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The Collapse of Fukae (Hanshin Expressway) Bridge, Kobe, 1995: The Role of Soil and Soil-Structure Interaction

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THE COLLAPSE OF FUKAE (HANSHIN EXPRESSWAY) BRIDGE, KOBE, 1995: THE ROLE OF SOIL AND SOIL-STRUCTURE INTERACTION

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ABSTRACT

The paper investigates the role of soil in the collapse of a 630m segment (Fukae section) of the elevated Hanshin Expressway during the severe Kobe earthquake of 1995. From a geotechnical viewpoint, the earthquake has been associated with extensive liquefactions (notably of reclaimed ground), lateral soil spreading, and damage to waterfront structures. However, there is evidence that soil-foundation-structure interaction (SFSI) in non-liquefied ground played a detrimental role in the seismic performance of local structures, including the one under investigation. The bridge consisted of single circular concrete columns monolithically connected to a concrete deck, founded on pile groups in alluvium sand and gravel. There were 18 spans in total, all of which suffered a spectacular pier failure and transverse overturning. Several factors associated with poor structural design have already been identified. The scope of this paper is to complement the earlier studies by examining the role of soil in the collapse. Specifically, the following issues are discussed: (1) seismological and geotechnical information pertaining to the recorded ground- motions; (2) soil amplification; (3) response of soil-foundation-superstructure system; (4) response of nearby structures that did not collapse. Results indicate that the role of soil in the collapse was *triple*: *First*, it modified the bedrock motion so that the frequency content of the resulting surface ground motion became disadvantageous for the particular structure. *Second*, the compliance of soil and foundation altered the vibrational characteristics of the bridge and moved it to a region of stronger response. *Third*, ductility demand on the pier was higher than the ductility demand of the system. The increase in seismic demand on the piers may have exceeded 100% in comparison with piers fixed at their base. The results of the study contradict the widespread view of an always-beneficial role of soil-foundation-structure interaction on seismic response.

Fig. 1. Partial view of the collapsed bridge

INTRODUCTION

The $M_s = 6.8$ ($M_w = 7$) earthquake struck the city of Kobe at 5.46 a.m. local time on Tuesday January 17, 1995, exactly one year after the severe earthquake in Northridge, California. It is the first major earthquake to hit a modern city with a high concentration of population and urban facilities. The event occurred right under the city and resulted in the worst earthquake-related disaster in Japan since the Kanto earthquake of 1923. In Kobe 5,500 lives were lost, 35,000 people were injured and more than 150,000 buildings collapsed or suffered damage beyond repair. The port of the city, which was of considerable importance to the Japanese economy, was destroyed almost completely. The overall economic loss has been estimated at U.S. $100 billion (Kimura, 1996). Detailed reports on the earthquake have been published by Akai et al (1995), EERI (1995), NIST (1996), JGS (1996; 1998), Werner et al (1995).

The earthquake came as surprise to seismologists and earthquake engineers, not only because it hit a relatively "aseismic" region without a major event in over 300 years, but primarily because of the extremely severe recorded ground motions --- much stronger than in any previous Japanese earthquake.

In the devastation caused by the earthquake, the collapse and transverse overturning of the 630m section of Hanshin Expressway at Fukae was perhaps the most spectacular failure (Fig. 1). The bridge was part of the elevated Route 3 that runs parallel to the shoreline. Built in 1969, it consisted of single circular columns, 3.1m in diameter and about 12 ± 1 m in height, monolithically-connected to a concrete deck, founded on groups of 17 piles. The main geometric characteristics of the structure are shown in Figs. 2 and 3.

