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ABSTRACT

The paper presents a critical assessment of the currently prevailing view of structural engineers, as expressed in seismic codes, that the role of SSI is always beneficial for the design seismic forces developing in a structure. Using recorded strong ground motions and theoretical analyses it is shown that, in certain seismic and soil environments, an increase due to SSI in the fundamental period of a moderately flexible structure may have a detrimental effect on seismic demand, contrary to the conclusion drawn on the basis of idealized ("average") code spectra. Using a simple 2-dof system and a number of actual ground motions as excitation, it is also shown that indiscriminate use of presently popular "geometric" ductility relations may lead to erroneous conclusions in the prediction of seismic performance of flexibly-supported structures. A significant case history, referring to the failure of the 630-m Fukae bridge section of the Hanshin Expressway Route 3 in Kobe (1995), further supports the main findings of the paper, by showing that soil-structure interaction may have played a decisive even if subtle role in that failure.

KEYWORDS

SSI, ductility, inelastic response, seismic regulations, bridge

INTRODUCTION

The way seismic soil-structure interaction (SSI) is presently treated in seismic codes is not free of misconceptions. Despite extensive research in the last 30 years on the subject, there is still controversy regarding the role of SSI in the seismic performance of structures founded on soft soil. In fact, SSI has been declared by structural engineers as *beneficial* for seismic response. Apparently, this perception stems from oversimplifications in the nature of seismic demand adopted in code provisions. The most important of these simplifications (with reference to SSI) are:

- Acceleration design spectra that decrease monotonically with increasing structural period
- Response modification coefficients (i.e., "behavior factors" used to derive design forces) which are either constant (period-independent) or increase monotonically with increasing structural period
- Foundation impedances derived assuming homogeneous halfspace conditions for the soil, which tend to overpredict the damping of structures on actual soil profiles.

The belief for a beneficial role of SSI has also come from analytical studies of the seismic response of elastoplastic oscillators. Results from several such studies, performed for both fixed-base and flexibly-supported systems (e.g., Newmark & Hall 1973; Hidalgo & Arias 1990; Ciampoli & Pinto 1995), have shown that the *ductility demand* imposed on an elastoplastic structure tends to *decrease* with increasing structural period. Other analyses (Miranda & Bertero 1994), however, based on motions recorded on soft soils, indicate that in certain frequency ranges the trend may reverse that is, ductility demand may *increase* with increasing structural period. In addition, theoretical studies by Priestley and Park (1987) showed that the additional flexibility of an elastoplastic bridge pier due to the foundation compliance reduces the *ductility capacity* of the system, an apparently detrimental consequence of SSI.

The objectives of this paper are to:

(a) Review the approach seismic regulations propose for assessing SSI effects on *elastic* single-degree-of-freedom (SDF) oscillators;

(b) Examine the effects of an increase in period, due to SSI, on the ductility demand imposed on *elastoplastic* SDF oscillators.

(c) Evaluate the model of Priestley and Park (1987) for assessing SSI effects in elastoplastic bridge piers.

(d) Demonstrate the surprisingly detrimental role of SSI on the failure of the Fukae section (18 piers) of the elevated Hanshin Expressway in the Kobe earthquake.

With reference to (d), analytical and field evidence is provided in the second part of the paper. Several factors associated with poor structural design have already been identified by earlier investigators. The scope of the herein reported work is to complement the existing studies by examining the role of soil in the failure. Specifically, the following issues are discussed: (1) seismological and geotechnical information pertaining to the ground motion at the site; (2) the free-field soil response; (3) the response of the soil foundation-superstructure system. Analytical results show that the role of soil in the collapse could have been double. First, it modified the seismic wave field so that the predominant frequencies of the surface motion became disadvantageous for the particular structure. Second, the compliance of the soil and the foundation modified the vibrational characteristics of the system and moved it to a region of stronger response. The associated increase in ductility demand on the piers may have exceeded 100% as compared to piers fixed at the base.

CRITICAL REVIEW OF CURRENT SEISMIC PROVISIONS

The presence of deformable soil supporting a structure affects its seismic response in many different ways, as illustrated in Fig 1 (Veletsos, 1977). First, a flexibly-supported structure has different vibrational characteristics, most notably a longer fundamental natural period, $\tilde{T}$, than the period T of the corresponding rigidly-supported (fixed-base) structure. Second, part of the energy of the vibrating flexibly-supported structure is dissipated into the soil through wave radiation (a phenomenon with no counterpart in rigidly-supported structures) and hysteretic action, leading to an effective damping ratio, $\tilde{\beta}$, which is usually higher than the damping β of the corresponding fixed-base structure.

With little exception seismic codes today use idealized smooth design spectra which attain constant acceleration up to a certain period (of the order of 0.4 sec to 1.0 sec at most depending on soil conditions), and thereafter decrease monotonically with increasing period. As a consequence, consideration of SSI leads invariably to *smaller* accelerations and stresses in the structure and its foundation. For example, the reduction in base shear according to ATC-3 is expressed as (Fig 2):

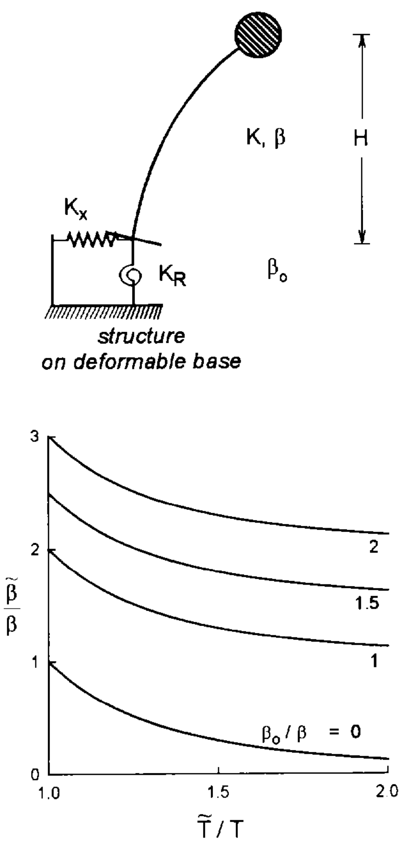

Fig 1. Effect of soil-structure interaction on effective damping according to ATC-3 provisions.

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\beta)\left(\frac{\beta}{\tilde{\beta}}\right)^{0.4} \right] W \tag{1}$$

where

$$\tilde{T} = T\sqrt{1 + \frac{K}{K_X} + \frac{KH^2}{K_R}} \tag{2}$$

$$\tilde{\beta} = \beta_o + \beta\left(\frac{\tilde{T}}{T}\right)^{-3} \tag{3}$$

in which C_s is the seismic response coefficient obtained from the pertinent design spectrum and W is the weight of the structure; the term $(\beta/\tilde{\beta})^{0.4}$ on the right-hand side of Eqn 1 accounts for the difference in damping between the rigidly- and the flexibly-supported structure. *Coefficients* K, K_X, K_R, H, and β_o are indicated in Fig 1.

This "beneficial" role of SSI has been essentially turned into a dogma. Thus, frequently in practice dynamic analyses avoid the complication of accounting for SSI --- a supposedly conservative simplification that would lead to improved safety margins. This beneficial effect is recognized in seismic provisions as, for example, NEHRP-97.

