurements. Successes and failures of the theory are discussed. Emphasis is given to the distribution of seismic bending strains along the pile; the theoretically-anticipated concentration of such strains at an interface between two layers with sharply-differing soil stiffnesses is fully confirmed.

INTRODUCTION

The need to use field observations to calibrate/validate theoritical methods before they can be adopted in practice is a deeply-entrenched "principle" in geotechnical engineering. With pile foundations subjected to seismic shaking, such observations can not be performed easily; they require measurements (in the form of accelerograms and strain-history records) or, perhaps, post-earthquake retrieval of piles. These are not routine operations, and they require substantial effort and cost.

Thus, a well-documented case history involving measurements of pile, footing, and structure response to earthquakes can be an invaluable resource. This is the type of information obtained by Tazoh & co-workers (1984, 1988) on the main pier of Ohba-Obashi Bridge in Japan. This article summarizes the recorded response and uses it to assess the predictive power of a number of available theoretical methods.

We first outline a general methodology for seismic soil--pile--footing--structure interaction. In addition to providing a convenient means of computing the seismic response, this methodology offers an attractive conceptual framework for interpreting the field measurements and, eventually, for designing pile foundations against earthquake shaking.

GENERAL FRAMEWORK FOR SEISMIC SOIL--PILE--STRUCTURE INTERACTION ANALYSIS

Of interest is the response of a simple structure and its pile foundation when the whole system is subjected to seismic "incoming" waves, as is shown on the top sketch of Fig. 1. While available methods usually require that only vertically-propagating shear (S) waves be considered as excitation, reality is undoubtedly more complex, with obliquely-incident and surface waves carrying some of the arriving seismic energy.

Whether the excitation consists of vertical or oblique waves, the system of Fig. 1 can be conveniently analysed in three consequtive steps, as illustrated in the same figure:

1. Determine the kinematic response, involving pile deflections and the motion of the foundation, in the absense of a superstructure. This so-called "Foundation Input Motion" (FIM) includes translational and rotational components.

2. Determine the dynamic impedances ("springs" and "dashpots") associated with swaying, rocking, and cross-swaying-rocking oscillations of the foundation.

3. Compute the inertial response of the superstructure and the forces/moments transmitted onto the foundation, for a base motion equal to the FIM of step 2 and with the structure supported through the "springs" and "dashpots" of step 2.

For each step of the analysis several alternative formulations have been developed and published in the literature, including finite-element formulations, boundary-element, semi-analytical and analytical solutions, and a variety of simplified methods. Table 1 lists some of the available methods, and underlines the ones utilized in this case history. More details on these methods can be found in the thesis of Fan (1992) and in the article by Gazetas et al (1992).

THE OHBA-OHASHI BRIDGE CASE HISTORY

The Ohba-Ohashi bridge is located in Fujisawa City of Kanagawa Prefecture, near Tokyo. It is supported by eleven piers and is 485 meters long and 10.8 meters wide. The girder is continuous from pier 5 to pier 8. Piers 5, 7, and 8 are equipped with movable bearings, but pier 6 is of the fixed-shoe type. Fig. 2 sketches the plan view and cross section of the bridge between pier 5 and pier 8, and the arrangement (location) of accelerometers.

Of interest in this study is Pier 6, supported on 8 x 8 = 64 steel pipe piles (32 batter and 32 vertical piles), as shown in Fig. 3. The piles have the following characteristics: diameter = 0.60 m, length = 22 m, and wall thickness = 9 mm (for the vertical piles) or 12 mm (for the batter piles). The strain gauges were installed along one vertical and one batter pile at four depths, each of which had four measuring points along the circumference.

The river runs between pier 6 and pier 7, and the soil profile obtained from a borehole near pier 6 is shown in Fig. 4. The ground water table is 1 m below the ground surface. The top layers through which the piles penetrate consist of extremely soft alluvial strata of humus and silty clay, the standard-penetration-test N values of which remained zero after a 6-month preloading, while the shear wave velocity reached values in the range of 50 m/s to 60 m/s (measured in a down hole test). The total thickness of the alluvium is about 22 meters. The underlying substratum of dilluvial deposits of stiff clay and sand is much stiffer, with shear wave velocity of about 400 m/s and N values in excess of 50. Among much stiffer, with shear wave velocity of about 400 m/s and N values in excess of 50. Among the other soil characteristics, please note the very high water content of the topmost soil layers: 100% - 250 %, with correspondingly small soil densities. Preloading was necessary before installing the piles!