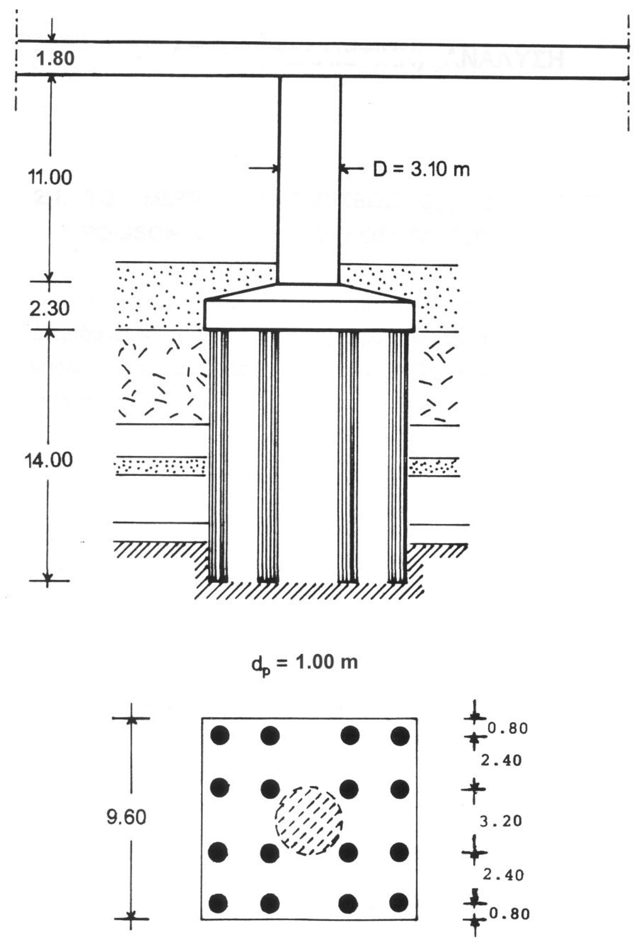


Fig. 2. Geometric characteristics of typical collapsed pier

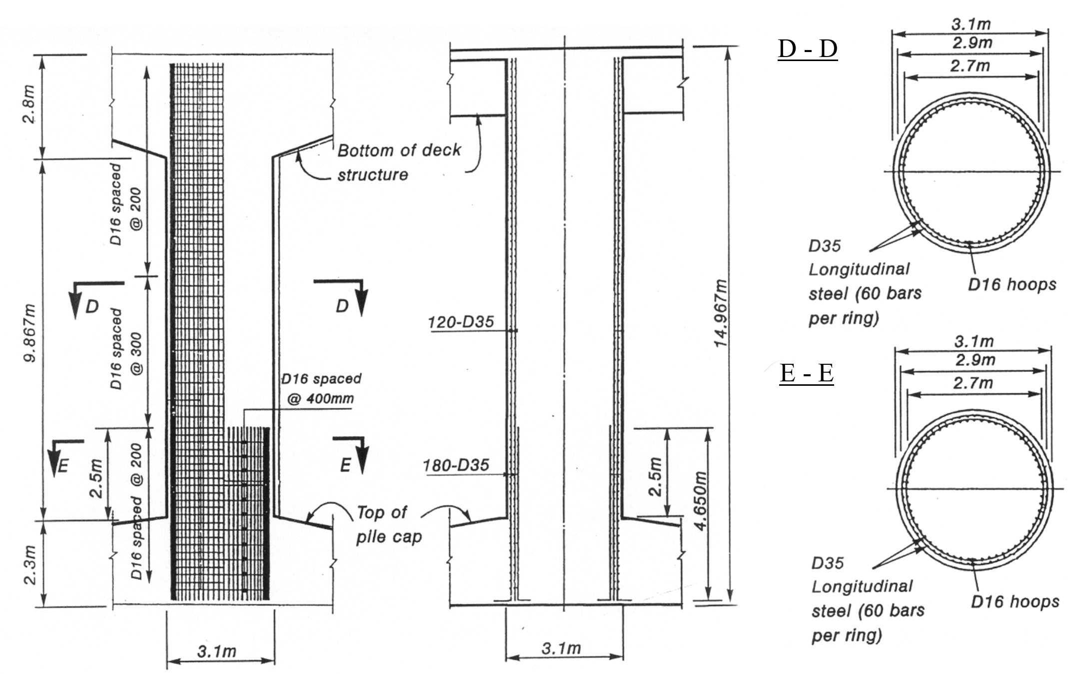
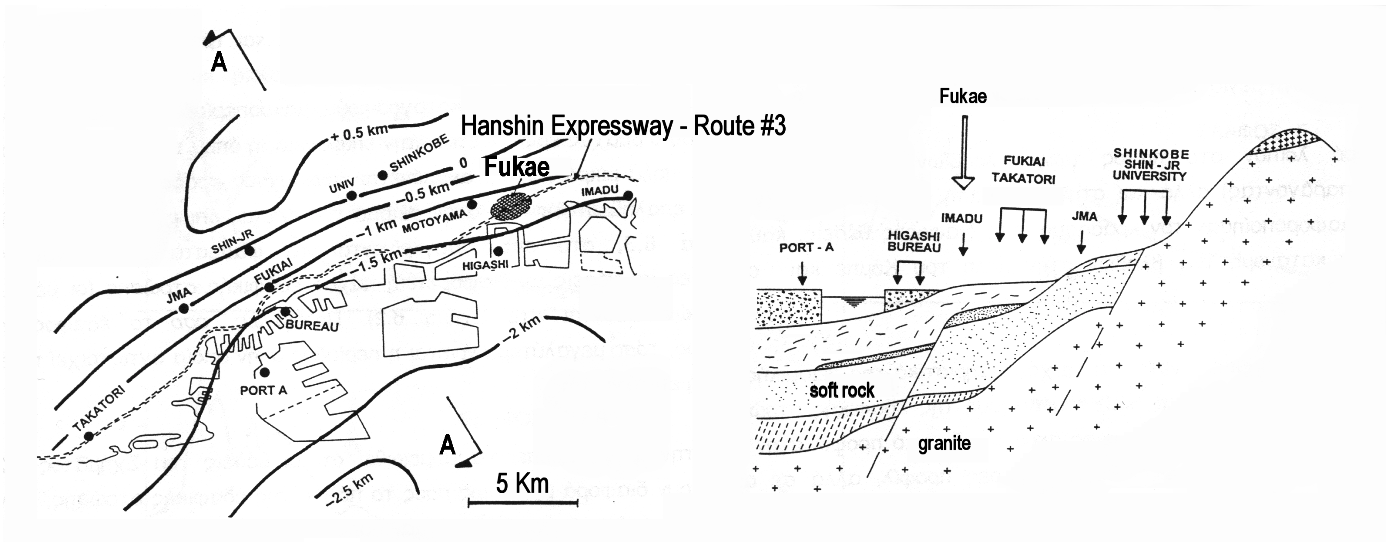


Fig. 3. Longitudinal and transverse reinforcement of typical collapsed pier



(a) (b)

Fig. 4. (a). Contours of bedrock elevation and location of accelerometers; (b) approximate geologic section A-A.

Detailed structural investigations of the performance of Fukae section have been conducted (e.g., Seible et al 1995, Park 1996, Kawashima & Unjoh 1997, Anastasopoulos 1999, Sun et al. 2000; Abe et al 2000) to explore the causes of the collapse. In these studies, factors associated with poor structural design have been identified including:

- Inadequate transverse reinforcement in the piers;
- Inadequate anchorage of longitudinal reinforcement;
- Use of un-conservative (elastic) methods for determining design shear forces.

Notwithstanding the importance of these findings, there is evidence presented in this paper that local soil conditions and dynamic interaction between foundation and superstructure further aggravated its inelastic behavior, thereby contributing to the collapse.

Additional concerns come from the fact that Soil-Foundation-Structure Interaction (SFSI) has been traditionally considered as *beneficial* for seismic response. Apparently, this perception stems from oversimplifications in the nature of seismic demand adopted in seismic code provisions. The most important of these simplifications, with reference to SFSI, are (Mylonakis & Gazetas 2000): (1) design acceleration spectra that either remain constant, or decrease monotonically with increasing structural period; (2) response modification coefficients (i.e., "behavior" factors used to derive seismic forces) which are either period independent or increase with increasing structural period; (3) foundation damping derived assuming homogeneous half-space conditions, which tends to over-predict overall damping; (4) kinematic foundation response analyses indicating that the "effective" excitation imposed at the base of a structure is smaller than the free-field soil motion; (5) use of "system" ductility factors, which may not reflect the actual seismic demand in the piers.

This apparently beneficial role of SSI has been essentially turned into a dogma. Thus, practicing engineers frequently avoid the complication of accounting for SFSI, as a conservative simplification that would supposedly lead to improved safety margins. Results presented in this paper are in contradiction with this perception. It is worth mentioning that detrimental effects of SFSI in seismic response have been pointed out in the past (e.g., Bielak 1978, Resendiz & Roesset 1995, Meymand 1998, Celebi 1998, Takewaki 1998, Mylonakis & Gazetas 2000). However, these studies have apparently received little attention by code writers and engineers.

The work reported in this paper involves:

- Discussion of seismological and geotechnical information pertaining to the bridge site;
- Analysis of free-field soil response;
- Analysis of response of the foundation-superstructure system;
- Evaluation of results through comparisons with earlier studies that did not consider SSI.