Since design spectra are derived conservatively, the above statement may indeed hold for a large class of structures and seismic environments. But not always. There is evidence documented in numerous case histories that the perceived beneficial role of SSI is an oversimplification that may lead to unsafe design for both the superstructure and the foundation.

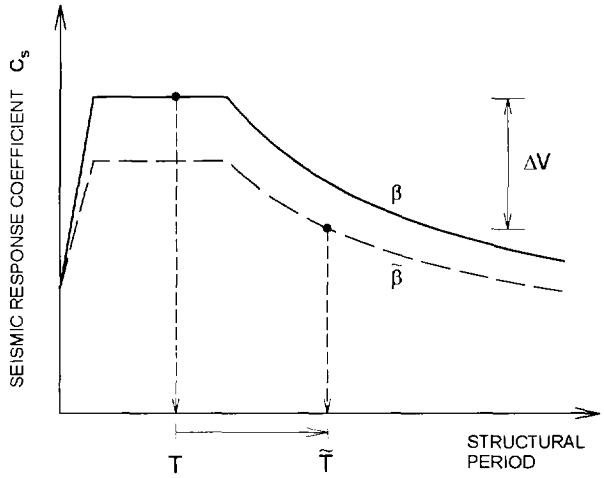

Fig 2. Reduction in design base shear due to SSI according to ATC-3 seismic provisions.

To elucidate this, the ordinates of a conventional design spectrum for soft deep soil, are compared graphically in Fig 3 against four selected response spectra: Brancea (Bucharest) 1977, Michoacan [Mexico City (SCT)] 1985, Kobe (Fukiai, Takatori) 1995, presented in terms of spectral amplification. Note that all the recorded spectra attain their maxima at periods exceeding 1 second. The large spectral values of some of these records are undoubtedly the result of resonance of the soil deposit with the incoming seismic waves (as, for example, in the case with the Mexico City SCT record).

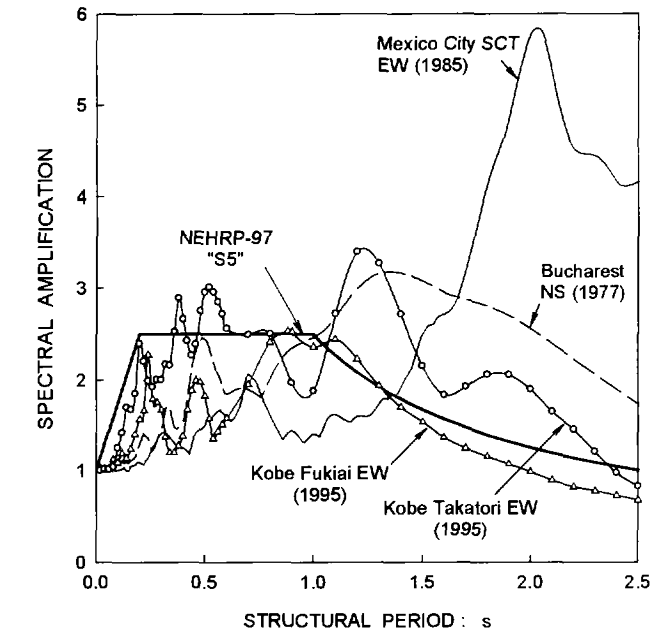

Fig 3. Comparison of a typical seismic code design spectrum to actual spectra from catastrophic earthquakes (β=5%).

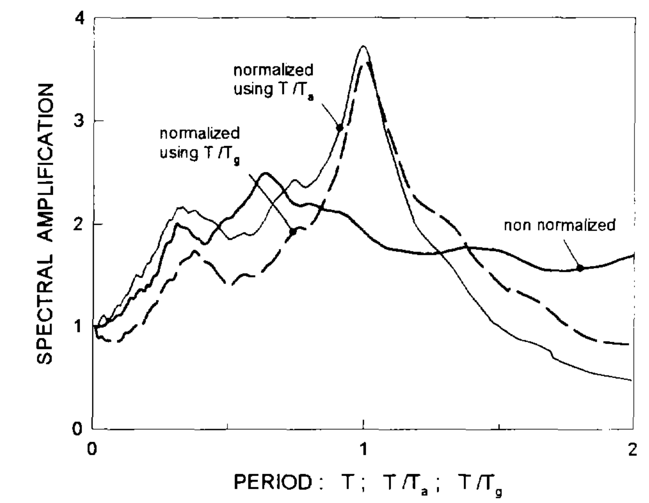

Fig 4. Average acceleration spectra (β=5%) of 24 motions recorded on soft soil normalized with 3 different methods.

To further illustrate the above, results from a statistical study performed by the authors using a large set of motions recorded on soft soil are presented. The set of motions consists of 24 actual records which is an extended version of the set used by Miranda (1991). The average acceleration spectrum obtained from these motions is presented in Fig 4, plotted in terms of spectral amplification. The structural period is presented in three different ways: (i) actual period T; (ii) normalized period T/T_g [T_g = "effective" ground period, defined as the period where the 5% velocity spectrum attains its maximum (Miranda & Bertero 1994)]; (iii) normalized period T/T_a [T_a = period where acceleration spectrum attains its maximum.] It is seen that with the actual period, the resulting average spectrum has a flat shape (analogous to that used in current seismic codes) which has little resemblance to an actual spectrum. The reason for this unrealistic shape is because the spectra of motions recorded on soft soil attain their maxima at different, well separated periods and, thereby, averaging them eliminates their peaks causing this effect. In contrast, with the normalized periods T/T_g and T/T_a the average spectrum exhibits a characteristic peak close (but not exactly equal) to 1, which reproduces the trends observed in actual spectra. It is well known that the issue of determining a characteristic "design" period (i.e., T_g or T_a) for a given site is controversial and, hence, it has not been incorporated in seismic codes. Nevertheless, it is clear that current provisions treat seismic demand in soft soils in a non-rational way, and may provide designers with misleading information on the significance of SSI effects.

Directivity

Another phenomenon, however, of seismological rather than geotechnical nature, the "forward fault-rupture directivity" (Somerville 1998), may be an important contributing factor in the large spectral values at $T > 0.50\ s$ in near-fault seismic motions (e.g., in Takatori and Fukiai). Figure 5 shows the effects of rupture directivity in the response spectra of the JMA and Rinaldi records of the 1995 Kobe and 1994 Northridge earthquakes, respectively. Evidently, records with enhanced spectral ordinates at large periods are not rare in nature --- whether due to soil or seismological factors.

It should be noted that due to SSI large increases in the natural period of structures ($\tilde{T} / T > 1.25$) are not uncommon in relatively tall yet rigid structures founded on soft soil (Tazoh et al 1988; Mylonakis et al 1997; Stewart et al 1999). Therefore, evaluating the consequences of SSI on the seismic behavior of such structures may require careful assessment of *both seismic input and soil conditions*; use of conventional design spectra and generalized/simplified soil profiles in these cases may not reveal the danger of increased seismic demand on the structure.