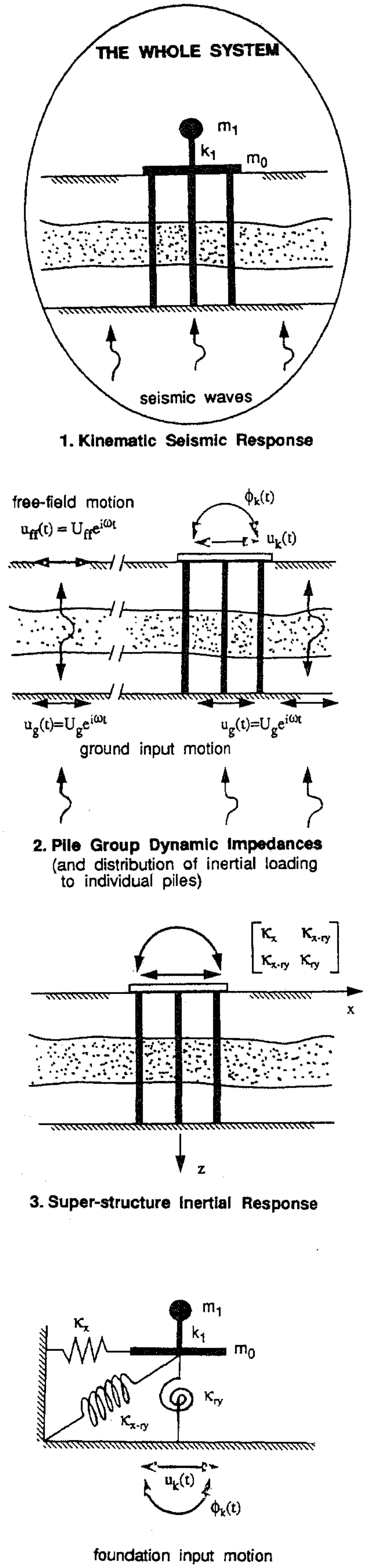


FIG. 1 General Framework for Seismic Soil--Pile-Foundation--Structure Interaction Analysis

Table 1. General Methodology for Seismic Soil--Pile-Foundation--Structure Interaction

1. DETERMINATION OF KINEMATIC SEISMIC RESPONSE
a. FREE-FIELD (SITE) RESPONSE
One dimensional elastic or inelastic wave propagation theories
Two and three dimensional elastic wave propagation theories
b. SINGLE PILE RESPONSE
*** Beam-on-Dynamic-Winkler-Foundation (BDWF) model**
*** Extended Tajimi formulation**
Finite-element formulations
Semi-analytical and boundary-element-type formulations
c. PILE GROUP RESPONSE
*** Simplified wave-transmission model**
*** Extended Tajimi formulation**
Semi-analytical and boundary-element-type formulations
2. DETERMINATION OF PILE-HEAD IMPEDANCES
a. SINGLE PILE
*** Simple expressions**
*** Extended Tajimi formulation**
*** BDWF model**
Novak's plane-strain formulation
Novak-Nogami's axisymmetric formulation
Finite-element formulations
Semi-analytical and boundary-element-type formulations
b. PILE GROUP
*** Superposition method (using dynamic interaction factors)**
*** Extended Tajimi formulation**
Finite-element formulation
Other simplified solutions
Semi-analytical and boundary-element-type formulations
3. DETERMINATION OF SUPERSTRUCTURE SEISMIC RESPONSE
Must account for SSI through frequency-dependent foundation "springs" and "dashpots" from step 2 and must use the seismic response from step 1 as foundation excitation.

* Methods addressed, developed, or compared in this study are shown in bold face.

Earthquake observations were carried out by the Institute of Technology of Shimizu Corporation, using the installed array of accelerometers and strain-meters. From April 1981 to April 1985 fourteen earthquakes were recorded. From those, five selected earthquakes are analysed in the sequel; their characteristics are listed in Table 2. These earthquakes can be roughly classified into two categories: near-distant (earthquakes 11, 12, and 13) and far-distant (earthquakes 4 and 7). Earthquake 12 induced the largest Peak Horizontal Ground Surface Acceleration, PHGSA = 0.115 g.

THE RESPONSE OF THE FREE FIELD

As explained in Fig. 1, starting point of a complete solution to a soil-pile-structure interaction problem is to estimate the seismic response of the free-field. Moreover, it is only through successful analyses of the free-field response that confidence can be gained on the soil parameters needed for the subsequent soil-pile interaction analyses and on the type of waves that produce the seismic shaking.

Fortunately, in Ohba-Ohashi, the free-field response has been adequately recorded with the accelerographs GB1, GB2, GB3, GB4 and Gs1 (Fig. 1). Some characteristic recording are given herein for the strongest shaking (event 12). The acceleration histories GB1 (at the base) and GS1 (at - 1 m from the ground surface) are given in Figs. 5 and 6 for both H1 and H2 directions. Notice that peak ground accelerations are amplified 3.4 times in direction H1 (parallel to bridge axis) and 2.4 times in H2 (perpendicular to bridge axis). In Table 2 one should observe that during the weaker events (Erathquake No2 4, 7, 11 and 13) peak ground accelerations were amplified by a factor in the range of 3 to 5.