THE FIRST ROLE OF SOIL: INFLUENCE ON GROUND MOTIONS

Geology and Ground Motions

Kobe and the nearby suburbs of Asiya, Nisinomiya, and Amagasaki are located in Honshu island, about 450 km southwest of Tokyo. They are built along the shoreline in the form of an elongated rectangle with length of about 30 km and width 2 to 3 km. The biotite granitic bedrock (known as the Rokko granite), outcrops in the mountains and dips steeply under

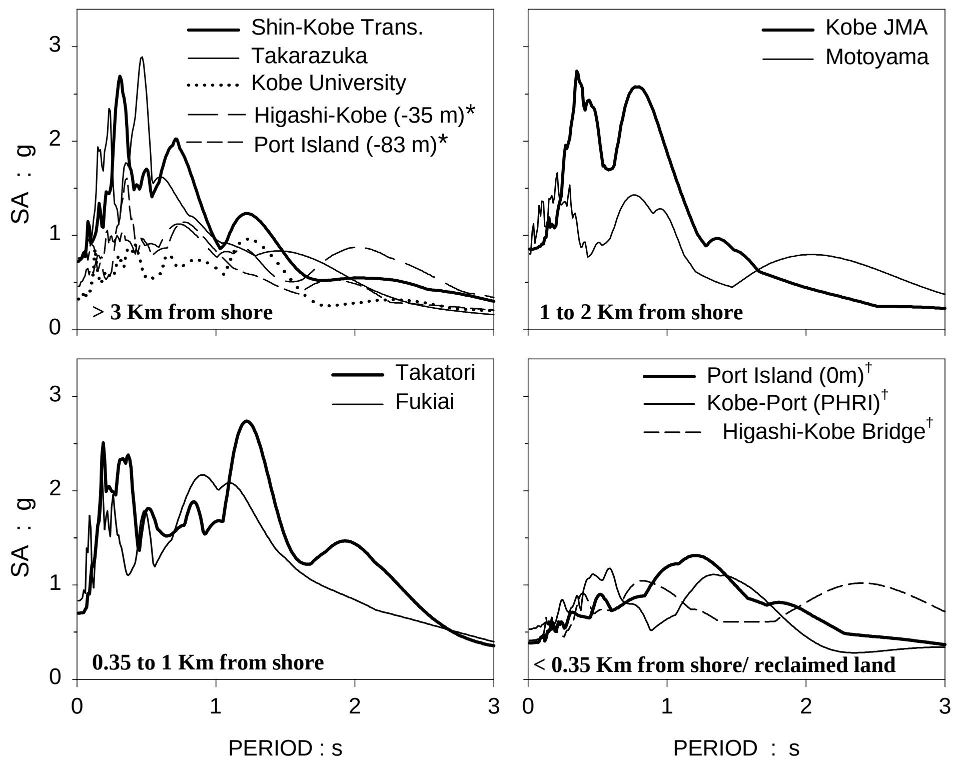


Fig. 5. Acceleration spectra grouped with respect to distance from the shoreline. Note the differences in predominant periods. Plotted are spectral of the fault normal components of each motion. *[* denotes motions at depth; † denotes liquefied sites; ζ = 5%]*

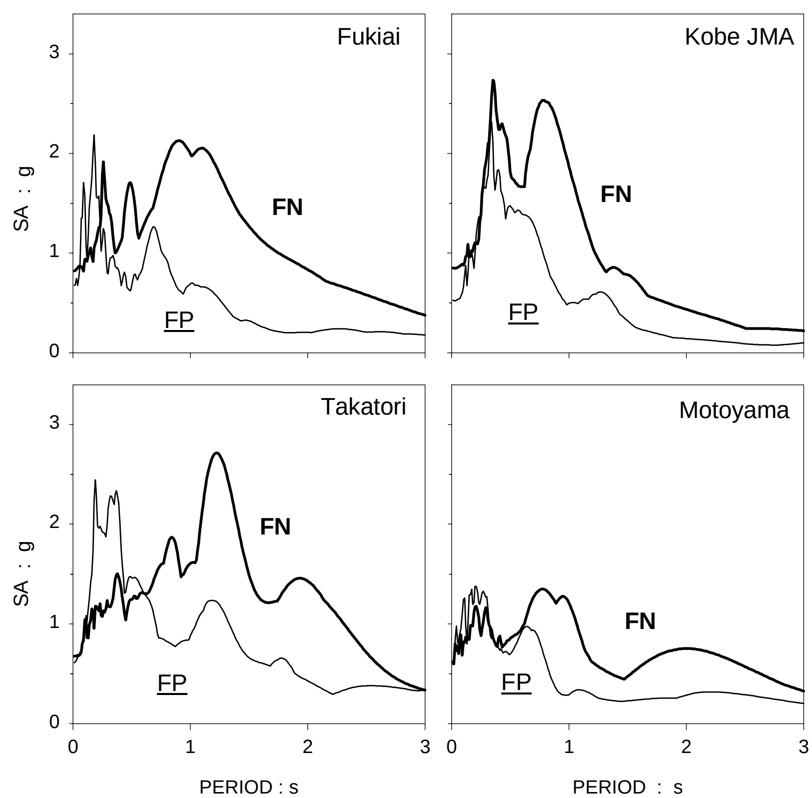


Fig. 6. Differences between the response spectra of the fault-normal (FN) and fault parallel (FP) components of four main records (see Fig. 9).

the soft formations reaching a depth of about 1 to 1.5 km at the shoreline (Kawase 1996, Tokimatsu et al 1996). Depths as high as 2.5 km under Rokko island have been reported (Iwasaki 1995). The soil in the region consists primarily of Holocene alluvial deposits (sand, gravel, and layers of clay) of variable thickness (10-80m), underlain by stiffer Pleistocene deposits. The thickness of the Holocene alluvium increases to the southeast, from about 10m in uptown Kobe (e.g., Motoyama), to 20m in Downtown Kobe (e.g., Fukae, Takatori), to more than 40m in Port Island. Figures 4a and 4b show an approximate geologic plan and a cross section of the region, including the locations of selected strong motion accelerometers.

The mainshock was recorded in over 200 strong motion instruments. Several records were of unusually high intensity measuring Peak Ground Accelerations (PGA) and Velocities (PGV) in excess of 0.8g and 100cm/s, respectively. PGA's above 0.4g were recorded at 17 sites. At least in three locations, PGA exceeded the astounding 0.80g.

Variability in local soil conditions among the recording stations might be partly responsible for the significant differences in the intensity and frequency content of the recorded motions, as clearly shown in Fig. 5. Three additional effects however, also have affected the surface motions in Kobe: *forward rupture directivity*, *2D basin effects*, and *soil liquefaction*.

The first is of a seismological nature, affecting ground shaking at near-fault sites located in the direction of fault rupture propagation. The effect of forward fault-rupture directivity on the response spectrum is primarily to increase the spectral values of the horizontal component *normal* to the fault strike, at periods longer than about 0.5 sec. The resulting differences between Fault–Normal (FN) and Fault–Parallel (FP) response spectra, plotted in Fig. 6 are indeed striking.