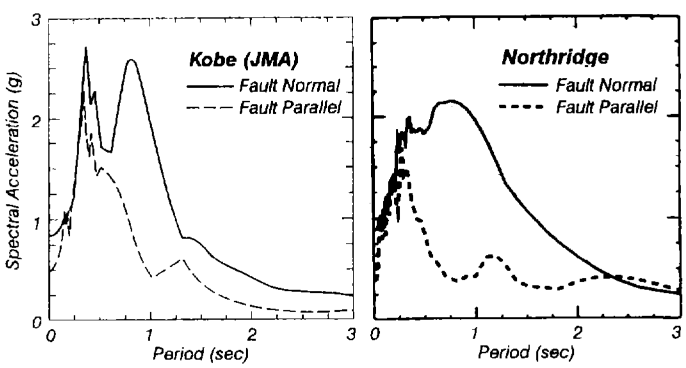

Fig 5. Response spectra for strike-normal and strike-parallel from the 1995 Kobe and 1994 Northridge earthquakes; β=5%.

Increase in Period and Inelastic Response

The foregoing discussion was based on the assumption that the response of the structure is linearly visco-elastic. However, during strong earthquake shaking a structure may exhaust its elastic strength and deform beyond its yielding point (i.e., inelastically) without collapsing. Accordingly, engineers design structures with strength which is only a fraction of that required to prevent yielding (*elastic force demand*), provided that the displacement imposed to the structure by the earthquake (*displacement demand*) is smaller than the ultimate displacement the structure can sustain (*displacement capacity*). The foregoing can be put in a dimensionless form in terms of the following well-known parameters: (i) *ductility demand* μ (= displacement demand / yielding displacement); (ii) *response modification coefficient* R (= elastic force demand / yielding strength).

In a seminal work, Newmark & Hall (1973) proposed two approximate relationships between μ and R. Using a limited number of recorded motions available at that time, they observed that: (1) in the moderately-long and long period ranges, an elastic and an inelastic oscillator of the same initial period have approximately the same maximum relative displacement ("equal displacement rule"); (2) in the moderately short period range, the energy defined by the area under a monotonic force-displacement diagram is approximately the same for an elastic and an inelastic oscillator ("equal energy rule"). Based on these assumptions it is a simple matter to show that μ and R can be related as

$$\mu = \begin{cases} (R^2 + 1) / 2\ , & moderately-short\ periods \\ R\ , & moderately-long\ to\ long\ periods \end{cases} \quad (4)$$

In the limiting case of a very stiff elastoplastic oscillator, $T \to 0$, and its yielding displacement u_y is practically zero. If the system has less strength than that required to remain elastic during shaking ($R > 1$), the ductility demand (computed by dividing the finite displacement response of the system by its zero yielding displacement) will be of infinite magnitude

$$\mu (T \to 0, R > 1) \to \infty \quad (5a)$$

On the other hand, for a very flexible oscillator, $T \to \infty$, and the maximum relative displacement will be equal to the peak ground displacement regardless of yielding strength. This leads to the well-known result

$$\mu (T \to \infty,\ R) = R \quad (5b)$$

Note that contrary to Eqs. 3 which are approximate, the asymptotic relations (5) are exact. (In fact, Eqn 4b is an extrapolation of Eqn 5b to the moderately long period range.)

The trend incited by Eqs. 4 and 5 is clear: For a given R, the ductility demand μ will *decrease* with increasing structural period [i.e., from infinity at zero period, to $(R^2 + 1) / 2$ at moderately long periods, to R at long periods]. Conversely, for a given "target" ductility μ the associated response modification factor R will *increase* with increasing period. *This increase in R implies that the yielding strength required to achieve the pre-specified target ductility will tend to decrease with increasing period, and, accordingly, the role of SSI will be beneficial.*

With relatively few exceptions (e.g., NZS4203, Caltrans 1990), period-dependent strength reduction factors have not been widely incorporated in seismic codes. [This is apparently because such period-dependent factors are difficult to be embodied into multi-mode dynamic analyses.] The work of

Newmark and Hall (1973) has greatly influenced the development of modern seismic regulations; yet this work further strengthens the belief of an always-beneficial role of SSI.

Although several subsequent studies (based primarily on artificial motions or motions recorded on *rock* sites) have more or less confirmed the foregoing trends, analytical results based on motions recorded on *soft soils* trends (see review article by Miranda & Bertero 1994), are in contradiction with the results of Newmark and Hall. Miranda (1991) analyzed a large set of ground motions recorded on a wide range of soil conditions and computed strength reduction factors for a set of pre-specified ductility demands μ. An important finding of his work is that in soft soils (in which SSI effects are typically most pronounced), an increase in structural period may *increase* the imposed ductility demand. To elucidate this, the expression fitted by Miranda & Bertero (1994) to the mean strength reduction factors for soft soil conditions is illustrated in Fig 6, plotted in terms of ductility demand μ versus structural period. The period is normalized by the predominant period T_g of the record. Also plotted in the graph are the generic expressions of Newmark & Hall (1973) (Eqns 2 to 5), normalized using $T_g = 1$ sec. For periods higher than about 1.2 T_g, the ductility demand becomes an *increasing* function of period, which contradicts to the trends suggested by Newmark & Hall (1973). Note that the increasing trend becomes stronger for weaker oscillators (i.e., for higher R values). Similar trends have been presented by Nassar & Krawinkler (1991).

The foregoing discussion considered structures that are perfectly fixed at the base. The effects of SSI could only be studied indirectly (i.e., through the increase in natural period). A more accurate study of the inelastic response of *flexibly-supported* structures is presented below. It will be

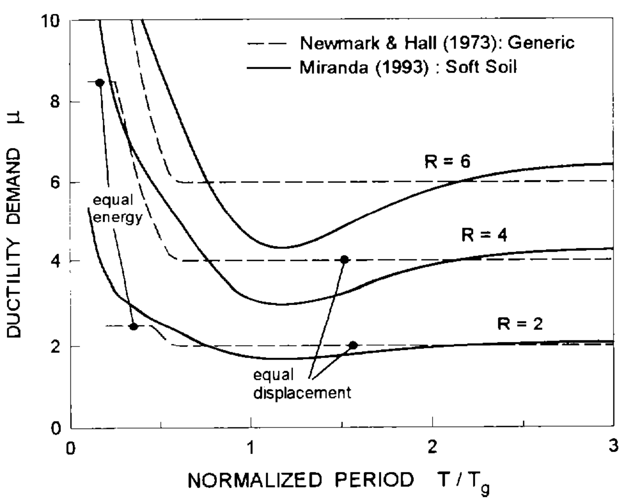

Fig 6. Ductility demand vs. dimensionless structural period (β=5%). Comparison of Newmark & Hall (1973) with Miranda (1993).

shown that, in this case, the interpretation of ductility coefficients may involve pitfalls.