Our first attempts to analytically reproduce GS1 using 1-D wave propagation analyses with the recorded GB1 as the input motion have failed spectacularly! Two different sets of such analyses have been performed, with the shear modulus-versus-strain and damping-versus-strain relations being the main variable. Initially, we assumed that the "standard"

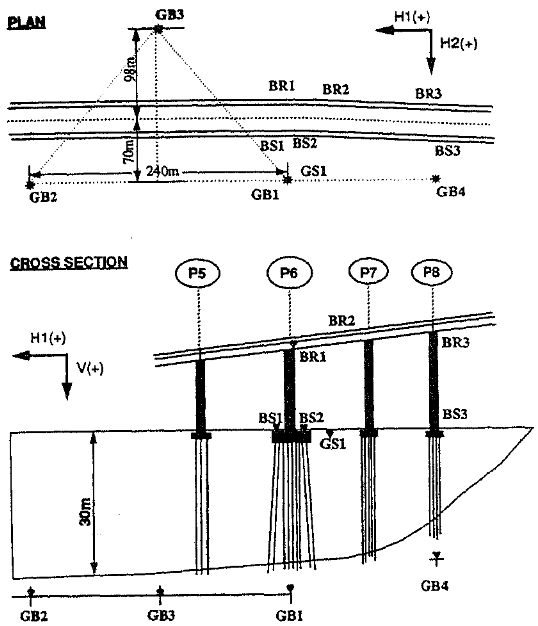


FIG. 2 Plan and longitudinal section of the Ohba-Ohashi Bridge near Pier 6, showing the location of the recording instruments GB1, GB2, GB3, GB4, GS1, BS1, BS2 and BR1.

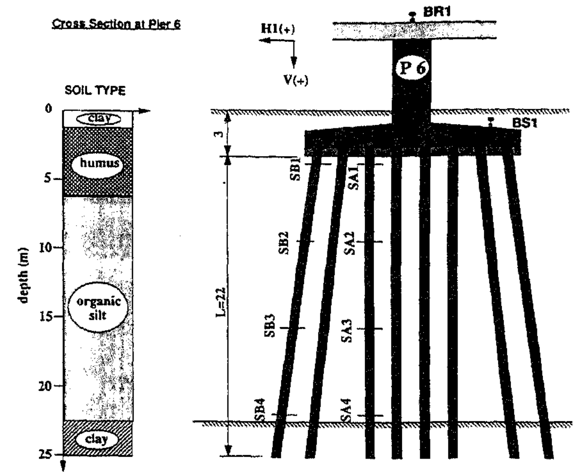


FIG. 3 Cross-section of Pier 6 with its supporting piles and the soil profile. Also shown: the location of the strain gauges in a vertical (SA1 - SA4) and a racker (SB1 - Sb4) pile.

relations for clayey soils are applicable. The resulting ground surface motions reached peak accelerations (PGA), in both directions, of only about 0.03 g, compared with the recorded 0.114 g and 0.0092 g, respectively. We found out that a major cause of this huge under-prediction of the recorded soil amplification was the strong soil nonlinearities developing in these analyses. As a result: (i) soil modulus was being (unrealistically) reduced, enlarging the natural fundamental perid of the deposit beyond the dominant periods of the base (input) motion; (ii) soil damping was being (also unrealistically) increased to values exceeding 15%.

When the complete set of soil data was studied carefully, we realized that the initial soil properties for large strains were inappropriate, as the clays in Ohba-Ohashi were of large to extremely large "plasticity index": PI > 100% (up to PI = 250%). As has been shown following the Mexico City 1985 Earthquake, such high plasticity-index clays are far more *elastic* than the "standard" clays. From the well-known curves of Vucetic & Dobry (1991) one can see the "quasi-elastic" behavior of such clays for shear strains of up to about 0.002. In particular, even at this very large strain, damping values remain below 5%. Thus, a new set of 1-D analyses were performed using the modulus degradation and damping curves of Vucetic-Dobry for the appropriate plasticity indeces, while allowing for some parametric variation of material properties. The results improved, but not enough! Pak surface accelerations reached 0.07 g only.

To find out the cause of the remaining under-prediction, Fourier Spectra Amplification Ratios

$$\mathbf{A} = \mathbf{F}(GS1) \;/\; \mathbf{F}(GB1)$$

where F( ) denotes Fourier amplitude, are compared in Fig. 5. It is seen that the "recorded" **A** spectra show *a large number of substantial peaks* between the 1-D fundamental natural period (T = 1.3 s) and the 1-D second natural period (T = 0.4 s) of the deposit. At lower periods the "recorded" spectra have even larger number of peaks (at well-seperated periods) than the 1-D analysis.