Additional evidence on directivity effects is given in Fig. 7: *Polar plots* of horizontal spectral accelerations are plotted for the JMA, Fukiai, and Takatori motions, for three selected periods. The fault normal and fault parallel directions are also indicated in the graphs. It is observed that while PGA is essentially independent of orientation, long-period acceleration components (T > 0.6s) attain their maxima in the fault-normal direction and their minima in the fault-parallel direction. The opposite seems to be true with the short-period component (with the exception of JMA record). Similar patterns (not shown) are observed with other records obtained in the vicinity of the fault. It is important to mention here that this attribute seems to be independent of local soil conditions (Ejiri et al 1996).

The 2D basin (valley) effect has been shown to increase or decrease the intensity, duration, and frequency characteristics of ground motion depending on the proximity to the edge of the valley, the dipping angle, the frequency content of the excitation, and the incidence wave angles (Bielak et al 1999). Finally, soil liquefaction results in significant reduction of high-frequency acceleration peaks, increase of dominant period of vibration, and

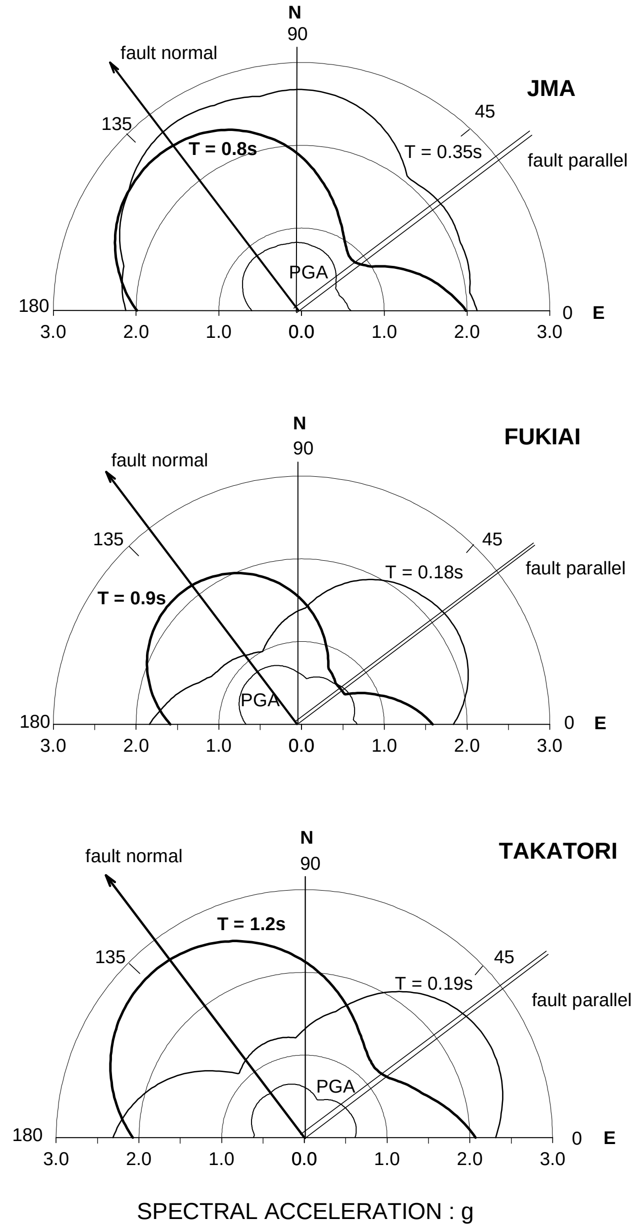


Fig. 7. Polar plots of spectral accelerations of three selected records, at three different periods. Note the pronounced values along the fault normal direction at long periods; $\zeta = 5\%$

in large permanent deformations if static (permanent) shear stresses exist in the ground.

All these effects, have contributed (more or less) to the differences in ground motions seen in Fig. 4. Evidently, the closer the site to the shore, the deeper and softer the soil deposit, thereby leading to a longer predominant period and a flatter spectrum. Interestingly, the site groups in Fig. 5 differ not only with respect to distance from shore, and flexibility of soil, but also with respect to distance from fault. It is important to mention here that the site categories in Fig. 5 would have been different if the site classification scheme of NEHRP-2001 had been adopted.

For example the Takarazuka site (a shallow site located approximately 6 km inland), would classify in the same group as the (much deeper and softer) Fukiai and Takatori sites, due to the presence of soft soil close to the surface. Similar "misclassifications" would occur with the Shinkobe Trans and Fukiai records, which would classify in the same group as the much shallower Motoyama and JMA sites. Evidently, site thickness has an important influence of surface ground motions, which is not adequately recognized in existing codes.

Ground Shaking at the Site

Unfortunately, no records were obtained at the site during the main shock. The closest stations to the bridge were Motoyama and Higashi Kobe, located more than 1km from the site. The first instrument is a velocity meter stationed at Motoyama elementary school, about 1500m to the northwest of the collapsed segment, just south of the Rokko hills. The second instrument is part of a vertical array installed at the east abutment of Higashi Kobe Bridge, which is located on the shoreline, about 1.3km south of the collapsed bridge.

The uncertainty in the characteristics of the ground motion and the soil profile at the location of the bridge dictated the use of plausible scenarios. From the site boreholes, the soil profile is judged as a relatively deep, moderately stiff to soft deposit of sand and gravel with low-strain shear-wave velocity for the upper 20 meters of the order of 200 to 300 m/s (Fig. 8). Six acceleration records with different peak ground accelerations and frequency characteristics are examined:

- The accelerogram FUKIAI, with PGA of about 0.83 g and PGV of 115 cm/s in the fault normal direction, recorded on a medium-soft and relatively deep deposit (60 m of soil with average V_s less than 400 m/s)
- The accelerogram TAKATORI, with PGA of 0.68 g and PGV of 169cm/s in the fault-normal direction, recorded on a soft and deep deposit (80 m of soil with V_s less than 400 m/s)
- The accelerogram JMA, with PGA of 0.83 g and PGV of 96 cm/s in the fault-normal direction, recorded on a stiffer soil formation (10-15 m of stiff soil)
- The accelerogram MOTOYAMA, with PGA of 0.62 g and PGV of 75 cm/s recorded on a shallow soil site (soil thickness of about 20m), about 1 km to the northwest of the bridge