DUCTILITY COEFFICIENTS IN FLEXIBLY-SUPPORTED STRUCTURES

To assess the effects of soil flexibility on the inelastic response of structures (particularly bridges) engineers have been using the simple structural idealization of Fig 7: a single bridge pier connected to the deck monolithically (or through bearings), and subjected to a transverse seismic excitation (Priestley & Park 1987, Ciampoli & Pinto 1995). Elastoplastic bilinear behavior is usually considered for the pier, while the soil-foundation is modeled with translational and rotational springs. Moment-free (cantilever) conditions at the deck are often assumed. A simple approach has been proposed for evaluating the effects of SSI on the seismic performance of the inelastic system, by subjecting the bridge

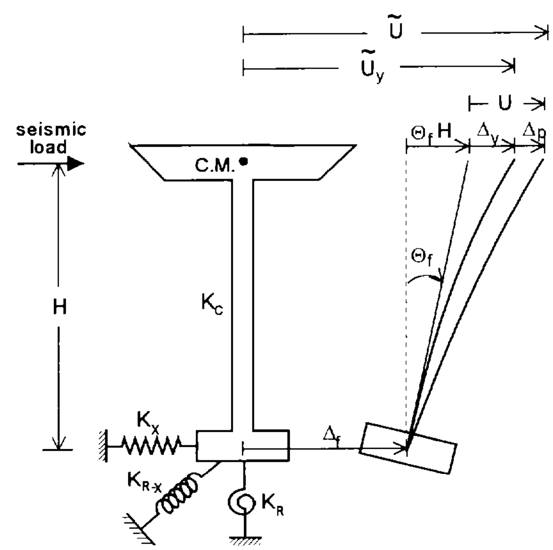

Fig 7. The model used to investigate the significance of SSI in the inelastic seismic performance of cantilever bridge piers.

pseudostatically to a lateral load.

The lateral displacement of the deck relative to the free-field soil, $\tilde{U}$, can be decomposed as (Priestley & Park 1987):

$$\tilde{U} = \Delta_f + \Theta_f H + \Delta_y + \Delta_p \qquad (6)$$

in which:

- Δ_f and (Θ_f x H) are rigid body displacements of the pier due to the swaying (Δ_f) and rocking (Θ_f) of the foundation, respectively
- Δ_y and Δ_p represent the yield and plastic displacement of the pier, respectively. [Presence of bearings is not considered for simplicity]

- $\Delta_y = F_y / K_c$ in which F_y is the yield shear force and K_c ($\sim 3 E_c I_c / H^3$) the stiffness of the column
- Δ_p is the plastic component of deck displacement due to the yielding of pier, which is concentrated at the base of the column ("plastic hinge").

If the column were fixed at its base, Eqn 6 would simplify to:

$$U = \Delta_y + \Delta_p \tag{7}$$

and dividing by Δ_y would yield the displacement ductility factor of the *column*

$$\mu_c = \frac{\Delta_y + \Delta_p}{\Delta_y} \tag{8}$$

For the flexibly-supported system, the yielding displacement, $\tilde{U}_y$, of the bridge is obtained by setting $\Delta_p = 0$ to Eqn 6:

$$\tilde{U}_y = \Delta_f + \Theta_f H + \Delta_y \tag{9}$$

The ratio $\tilde{U} / \tilde{U}_y$ defines the so-called "global" or "system" displacement ductility factor of the bridge-foundation system:

$$\mu_s = \frac{\tilde{U}}{\tilde{U}_y} = \frac{\Delta_f + \Theta_f H + \Delta_y + \Delta_p}{\Delta_f + \Theta_f H + \Delta_y} \tag{10}$$

Dividing by Δ_y yields the dimensionless expression between μ_s and μ_c:

$$\mu_s = \frac{c + \mu_c}{c + 1} \tag{11}$$

in which

$$c = (\Delta_f + \Theta_f H) / \Delta_y \tag{12}$$

is a dimensionless coefficient expressing the foundation to structure displacement.

Equations 9 and 10 implicitly assume that the response of the foundation, $(\Delta_f + \Theta_f \times H)$, is the same in both yielding and ultimate conditions. This assumption holds for an *elastic-perfectly plastic* pier supported on a foundation with higher yielding strength than the column. For a pier with bilinear behavior, Eqs 10 and 11 should be replaced by

$$\mu_s = \frac{\tilde{U}}{\Delta_y (1 + c)} \tag{13}$$

From Eqn 11, it is evident that for $c = 0$ (a structure fixed at its base), the values of μ_s and μ_c coincide. For $c > 0$, however, μ_s is always smaller than μ_c, decreasing monotonically with increasing c. In fact, in the limiting case of $c \rightarrow \infty$ (an infinitely-flexible foundation or an absolutely rigid structure), the "system" ductility μ_s is 1 regardless of the value of μ_c.

The above trends have been widely interpreted in the following way (Priestley & Park 1987; Ciampoli & Pinto 1995): *Given a ductility capacity μ_c of the column ($\mu_c > 1$), the ductility capacity μ_s of the SSI system associated with a flexibility ratio $c > 0$ is lower than it would be for a fixed-base cantilever (with $c = 0$).* As an example, for the typical values $\mu_c = 4$ (a well-designed column) and $c = 1$ (a moderately soft soil), μ_s is equal to only 2.5, i.e., only 62% of the μ_c value.

On the other hand, to achieve a certain ductility capacity for the system, say $\mu_s = 4$, the ductility capacity of the column for $c = 1$ should, according to Eqn (11), be $\mu_c = 7$, which may require a substantial increase in deformation. This implies that the additional flexibility due to the foundation compliance reduces the ductility capacity of the system (Priestley & Park 1987; Ciampoli & Pinto 1995). As a straightforward extension to the above statement, one may conclude that *soil-structure interaction has a detrimental effect on the inelastic performance of a bridge-foundation system by reducing its ductility capacity.* This is in apparent contradiction with the "beneficial" role of SSI discussed earlier. Although evidence for a detrimental role of SSI has already been discussed (and additional such evidence will be presented later on), it will be shown here that drawing such a conclusion using Eqn 11 is incorrect. This is done using two different approaches.

First, consider a counterexample: Suppose that we are interested in the ductility *demand* (i.e., instead of capacity) imposed to the system by a transient dynamic load. To calculate the ductility demand one must solve the non-linear equation of motion of the system to determine the peak plastic displacement Δ_p (as, for instance, done by Ciampoli & Pinto 1995). For any value of Δ_p, however, Eqn 11 will yield smaller values than Eqn 8 due to the presence of the additional positive number c in both the numerator and denominator. Thus, the ductility demand imposed on the SSI system will be smaller than that of a fixed-base system with the same vibrational characteristics and, thereby, SSI will apparently have a *beneficial* role to the system's performance --- exactly the opposite to the first interpretation.

The apparent paradox stems from the fact that Eqn 11 is a *kinematic* expression which does not distinguish between capacity and demand; it tends to reduce both ductilities and provides no specific trend on the effect of SSI on the inelastic performance of the system.

The second argument against the validity of μ_s as performance indicator is the presence of rigid body displacements (due to the foundation translation and rotation) which are not associated with strain in the pier. In fact, the addition of these displacements in both the numerator and denominator of Eqn 10 is the only reason for the systematic drop in that ratio. This implies that the ductility ratio μ_s (expressed through Eqs. 10-12) is *not* a measure of the distress of the pier, as correctly pointed out by Ciampoli & Pinto (1995). For example, additional rigid body motions could be introduced to the analysis by, say, rotating the reference system during the response. This would reduce μ_s, but without having any physical connection to the actual problem. As another example, one may introduce to Eqn 10 the seismic ground displacement. Incorporation of this additional displacement would better reflect the absolute motion of the system, and further reduce μ_s. As an extreme case, one may consider the translation due to the motion of the earth; the addition of such a huge displacement to both the numerator and denominator of Eqn 10 will make μ_s equal to 1 (implying response without damage) even for a system that has failed!