These numerous peaks in the recorded AR spectra stem form the 3-D shape of the valley. Indeed, a longitudinal cross-section of the ground, reveals that the base of the soft soil deposit is not horizontal (as 1-D analyses implicitly assume) but dips at an angle of about 15 degrees close to Pier 6. Evidently, we are dealing with a relatively narrow alluvial valley, one edge of which has a relatively steep slope.

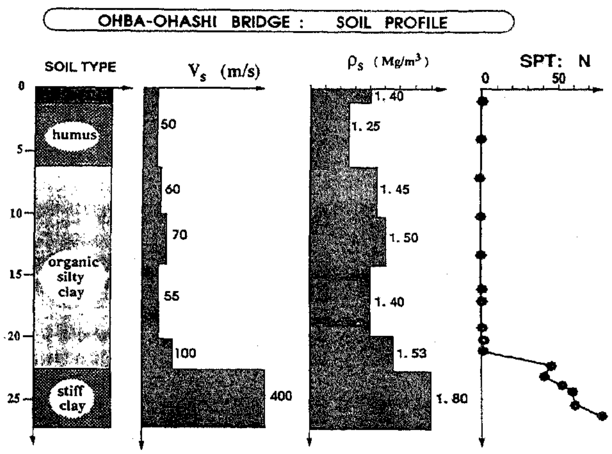


FIG. 4 Soil profile with mass density and SPT N values versus depth

Empirical and theoretical evidence, compiled in recent years, shows that earthquake ground motions on the surface of such valleys are *stronger and longer* than the motions predicted with 1-D wave propagation theories or recorded/experienced on top of very wide plains. Several wave-propagation phenomena, akin to the 3-D geometry, have been recognized as producing these deleterious effects: wave *"focusing"* tends to amplify the motion primarily near the center of the valley; surface waves, generated at the (steep) edges, propagate back and forth across the valley, attenuating slowly and thereby prolonging the motion; *"trapping"* of obliquely-incident body waves amplifies the motion experienced near the edges of the valley. One of more of these phonomena were clearly evident in several recent earthquakes. For example, in Mexico City, large Fourier-spectral amplifications and extremely large strong-motion durations observed in accelerograms recorded on the "lake bed" during the 1985 Michoacan earthquake, were attributed to variations in the thickness of the soft clay layers (Sanchez-Sesma et al 1988, Faccioli 1991). In Caracas, the high concentration of damage in the area of Palos Grandes during the 1967 earthquake was attributed to the steep slope (dip of about 35 degrees) of the

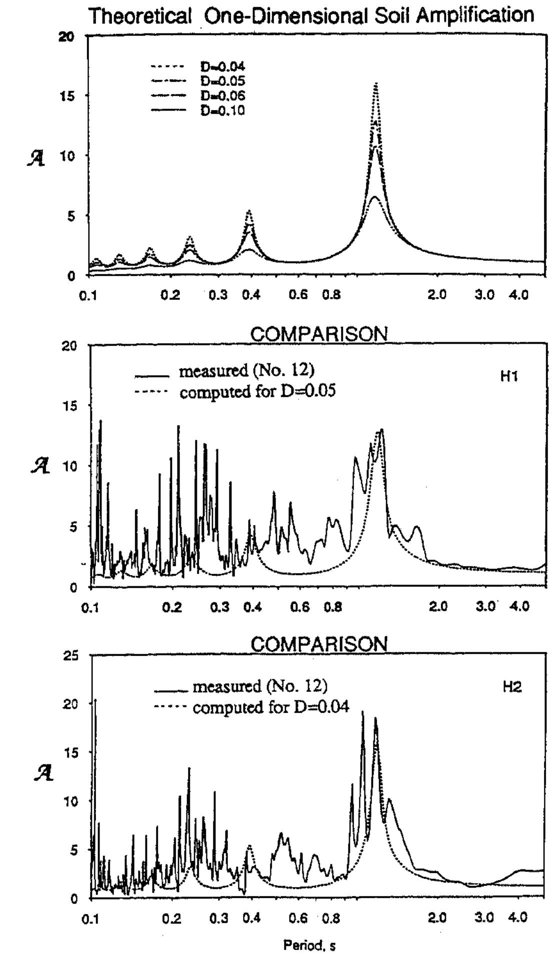


FIG. 5 Comparison of linear 1-D wave propagation results with measured amplification ratios in the two horizontal directions (H1 and H2).

supporting bedrock at the northern boundary of the 3-km long sedimentary valley (Papageorgiou and Kim 1991). In the Armenia 1988 disastrous earthquake, the disproportionately large degree of damage observed in one small region in the city of Kirovakan, was recently attributed to the underlying triangularly-shaped sedimentary basin---maximum soil depth: 150 m, width-to-depth ratio: 5 (Yegian, Ghahraman & Gazetas, 1993).