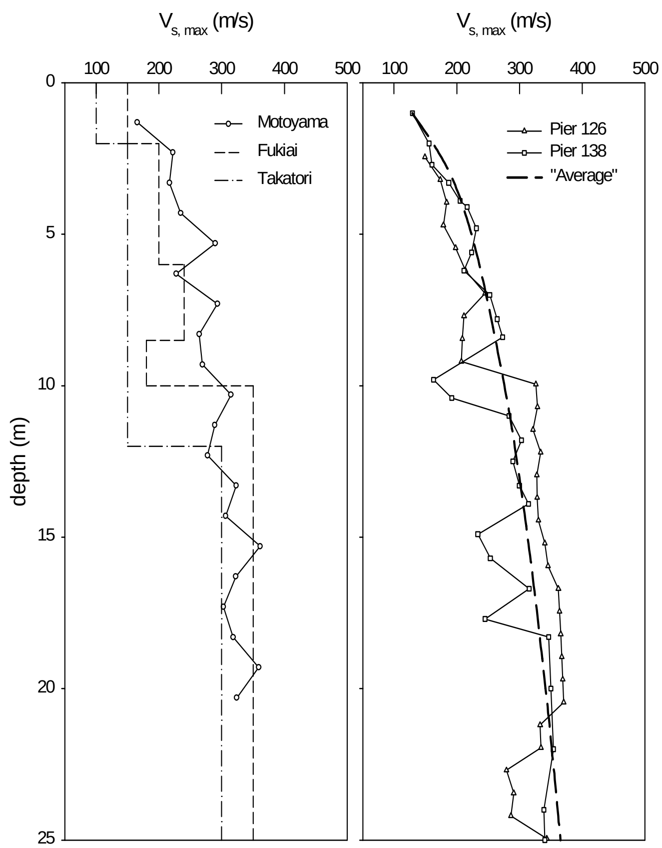


Fig. 8. Low-strain shear wave velocities for soil profiles considered in the analyses

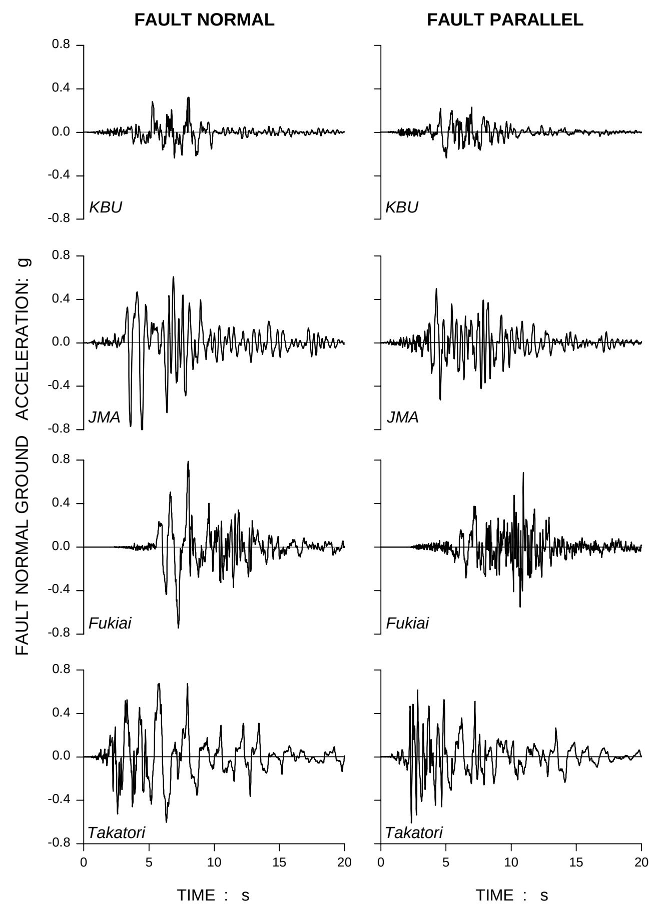


Fig. 9. Selected accelerograms from the earthquake. Comparison of fault-normal and fault-parallel motion components.

- The accelerogram HIGASHI Kobe, with PGA of 0.44 g and PGV of 81 cm/s recorded in a stiff layer, at a depth of 35 meters, below a liquefied layer, about 1 km south of the bridge.

- A SYNTHETIC motion was used, which has been derived by Matsushima & Kawase (1999) based on a multiple asperity model and a 3-D basin structure. The time history was obtained at the location of the collapsed bridge considering a “reference” rock stratum with Vs = 400m/s. The peak ground motions are 0.4g (PGA) and 55cm/s (PGV).

soil to be representative of the Fukae motion. The last three records (MOTOYAMA, HIGASHI Kobe, SYNTHETIC) were selected because of proximity to the structure. Owing to the very different ground conditions between these recording sites and the Fukae bridge site, the three records were suitably amplified using 1D wave-propagation theory based on the wave velocities of Fig. 8 to obtain pertinent surface motions. Thus six motions were obtained and used as excitation.

Selected time histories and corresponding response spectra are shown in Fig. 9 and 10, respectively for rock (Fig 10) and free-field surface conditions (Fig. 11)**,** in the fault-normal direction.