Finally, it is important to note that μ_s in Eqn 10 was derived by examining just the static deflection of the system, i.e. without using time history analysis or any "dynamic" reasoning. In contrast, it is well known that seismic SSI effects are influenced (if not governed) by dynamic phenomena such as resonance and de-resonance which cannot be captured by purely static or geometric considerations.

INELASTIC RESPONSE ACCOUNTING FOR SSI

To further investigate the role of SSI on the inelastic performance of bridge piers, non-linear inelastic analyses were carried out using the model of Fig 7. Both column and system ductilities were obtained using different oscillators and ground excitations, and results were compared with corresponding demands for fixed-base conditions. A similar investigation has been performed by Ciampoli & Pinto (1995). However, there are some differences between the two studies. While the foregoing study was based on a set of *artificial* ground motions matching the EC-8 (1994) spectrum for intermediate-type soils, the present study uses exclusively *actual* motions recorded on soft soils. In addition, a two-degree-of-freedom system is adopted here (as opposed to a single-degree-of-freedom system in the earlier work), to better represent the dynamic response of the footing. In the present analyses, bilinear elastoplastic behavior is considered for the pier with post-yielding stiffness equaling 10% of the elastic stiffness. A footing mass equal to 20% of the deck mass and a Rayleigh damping equal to 5% of critical in the two elastic modes of the system were considered in all analyses.

Figure 9a presents column ductility demands obtained using the Bucharest (1977) motion. The results are plotted as function of the fixed structural period computed for four different foundation-to-structural flexibility ratios: $c = 0$ (which corresponds to fixed-base conditions), 0.25, 0.5, and

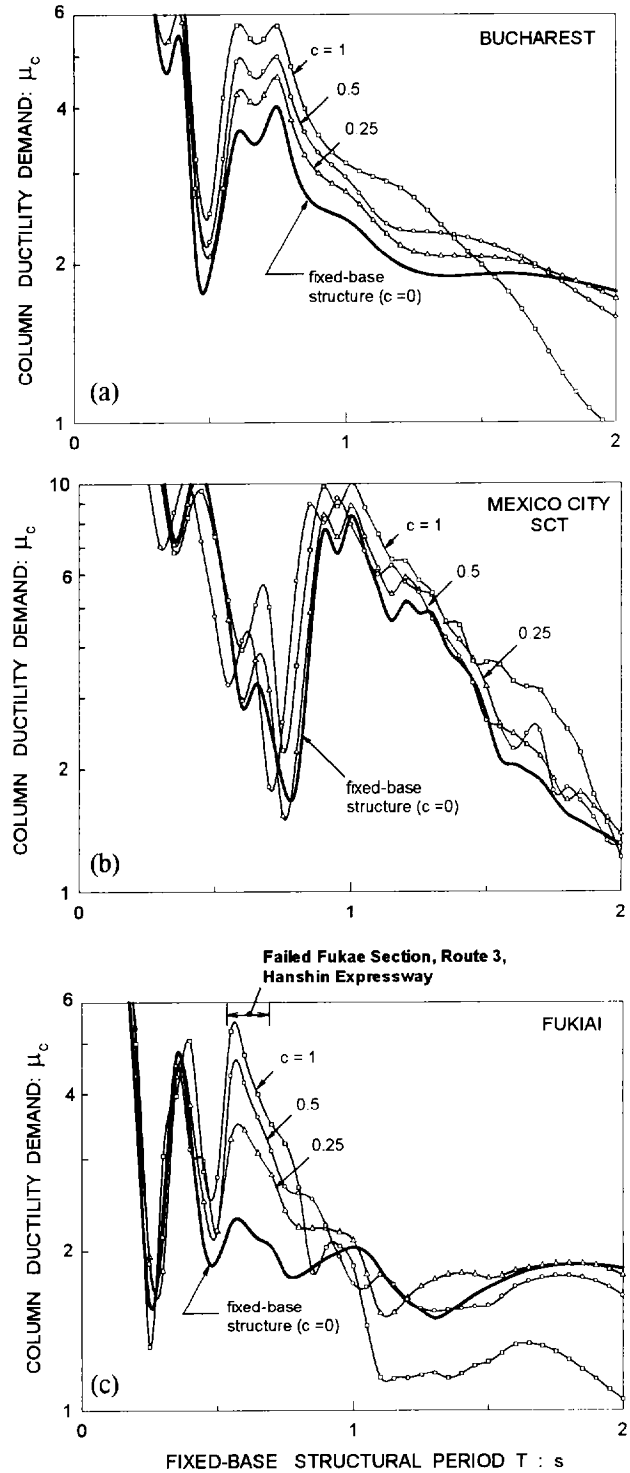

Fig 9. Effect of SSI on ductility demand of a bridge pier subjected to: (a) Bucharest Brancea (1977) N-S motion; (b) Mexico Michoacan (1985) E-W motion; (c) Kobe Fukiai (1995) E-W motion; R = 2.

1. [Note that with $c = 1$ the period of the flexibly-supported system is $(1+c)^{1/2} \approx 1.4$ times higher that that of the corresponding rigid-base system.] In the period range between about *0.5* to *1.5* seconds, the curves for $c > 0$ plot above that for $c = 0$ which implies that SSI increases the ductility demand in the pier. For example, in the particular case $T = 0.6$ *sec*, $c = 1$ ductility increases from *3.5* to about *5.5* without $(c = 0)$ and with $(c = 1)$ SSI, respectively. In smaller periods the increase in μ_c is less significant, while at longer periods SSI tends to reduce ductility demand.

Figure 9b refers to the Mexico City SCT (1985) record *and* $R = 2$. In this case the effects of SSI are somewhat less significant than in the previous graph. Yet, the tendency for increase in ductility due to SSI is evident with the curves for $c > 0$ plotting above that of $c = 0$ for periods between *0.70* and *2* seconds. Incidentally, it should be mentioned that most of the damage caused by this earthquake concentrated in buildings with fundamental fixed-base periods varying from about *0.9* to *1.3* seconds, which coincides with the region of the maxima of this graph.

An interesting case is presented in Fig 9c referring to the Kobe (1995) Fukiai record. A substantial increase in ductility due to SSI is observed at periods between about *0.5* and *1* seconds. For example, with a fixed-base period of 0.6 seconds and $c = 1$, the ductility demand increases from *2.2* for the fixed-base pier $(c = 0)$ to more than *5* for the flexibly supported system. It is important to mention here that the 18 piers (a 630 m segment) at Fukae section of the elevated Hanshin Expressway that failed spectacularly in that earthquake, had a fixed-base natural period of about 0.6 seconds, located at a site having similar soil conditions and located similarly with respect to the fault zone as the Fukiai and Takatori sites. The role of SSI on the collapse of that structure was perhaps more significant than originally suspected. More details on this failure are given later on.

Figures 10a and 10b present *system* ductility demands (Eqn 13) obtained from the Bucharest and Mexico City records. It is apparent that the use of *system* ductility completely obscures the detrimental role of SSI observed in Figs 9.