One can argue that 3-D effects have also played a role in the records of Ohba-Ohashi. Records of microtremors, performed by Tazoh & co-workers (1988), qualitatively provide a strong indication. Indeed, at a point near the middle of the valley where the dilluvial-layer base is flat, the Fourier spectra of the recorded tremors show essentially only one peak at T = 1.4 s. By contrast, near P6, at the location of station GS1, three distinct peaks appear between 1.1 and 1.4 seconds.

Theoretical studies have additionally provided evidence on the role of the sendimentary-basin 3-D geometry. Studies by Tazoh et al (1988) with a 2-D F.E. model, and by Ohtsuki et al (1984) with a hybrid numerical model, have explained the additional peaks in terms of natural periods of the whole deposit. On the other hand, using the simplified "geometric" method of solution of Sanchez-Sesma et al (1988), applied by Fan (1992) to the idealized geometry of the Ohba-Ohashi basin, shows the development of a broad-band peak in the Fourier spectra ratio around T = 0.90s, as a result of many oblique waves emanting (upon transmission and reflection) at the base of the deposit. It may well be that in reality both types of phenomena (response of the valley as a whole and generation of oblique wave rays) have conributed to the response of ground surface and the unexpectedly high recorded accelerations. More detailed studies are currently underway. Note, however, that our subsequent soil-pile-structure interaction analyses assume S waves that are transmitted vertically into the soft deposit.

FOOTING AND PIER RESPONSE TRANSFER FUNCTIONS

The response of the bridge at the location of the pier P6 was recorded at BR1 while the vertical and horizontal accelerations at two sides of the footing were recorded at BS1 and BS2. From the Fourier Spectra of the recorded motion six different transfer functions were calculated as the ratios:

- BS1/GS1 (footing to free field surface)
- BS1/GB1 (footing to diluvial base)
- BR1/GS1 (bridge to free field surface)
- BR1/GB1 (bridge to diluvial base)
- BR1/BS1 (bridge to footing)
- GS1/GB1 (free field surface to base)

The latter is none else than the already studied free field amplification function.

For Earthquake No. 12 and the H1 (longitudinal) direction these six transfer functions from the recorded motions are compared with one set of our theoretical predictions in Fig. 6. Please note, however, several additional comparisons are available in the thesis of Fan (1992), where the following six different methods of pile-soil-foundation analysis were performed:

(1) **"Rigorous Method for Piles, No Footing"** --- using pile impedances from rigorous solution and ignoring the contribution from the footing of the pier

(2) **"Rigorous Method for Piles, Plus 50% Footing"** --- using pile impedances from rigorous solution plus 50% of the footing impedances ofthe pier footing computed as if it were acting alone

(3) **"Rigorous Method for Piles, Plus 100% Footing"** --- using pile impedances from rigorous solution plus 100% of the impedances of the footing acting alone (upper bound)

(4) **"Simplified Method for Piles, No Footing"** --- using pile impedances from the simplified solution (dynamic interaction factors) but ignoring the contribution of the footing impedances

(5) **"No Pile-to-Pile Interaction, No Footing"** --- superimposing the impedances of all piles without any pile-to-pile interaction

(6) **"Static Interaction Factor, No Footing"** --- using static interaction factors (Poulos & Davis, 1980)

Details on these methods of analysis can be found in Gazetas et al (1992) and Fan (1992). The theoretical results that are compared in Fig. 6 are for the fourth of the above-mentioned methods ("Simplified Method / Piles Only"), but the following conclusions are drawn here from the complete comparative study.

We first note that in the comparison emphasis is placed on the key features of the transfer functions, in the period range of 0.30 to 2.0 seconds, as the observed low-period spikes from the records are of no practical significance.