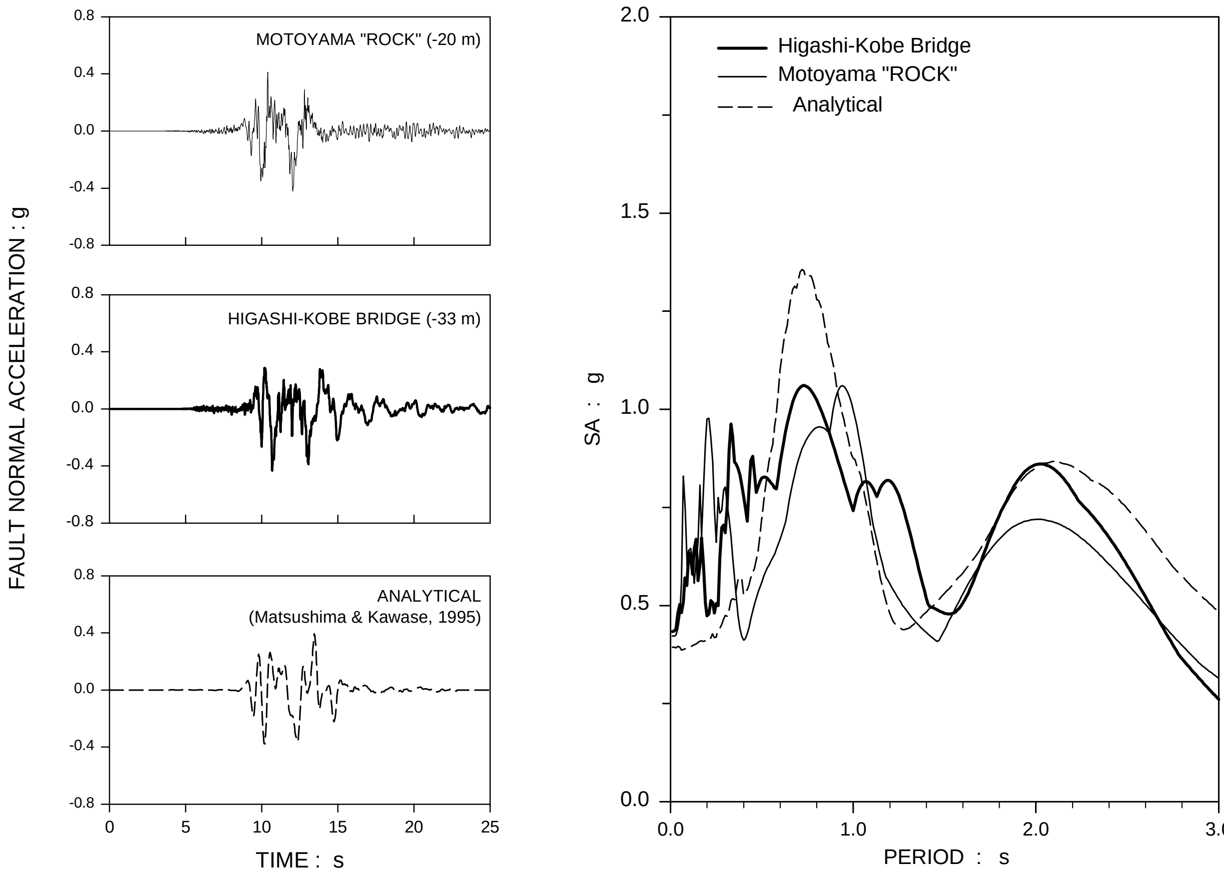


Fig. 10. Bedrock motions close to the bridge site, and corresponding 5%-damped acceleration spectra (fault-normal components).

Of the above records, the first two, although recorded far from the bridge, are believed to be the most representative of the motion in Fukae Route 3: (a) because of their similar distance from fault and shoreline (with Fukae Route 3), and (b) the similar orientation with respect to rupture as the collapsed segment. Indeed, judging from the geology of Fig. 4a, soil conditions at the location of the bridge seem to be closer to those of FUKIAI and TAKATORI than to any other station. The third accelerograms the famous JMA record, was selected because it has been invariably used (often as the only record) by previous investigators. It is much closer to the fault and on much stiffer

Note the similarities among the rock motions (particularly between Motoyama and Higashi Kobe bridge).

THE SECOND ROLE OF SOIL: FOUNDATION-SUPERSTRUCTURE INTERACTION

Elastic and Simplified Inelastic Analyses

The foundation consists of 17 reinforced concrete piles having length of about 15 m and diameter of 1m, connected through a

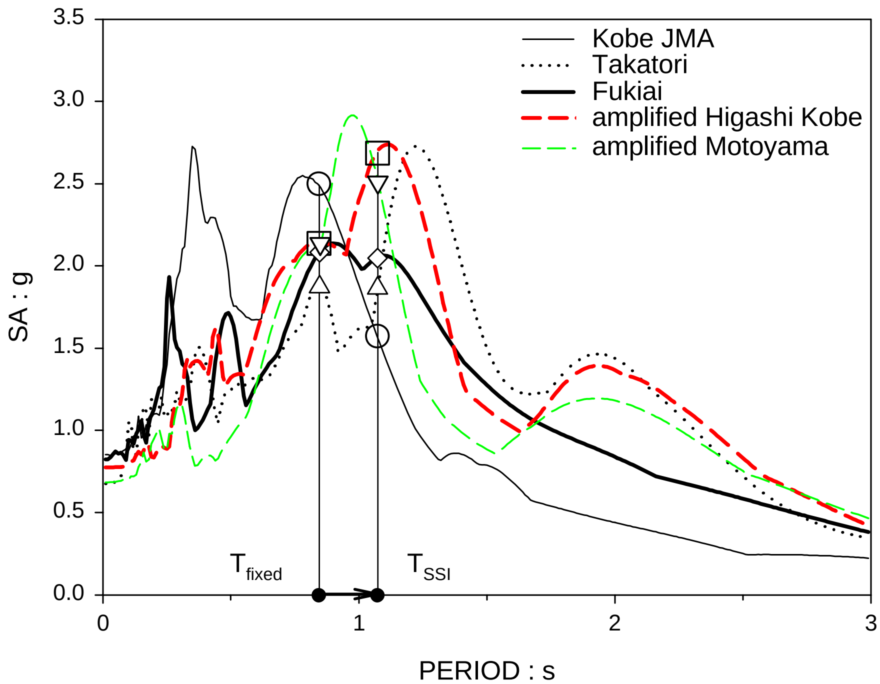


Fig. 11. Acceleration response spectra of selected ground motions in the fault normal direction; ζ = 5% (modified after Gazetas 1996)

rigid cap of planer dimensions 9.6 m x 10.6 m (Fig. 2a).
Structural parameters for the foundation-superstructure system used in previous studies are summarized in Table 1. Despite the differences in inertia and (especially) stiffness of the bridge among the various studies, the variation in fixed-base natural period is rather small, with T-fixed ranging between 0.55 to 0.75 seconds. Considering SFSI, natural period is longer varying between 0.75 and 0.93 seconds. Differences in pier strength are considerable with the normalized yielding strength C_y (= F_y / M_{deck} g) ranging between 0.5 and 0.7, depending primarily on the value of lateral yielding force F_y. These values are quite high given the year of the design (1964). Estimated displacement ductility capacity of the pier ranges between 1.6 to 3.2, depending on the assumptions. Additional parameters in Table 1 will be discussed later on.

Detailed calculations performed by the Authors suggest a participating mass of the deck of about 1000 Mg, a rotational moment of inertia approximately 32,300 Mg m^2, and a pier mass of about 226 Mg (Table 2). Following Seible et al, the cross-sectional moment of inertia of the cracked pier was taken at about 40% of its gross value. Using this information, the fixed-base natural period of the bridge modeled as a simple oscillator can be estimated from the energy expression (Syngros et al 2003)

$$T_{fixed} = 2\pi \sqrt{\frac{M_{deck} + \left(\frac{33}{140}\right) M_{pier} + \left(\frac{3}{2H}\right)^2 I_{deck}}{K_{pier}}} \qquad (1)$$

in which $K_{pier} = 3 E I / H^3$ denotes the lateral stiffness of the pier.
Eq.(1) yields :

$$T_{fixed} \cong 0.84\, s \qquad (2)$$

in agreement with Table 2. The difference from the periods estimated by Seible et al and Park in Table 1 is attributed to the inclusion of the rotational inertia of the deck.
The compliance of the foundation further increases both the natural period, T_{SSI}, and the damping, ζ_{SSI}, of the system. Modeling the bridge as a generalized single-dof oscillator, good estimates of natural period and damping can be obtained from the following energy-based expressions (Syngros et al 2003):