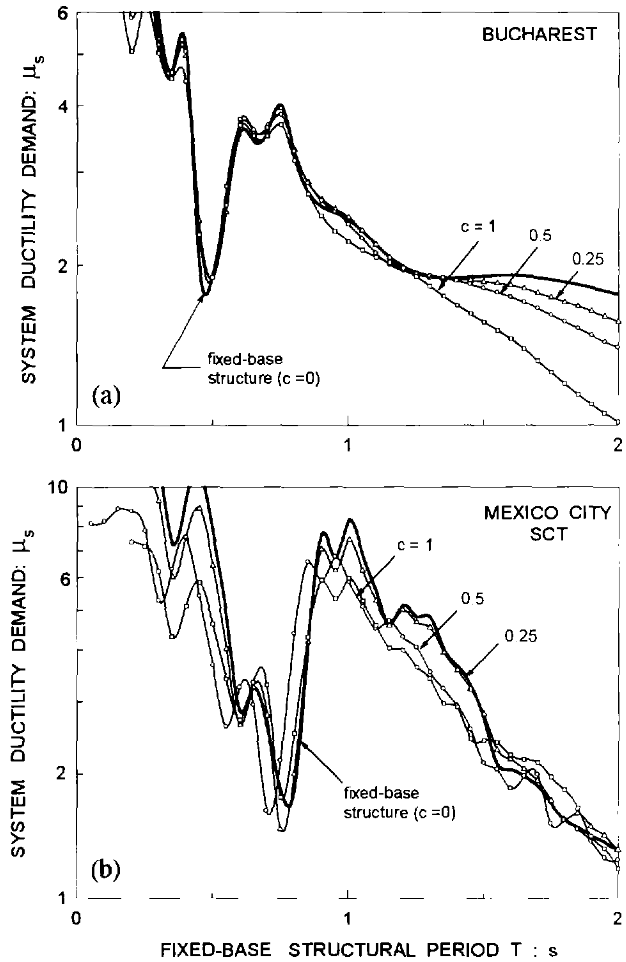

Fig 10. Effect of SSI on ductility demand of a bridge pier subjected to: (a) Bucharest Brancea (1977) N-S motion; (b) Mexico Michoacan (1985) E-W motion; R = 2. Note the reduction in ductility as compared to Figs 9.

COLLAPSE OF 18 PIERS OF THE HANSHIN EXPRESSWAY IN KOBE (1995)

In the devastation caused by the Kobe earthquake, the structural collapse and overturning of the 630m Fukae section of Hanshin Expressway was perhaps the most spectacular failure. The bridge was part of the elevated Hanshin Expressway Route No 3 that runs parallel to the shoreline. Built in 1969, it consisted of single circular columns 3.1 meters in diameter and about 11 meters in height, founded on groups of 17 piles. The columns were connected monolithically to a concrete deck. A cross section of the bridge is shown in Fig 11.

Detailed structural investigations of the behavior of Higashi-Nada bridge have been presented by Park (1996) and Kawashima & Unjoh (1997). In those studies several factors contributing to the collapse associated with poor structural

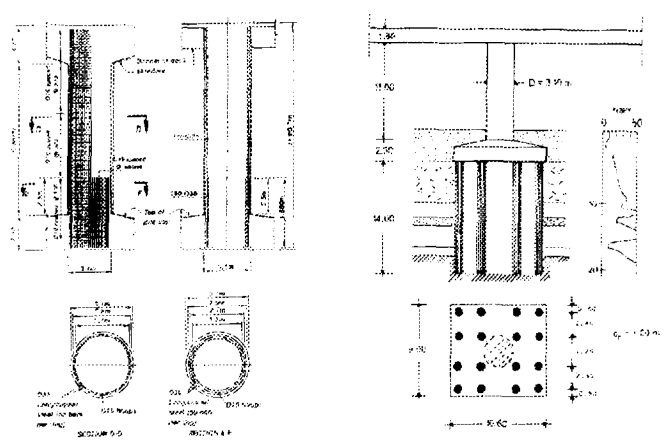

Fig 11. Characteristics of Hanshin Expressway Piers at Fukae Section.

design were identified. These factors include: (i) inadequate transverse reinforcement in the piers; (ii) inadequate anchorage of longitudinal reinforcement; (iii) use of elastic methods (instead of capacity design procedures) for determining the design shear forces in the piers. Notwithstanding the importance of these parameters, there is evidence that local soil conditions and dynamic interaction between the foundation and the superstructure further contributed to the collapse.

The First Role of Soil: Influence on the Pattern and Intensity of Ground Motion

Kobe is built in the form of an elongated rectangle with length of about 30 km and width 2-3 km along the shoreline. The soil in the region consists primarily of sand with gravel of variable thickness (10-80m), underlain by soft rock. The granitic bedrock that outcrops in the mountain region to the north of the city dips steeply in the northwest-southwest direction; in the shoreline it lies at a depth of about 1 to 1.5 km. Fig 12 shows an approximate geological plan and a cross section of the region as well as the locations of nearby strong motion accelerometers. Different soil thickness from one recording station to another may be responsible for the significant differences in the intensity and frequency content of the recorded motions (Soils & Foundations 1996).

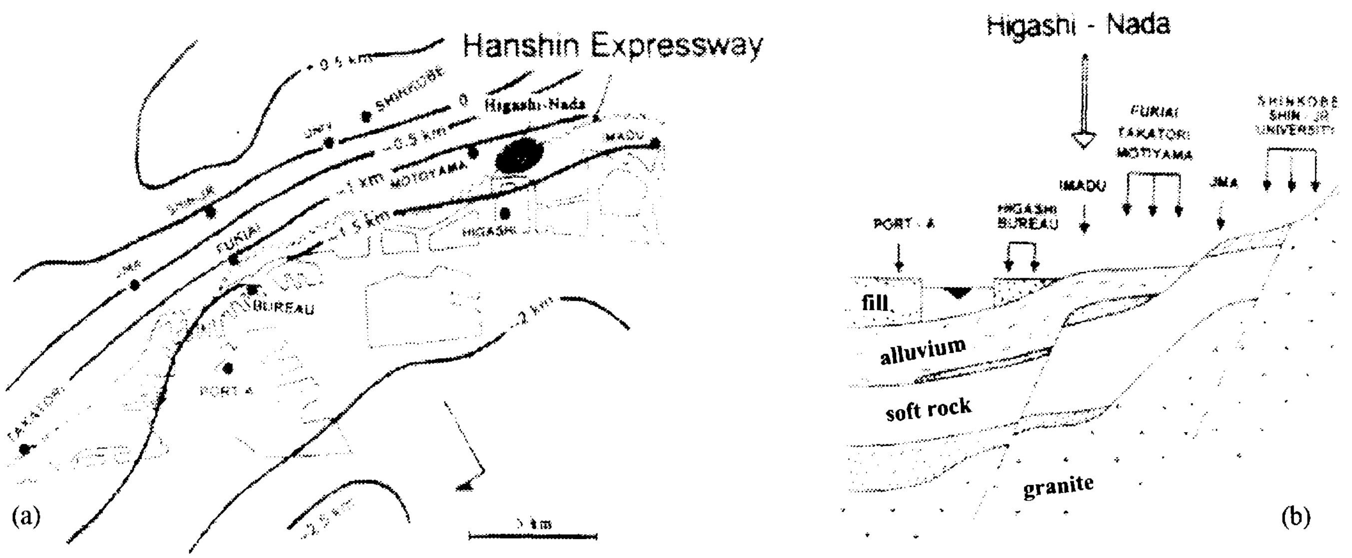

Fig 12. (a) Contours of bedrock elevation and location of accelerometers; (b) Approximate geological section of (a).