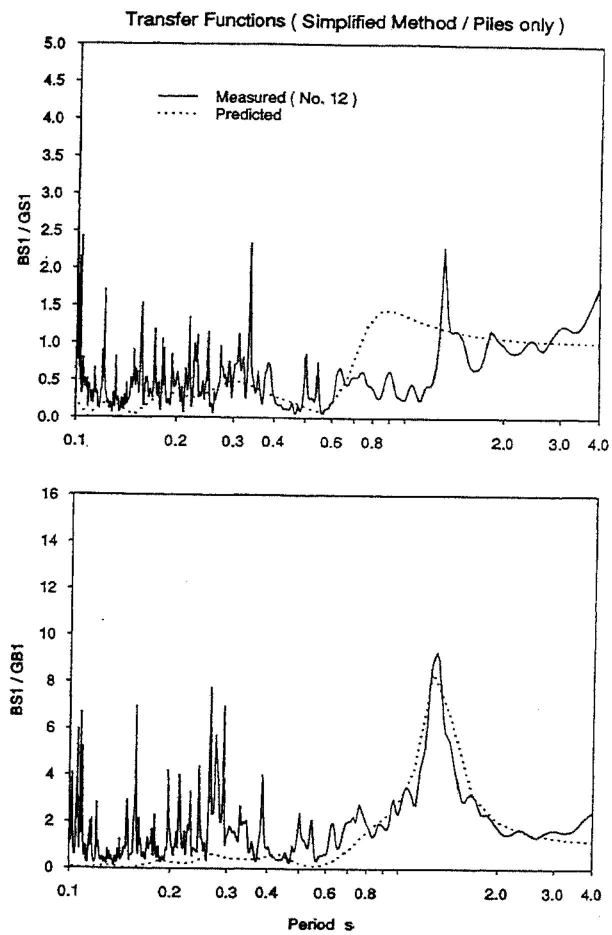
The agreement between theory(ies) and measurements is very good in only a few cases, namely when the Fourier amplitude spectrum (FAS) of a response quantity is divided by the FAS of the dilluvial-base motion, GB1. Thus, for instance, the theoretical BS1/GB1 curve (transfer function between horizontal footing motion and soil base motion) predicts very successfully the peak in the ratio of the FAS of the two recorded motion. Even the ratio BR1/GB1 is reasonably well predicted in most analyses.

By contrast, when the FAS of the ground surface motion, GS1, is in the denominator of a response FAS, the theoretical curves substantially overpredict the recorded ratio. Prime example: the ratio BS1/GS1, which is frequently used as a measure of soil-structure interaction. In the period range 0.7 to 1.2 seconds the theory predicts a very flat peak centered at about 0.9s and reaching about 1.5 --- contrary to the recorded ratio which exhibits small oscillations about a very low value, approximately 0.30 (i.e., about five times smaller on the average for this whole period range). These differences are also echoed (and in fact amplified) in the BR1/GS1 transfer function.

Table 2 EARTHQUAKE RECORDS AT OHBA-OHASHI BRIDGE

Earthquake Number	M J.M.A	R km	D km	PHGSA % g H1	PHGSA % g H2	PHGBA % g H1	PHGBA % g H2	PVGSA % g	PVGBA % g
4	6.7	263	0.0	1.02	0.96	0.24	0.25	0.5	0.15
7	7.0	238	10	1.70	1.85	0.44	0.41	0.73	0.28
11	6.0	81	70	2.93	3.13	0.62	0.72	1.67	0.49
12	6.0	42	20	11.36	9.16	3.31	3.85	2.91	1.37
13	5.4	38	20	1.90	2.55	0.48	0.89	1.29	0.34

Note: M = magnitude; R = epicentral distance; D = focal depth;
PHGSA = peak horizontal ground surface acceleration; PHGBA = peak horizontal ground base acceleration
PVGSA = peak vertical ground surface acceleration; PVGBA = peak vertical ground base acceleration