$$T_{SSI} = 2\pi \sqrt{\frac{M_{deck} + \phi_M M_{pier} + \phi_{21}{}^2 M_{cap} + \phi_{31}{}^2 I_{deck} + \phi_{41}{}^2 I_{cap}}{\phi_K K_{pier} + \phi_{21}{}^2 K_{hh} + \phi_{41}{}^2 K_{rr} + 2\phi_{21}\phi_{41} K_{hr}}} \qquad (3)$$

$$\zeta_{SSI} = \phi_K\, \zeta_{pier} + \phi_{21}{}^2\, \zeta_{hh} + \phi_{41}{}^2\, \zeta_{rr} + 2\phi_{21}\phi_{41}\, \zeta_{hr} \qquad (4)$$

in which ϕ_{21}, ϕ_{31}, ϕ_{41}, ϕ_K, and ϕ_M are dimensionless factors given by

$$\phi_{21} = K_{pier}\left(K_{rr} + K_{hr} H\right)\lambda^{-1} \qquad (5a)$$

$$\phi_{41} = K_{pier}\left(K_{hr} + K_{hh} H\right)\lambda^{-1} \qquad (5b)$$

$$\phi_{31} = -\left[\left(1 - \phi_{21}\right)\frac{3}{2H} + \frac{1}{2}\phi_{41}\right] \qquad (5c)$$

$$\phi_M = \frac{1}{H}\int_0^H \psi^2 dx \;\; ; \;\; \phi_K = \frac{H^3}{3}\int_0^H \left(\frac{d^2\psi}{dx^2}\right)^2 dx \; ; \tag{5d,e}$$

$$\psi = \left(1-\phi_{21}+H\phi_{41}\right)\left[\frac{3}{2}\left(\frac{x}{H}\right)^2+\frac{1}{2}\left(\frac{x}{H}\right)^3\right]+\phi_{21}-x\,\phi_{41} \tag{5f}$$

$$\lambda = \left(-K_{hr}{}^2+K_{rr}K_{hh}\right)+K_{pier}\left[K_{rr}+H\left(2K_{hr}+K_{hh}H\right)\right] \tag{5g}$$

Table 1. Structural parameters used in previous studies

Model	Seible et al. 1995	Park 1996	Kawashima & Unjoh 1996	Michaelides & Gazetas 1998
	Single pier on rigid foundation	Single pier on rigid foundation	Multiple piers on flexible foundation	Single pier on flexible foundation
L (m)	12.3	12	-	11
E (Gpa)	-	30.1	27.8	20
I / I_{gross} †	0.4	0.45	0.59	0.75
K_{pier} (MN/m)	80	107	128	155
M_{deck}* (Mg)	1100	1121	-	1200
I_{deck} (Mg m^2)	0	0	-	40000
M_{cap} (Mg)	0	0	-	0
I_{cap} (Mg m^2)	0	0	-	0
T_{fixed} (s)	0.75	0.64	0.55**	0.68
T_{SSI} (s)	-	-	0.75	0.93
F_y (kN)	5407	6640	4673	8240 (bottom)
C_y	0.5	0.6	0.43**	0.7
force-displacement relation	-	elastic-perfectly plastic	Takeda	elastic-perfectly plastic
ζ_{pier} (%)	-	5	-	5
ζ_{SSI} (%)	-	-	-	7.5
$\mu_{capacity}$	2.4	2.2	3.2	1.6
excitation	-	JMA	JMA	JMA, Fukiai
μ_{demand}	-	> 2.2	> 3.2	1.3 to 1.7

− = not reported

* *includes portion of pier mass*

** *estimated by the Authors considering M_{deck} = 1100 Mg*

Equations (3) and (4) differ from similar formulations developed for surface footings (Gazetas 1991, 1996), due to the presence of cross terms K_{hr} and ζ_{hr} in the foundation impedance matrix, and the rotational inertia of deck and cap. Both features are important given the large rotational inertia of the mushroom-type superstructure and the presence of piles in the foundation. Note that with increasing K_{hh} and K_{rr}, Eq.(3) duly reduces to Eq.(1).

Using pertinent analytical tools from the literature (Poulos and Davis 1980, Gazetas 1991, Mylonakis et al. 1997), estimates of foundation stiffness have been obtained as shown in Table 2. These values refer to soil strains in the free-field of 5 x 10^{-3} or higher. Corresponding values at low soil strains obtained by Michaelides & Gazetas (1998) are also given. The differences between the predictions, particularly in the swaying mode, are as expected.

Based on the parameters listed in Table 2, the natural period and damping of the system is estimated from Eqs.(3) and (4) as

$$T_{SSI} \approx 1.05s \quad ; \qquad \zeta_{SSI} \approx 0.10 \tag{6}$$

which are indicative of the role of SSI : increase of natural period by an appreciable 20%, and of damping ratio by 100 %. Note that the above damping ratio does not account for inelastic damping in the pier. Given the large imposed deformations, incorporating such a mechanism will increase the overall damping by at least 50%.

Table 2. Structural parameters used in the analyses

L (m)			12		
E (GPa)			27.8		
I / I_{gross}	0.4	0.5	1	0.5	0.5
K_{col} (MN/m)	88	109	219	109	109
M_{deck} (Mg)		1000		1000	1000
I_{deck} (Mg m^2)		32300		0	0
M_{cap} (Mg)		750		750	0
I_{cap} (Mg m^2)		9000		0	0
M_{pier} (Mg)		53		53	53
K_{xx} (MN/m)			310		
K_{rx} (MN)			1090		
K_{rr} (MN m)			48300		
T_{fixed} (s)	0.84	0.75	0.53	0.62	0.62
T_{SSI} (s)	1.04	0.98	0.84	0.89	0.87
ζ_{SSI} (%)	9.7	10.3	12.2	10.3	10.3
$(\psi)_{fixed}$	0.72	0.72	0.73	1.07	1.07
$(\psi)_{SSI}$	0.94	0.97	1.04	1.16	1.04

From the elastic spectra of Figure 11, the influence of SSI on the response starts becoming apparent. For instance, if the actual excitation was similar to the JMA record, the increase in period due to SSI and the progressive cracking of the pier would tend to slightly reduce the response, as indicated by the decreasing trend of the spectrum beyond about 0.8 sec. In contrast, with either Fukiai or Takatori motions (undoubtedly more likely surrogate motions to the unknown real ones), SSI would lead to higher response. The trend becomes more apparent with the Higashi-

Kobe amplified record (site thickness of 50m), for which elastic response at T_{SSI} may exceed 2.5g.