The differences in soil thickness at various locations were among the reasons for the differences in the recorded spectra, as shown in Fig 13. Of course, seismic directivity was one of the phenomena that took place in Kobe: it undoubtedly led to the large differences between spectral values in directions normal and parallel to the fault rupture zone as well as to pronounced vertical motions (not shown) and large spectral values at periods around 1 second in rock. Notwithstanding the importance of these effects, it is believed that soil further amplified the incoming seismic waves and produced variations in the characteristics of the records depending on the differences in the local soil conditions from site to site.

The Second Role of Soil: Soil-Pile-Superstructure Interaction

The bridge consisted of 19 single circular columns, 3.1 meters in diameter and about 11 meters in height, monolithically connected to the deck and founded on groups of 17 reinforced concrete piles. The piles have length of about 15 m and diameter of 1 m, connected through a rigid 11×11 m cap. The soil surrounding the piles consists of medium dense sand with gravel. SPT values for the upper 20 meters of the soil are given in Fig 11. Corresponding shear wave velocities were found to vary between 200 to 300 m/s down to 30 m depth. The structural parameters used in the following paragraphs have been taken from Park (1996), Kawashima & Unjoh (1997) and Michaelides (1997).

The mass of the bridge during the earthquake was found to be about 1,100 Mg while the rotational moment of inertia of the deck with respect to the longitudinal axis was about 40,000 Mg m^2. The horizontal stiffness of the pier (uncracked conditions) was estimated to be of the order of 150 MN/m. In addition, it was found that about 0.7g horizontal acceleration at the bridge deck was needed to cause the column to reach its probable yield strength, and that the available displacement ductility capacity of the column was of the order of 2. From the above data, assuming the pier to be perfectly fixed at the base and considering the rotational inertia of the deck, the fundamental low-strain period of the bridge is estimated to be (Michaelides 1998)

$$T_s \approx 0.65\,sec \qquad (13)$$

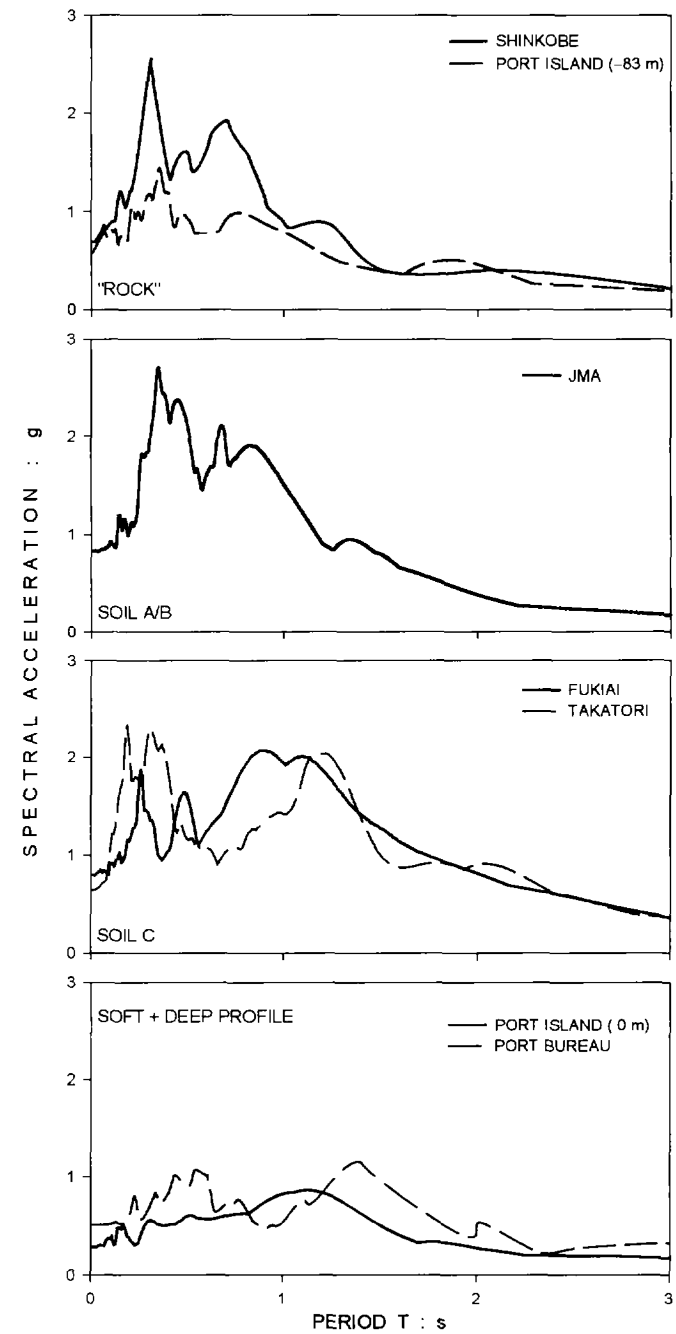

Fig 13. Selected Acceleration response spectra of main records during the Kobe earthquake; β = 5%.

Note that, if cracked conditions had been assumed for the pier, the fixed-base period would increase to about 0.75 sec (Kawashima & Unjoh 1997).

The dynamic interaction between soil, foundation and superstructure increases substantially the natural period of the bridge. An estimate of the period of the system can be obtained from the approximate formula of NEHRP-97 once the compliance of the pile group has been determined. More appropriately, Mylonakis et al (1997) and Gerolymos (1997) have studied the bridge and found that its actual period, including SSI, was approximately

$$\tilde{T} \approx 0.93\,sec \tag{14}$$

This preliminary result highlights the possible role of SSI as it increases the effective period of the bridge by an appreciable 30%. The result in Eqn. (14) was verified with more comprehensive analyses using the computer code SPIAB (Mylonakis et al 1997).

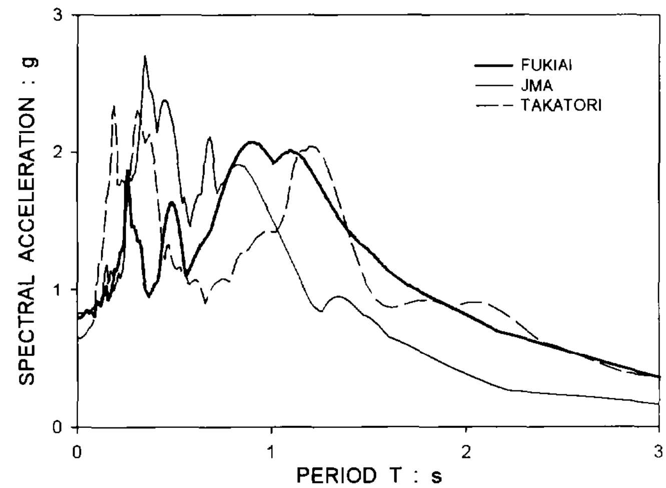

Fig 14. Acceleration response spectra of main records during the Kobe earthquake (β = 5%).