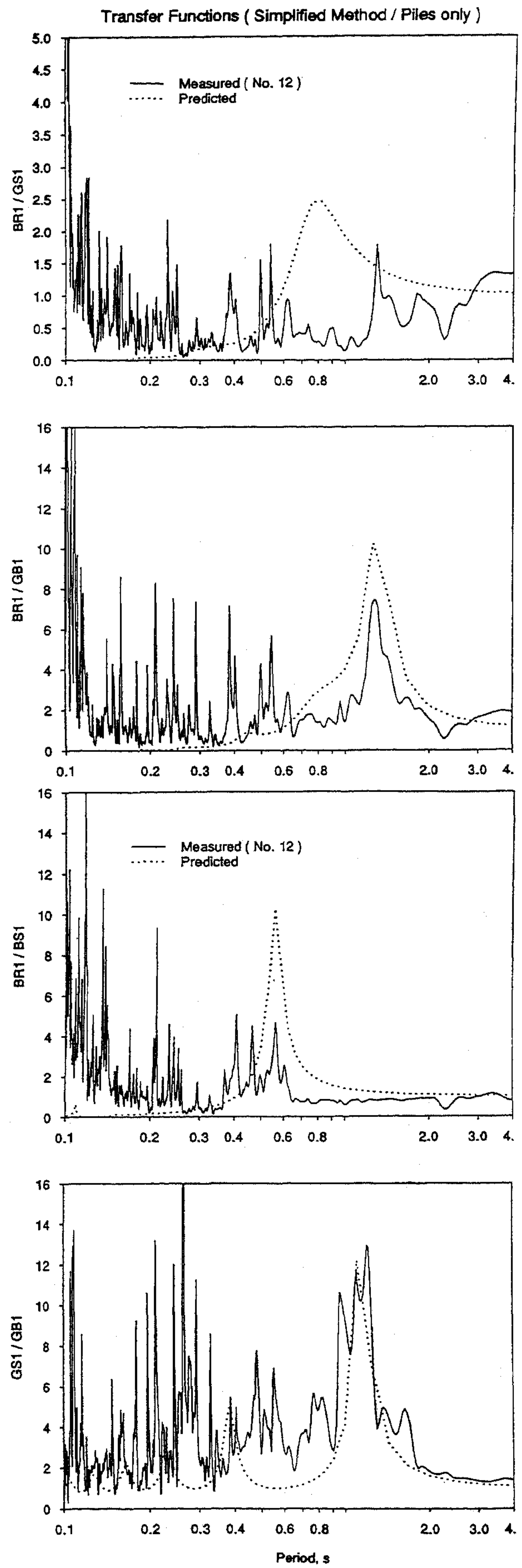


FIG. 6 Comparison of six recorded against six computed transfer function for the complete soil-pile-footing-bridge problem. (Simplified interaction-factor solution ignoring the contribution of the footing.)

The above success and failures of the theoretical predictions are more or less true with all types of analysis. However, using pile static interaction factors (method 6) and ignoring the interaction between piles (method 5) lead to the largest discrepancies between theory and records.

Key to understanding the overprediction of the BS1/GS1 when using the theory is the aforesaid "3-D alluvial base effect". Our soil-structure interaction theories assume vertically propagating S waves, which can not produce on the ground surface (at GS1) the substantial observed peaks between 0.70 and 1.20 seconds. Thus, the denominator (GS1) is underpredicted with the theories. On the other hand, inclined and vertical waves are "filtered" similarly through the pile-footing-structure system, so that the numerator (BS1) is probably predicted quite well. Hence the overprediction of the ratio BS1/GS1. And when we divide the BS1 spectrum directly with the FAS of the base input excitation (GB1), the resulting prediction becomes very satisfactory.

BENDING STRAINS IN THE PILES

From the strain meters attached inside the two pipe piles, axial and bending strain histories were recovered, for each of the five events listed in Table Thus, a unique set of field data is available for studying the internal forces developing in piles during earthquakes --- a subject on which the current knowledge is rather elementary.

Fig. 7 shows the recorded bending-strain time histories at four depths along the vertical pile during Earthquake 12. Their respective Fourier amplitude spectra (FAS) are plotted in Fig. 8. Also shown in Fig. 8 for comparison are the bending-strain FAS predicted with a simplified Beam-on-Dynamic-Winkler-Foundation (BDWF) method. Details on this method can be found in Gazetas et al (1992), Makris & Gazetas (1992), and Kavvadas & Gazetas (1993). Only the "kinematic" loading, however, is considered in this study.

Note that the largest peak values in both the recorded and computed FAS occur at the fundamental natural period (T = 1.4 s) of the soil deposit. Also note that the amplitude of the developing strains is largest at the top and bottom locations.

The districution with the depth of these largest peak spectral values is plotted in Fig. 9, where again the recorded values (in both the vertical and the raker pile) are contrasted with the values computed with the BDWF model, but considering the kinematic response only (i.e. ignoring the inertia of the superstructure).

The following practically-significant conclusions emerge from Figs. 7 - 9:

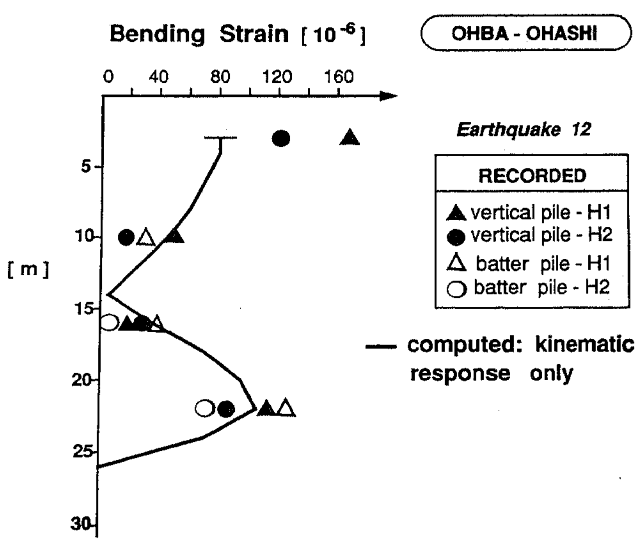


FIG. 9 Comparison of the recorded and computed maximum spectral bending strains along the vertical and racker pile.