As a first approximation, for the somewhat conservative estimate of SA ≈ 0.93 x 2.1 g x $(5/9.5)^{0.4}$ = 1.47 g, which is derived from the Fukiai record and accounts for both the modal participation factor of the generalized simple oscillator and the increased damping due to SSI, the force reduction factor based on a calculated strength ratio, C_y, of the column of about 0.5 would be equal to approximately 1.47 / 0.5 ≈ 3. Taking the *equal displacement* rule as approximately valid, the ductility demand on the system, $\mu^{(s)}_{demand}$, would be:

$$\mu^{(s)}{}_{\text{demand}} \approx R \approx 3 \tag{7}$$

The ductility demand on the *pier*, $\mu^{(p)}_{demand}$, is obtained by considering only pier deformations. For an elastic perfectly plastic system, this can be done using the expression (Mylonakis & Gazetas 2000)

$$\mu^{(p)}{}_{demand} = (1+c)\mu^{(s)}{}_{demand} - c \tag{8}$$

where *c* is a dimensionless factor expressing the relative flexibility of foundation and superstructure

$$c = K_{pier} \frac{H^2 K_{hh} + 2H K_{hr} + K_{rr}}{K_{hh} K_{rr} - K^2{}_{hr}} \tag{9}$$

For the problem at hand, c = 0.703; thus,

$$\mu^{(p)}{}_{demand} = (1+0.7)\times 3.1 - 0.7 = 4.3 \tag{10}$$

which is 40% higher than system ductility and far exceeds the ductility capacity of the pier (Table 1).

On the other hand, ignoring SSI, and for the conservative value of

$$\text{SA} \approx 0.72 \text{ x } 2.1 \text{ g} = 1.51 \text{ g} \tag{11}$$

which accounts for the participation factor of the generalized system in Table 2, the spectra of Fig. 11 would yield a ductility demand of

$$\mu^{(p)}{}_{demand} \approx R = \frac{1.51}{0.5} = 3 \tag{12}$$

which, although conservatively estimated, does not exceed the upper bound ductility capacity of 3.2 suggested by Kawashima & Unjoh (1997) and, thereby, could hardly explain the spectacular failure of the bridge.

Although approximate, the above results indicate that the role of soil in the collapse could have been triple: *First, the soil* modified (in 1D or 2D fashion) the seismic waves so that the frequency content of the surface motion at the site became disadvantageous for the particular structure (i.e., similar to Fukiai or Takatori, rather than JMA). *Second*, the compliance of soil at the foundation increased the period of the system and moved it to a region of stronger response and, hence, higher inertia. *Third*, ductility demand in the pier increased compared to that of the overall system, as suggested by Eq.(8).

Table 3. Tabulated results from DRAIN-2DX and simplified analyses of the inelastic bridge response

Excitation	Pier Ductility Demand		Increase (%)		Role of SSI	Prediction
	Fixed–Base (A)	Deformable Base (B)	DRAIN-2DX (columns A, B)	Simple model (Eqs. 7 to 10)		
Fukiai	3.1	4.1	32	41	detrimental	failure
Takatori	3.2	7.3	128	46	very detrimental	failure
Motoyama†	3.5 - 3.7	3.2 - 3.5	- 5 to - 9	- 9 to + 62	» minor	probably failure
Higashi†	3.9 - 4.6	4.8 - 6.4	+ 23 to + 39	- 8 to +91	detrimental	failure
JMA	2.5	2.2	-12	-9	slightly beneficial	heavy damage

† Amplified to account for soil effects

Non-Linear Inelastic Analyses

To gain further insight on the importance of SFSI on the inelastic performance of the bridge, a series of non-linear inelastic analyses were conducted using the program DRAIN-2DX. To this end, a multi-degree of freedom (m–dof), inelastic model of the pier was developed, with the column divided into four two-noded inelastic beam elements, each having one translational and one rotational degree of freedom at each end. Concentrated plasticity at the ends of the elements was adopted. The compliance of the foundation was modelled using a series of springs and dashpots attached to the base of the pier. Assuming initial yielding at the observed elevation of 2.5 meters above the cap, a yielding force of 5,636 kN is established, corresponding to a static yielding deck acceleration of about 0.5 g. The inherent (non-SFSI) damping of the structure was assumed of the Rayleigh form, taken equal to 5% of critical. The SFSI dashpots at each degree of freedom were computed from the linear coefficients ζ_{ij} of the foundation impedance at the characteristic period T_{SSI}. Eigenvalue analyses provided the values $T_{fixed} = 0.88$s and $T_{SSI} = 1.07$s which are in good agreement with the results of the simplified model in Table 2. Results obtained with five earthquake records are depicted in Table 3.

For the JMA record, SSI plays a beneficial role, as column ductility demand decreases from 2.5 for the fixed-base pier to 2.2 for the flexibly supported one. In contrast, with Fukiai and Takatori motions, SSI is clearly detrimental, increasing substantially the ductility demand in the pier. In the case of the Fukiai record, the agreement between the numerical results and those in Eqs.(11) and (12) is encouraging for the simple analysis. The strongest SSI effect is observed with the Takatori record: μ increases from 3.2 for the fixed-base structure to the astonishing 7.3 for the flexibly supported–a somewhat fortuitous consequence of the strong peak at about $T \approx 1.2$ seconds.

Substantial increase in ductility with SSI is also observed with the amplified Higashi–Kobe motion, while with Motoyama its role is rather minor. The ranges in computed ductility values for the Motoyama and Higashi stem from the different scenarios of soil thickness used in the amplification analyses. Again, the trends obtained with the simple analysis are in qualitatively corroborated with the numerical study.

The excessive seismic demand computed with Fukiai, Takatori, and Higashi–Kobe records may explain the spectacular failure of the 17 piers of the bridge, especially if one considers the simultaneous deleterious action of the cyclic shear force in the cracked and plastically deforming column cross–section. This suggests that the actual excitation at the site may have indeed resembled the Fukiai, Takatori, or amplified Higashi–Kobe motions much more than the JMA or amplified Motoyama accelerograms

CONCLUSIONS

Analytical and recorded evidence is presented on the triple detrimental role of soil in the collapse of Hanshin Expressway at Fukae. *First*, the soil modified the incoming seismic waves such that the resulting ground surface motion became very severe for the particular bridge. *Second*, the presence of compliant soil at the foundation resulted to an increase in natural period of the bridge which moved to a region of stronger response; *Third*, ductility demand in the pier was higher than the ductility demand of the system, as suggested by Eq .(8). All three phenomena might have simply worsen an already dramatic situation for the bridge due to: *(i)* its proximity to the fault and the strong forward rupture directivity effects which produced very high long–period acceleration normal to the fault, which is exactly in the transverse direction of the bridge; and *(ii)* the structural deficiencies of the pier which were almost unavoidable given the time of design of the bridge (1969).

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