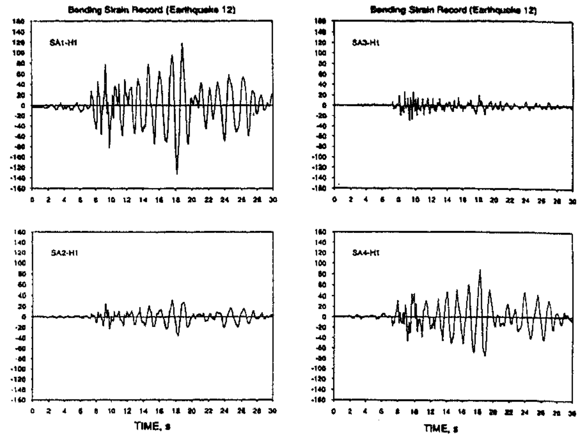


FIG. 7 Recorded bending strain time histories at four depths on the vertical pile during Earthquake 12.

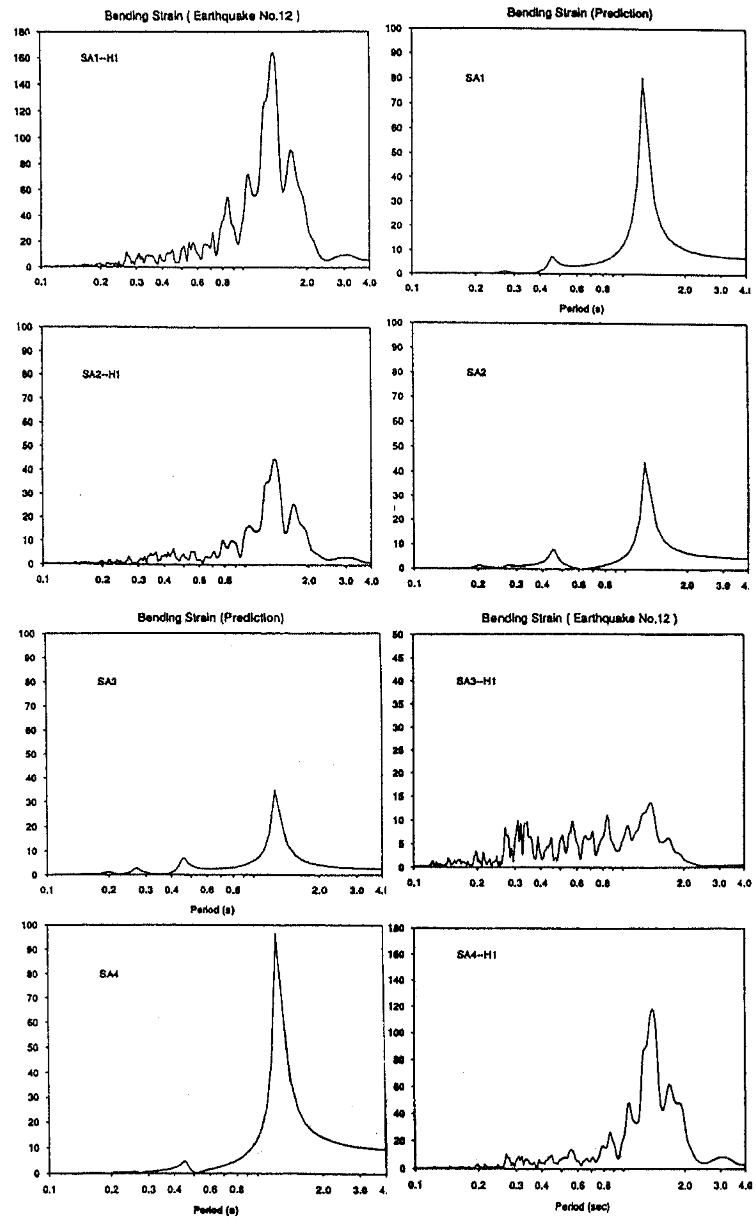


FIG. 8 Comparison of the recorded and theoretically computed bending strain transfer functions at each of the four depths shown in Fig. 3 (Earthquake 12).

(a) The kinematically-induced bending strains can be quite large at great depths wherever interfaces of soil layers with sharply differing stiffnesses exist, as at z = 22 m in Ohba-Ohashi. Such bending deformations are not affected by the inertial load transmitted from the superstructure onto the head of the piles; hence, our solely-"kinematic" analysis can predict them very well.

(b) By contrast, the inertia-induced bending strains are significant only near the top of the piles. They arise from both the horizontal inertia force and the restraining / / overturning moment atop the pile. Our BDWF kinematic analysis for fixed-head piles ignores the horizontal force and overturning moment, but does restrain the pile against rotation. As a result, it underpredicts by a factor of about 2, the pile-head strains. (It is theoretically simple to incorporate the superstructure inertia in the BDWF approximate analysis.)

(c) The conclusions reached in several theoretical studies of pile distress during earthquakes are largely confirmed by the measurements.

ACKNOWLEDGEMENTS

This project was funded by Shimizu Corporation, Japan, and by the National Center for Earthquake Research, SUNY at Buffalo. We are deeply thankful for this support.

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