The uncertainty on the exact characteristics of the soil profile at the location of bridge (the nearest complete soil profile available to the authors was about 300 m away from the end of the collapsed segment), dictated the use of different scenaria regarding the seismic excitation at the bridge site. Three acceleration records (Fig 14) with peak ground acceleration ranging between 0.65 and 0.83 g and quite different frequency characteristics were used in the analyses:

- The accelerogram *JMA*, with a peak value of 0.83g, was recorded on a relatively stiff soil formation (thickness of soft soil about 10-15 m)
- The accelerogram *Fukiai*, with a peak value of about 0.80 g, was recorded on a softer and deeper deposit (thickness of alluvium about 70 m)
- The accelerogram *Takatori*, with a peak value of 0.65 g, was recorded on a soft and deep deposit (thickness of alluvium about 80 m)

It should be noted that, as suggested by the geology of Fig 12, the soil conditions at the bridge location seem to be closer to those of Fukiai and Takatori, rather than to JMA.

From the spectra of Fig 14, the effect of SSI on the response of the bridge start becoming apparent. 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 ("de-resonance") of the spectrum beyond about 0.8 sec. In contrast, with either Fukiai or Takatori as excitation, SSI would lead to progressively larger accelerations in excess of 1g. As a first approximation, for a best estimate of SA = 1.4 g, the force reduction factor based on a calculated strength of the column of about 0.7g would be equal to approximately 2. Taking the equal displacement rule as approximately valid, the ductility demand on the pier would be:

$$\mu_d \approx 2 \qquad (15)$$

which is probably higher than the corresponding ductility capacity due to the inadequate transverse reinforcement of the pier (Park 1996; Michaelides 1998).

Detailed analyses were also performed to verify the results of the above simplified analysis. They include: (i) equivalent-linear SSI analyses (using the computer code SPIAB) and (ii) non-linear dynamic analyses in which the foundation stiffness had been computed independently. A typical set of results of the SPIAB analyses (from Michaelides 1998) is shown in Fig 15 using the Fukiai record as excitation. The acceleration histories predicted for the deck with and without SSI exhibit different peaks and different frequency characteristics. Indeed the complete response (with SSI) is 25% higher than the response assuming fixed base --- a perhaps crucial difference contributing to the failure of the bridge.

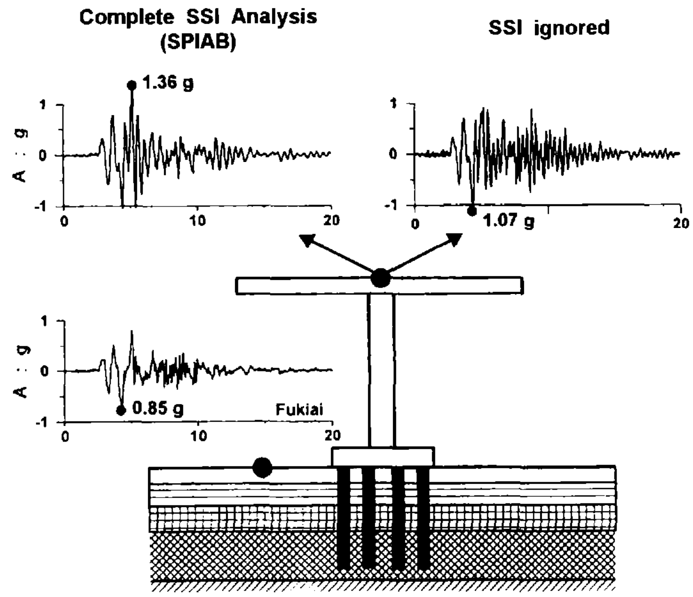

Fig 15. Effect of dynamic SSI on the acceleration response of the failed Route 3 Section of the Hanshin Expressway.

Non-linear Inelastic Analyses

To gain further insight on the importance of SSI on the performance of the system, a series of non-linear inelastic analyses were performed. To this end, a 2-degree-of-freedom inelastic model of the bridge was developed which is similar to that used by Ciampoli & Pinto (1995). In the present study, the compliance of the foundation was modeled using a series of linear springs and dashpots attached at the base of the pier. A yielding strength of 7,500 kN was considered for the pier, corresponding to an estimated yielding deck acceleration of 0.7g. A post yielding stiffness equal to 10% of the elastic stiffness of the pier was also assumed. The mass of the pile cap plus ½ the mass of the pier were considered lumped at the base of the pier (m_b = *541 Mg*). 5% and 15% damping under elastic conditions were assumed for the superstructure and the foundation respectively. All three earthquake records (JMA, Fukiai, Takatori) were used in the analyses.

The role of SSI on the performance of the bridge is visible in Table 1. First, using the JMA record the ductility demand in the column decreases when SSI effects are considered (i.e., from 2.86 for the fixed-base structure to 2.58 for the flexibly-supported). In contrast, with Fukiai and Takatori records SSI is detrimental, increasing the ductility demand in the pier. This is in (qualitative) agreement with the trends observed in Fig 14. In particular, Fukiai excitation leads to a substantial

Table 1. Tabulated results from inelastic analyses of the bridge response.

Excitation	Analysis type	Effective natural period (s)	Peak deck acceleration (g)	Peak deck displacement (cm)	Peak drift displacement (cm)	R	System ductility μ_s	Column ductility μ_c	Role of SSI
JMA	fixed base	0.65	0.87	21.0	21.0	2.67	2.86	2.86	*beneficial*
	flexible base	0.93	0.89	21.0	19.0	2.13	1.93	2.58	
Fukiai	fixed base	0.65	0.80	14.6	14.6	1.95	1.99	1.99	*detrimental*
	flexible base	0.93	1.02	41.9	31.2	2.56	2.79	4.24	
Takatori	fixed base	0.65	0.74	10.0	10.0	1.36	1.34	1.34	*detrimental*
	flexible base	0.93	0.79	24.4	13.4	1.67	1.62	1.83	

increase in μ_c : from *1.99* to *4.24* without and with SSI, respectively. The latter value is much higher than the estimated displacement ductility capacity of the pier ($\mu_{capacity} \approx 2$), and it is in agreement with Fig 9. Such an excessive amount of seismic demand may explain the spectacular failure of all 17 piers of the bridge. Accordingly, this could indicate that the actual excitation at the site resembled more the Fukiai rather than the JMA or Takatori excitations. It is also worth mentioning that the "system ductility" μ_s is always smaller than μ_c and provides a misleading index for assessing the bridge performance (it does not reflect the actual distress in the pier). Nevertheless, μ_s appear to match better the values of R (for the relatively long period system examined herein), so μ_s may be more appropriate for developing R-μ relationships.

CONCLUSIONS

The main conclusions of this study are:

(1) By comparing conventional code design spectra to actual response spectra, it was shown that an increase in fundamental natural period of a structure due to SSI does not necessarily lead to smaller response. The prevailing in structural engineering view of an always beneficial role of SSI is an oversimplification which may lead to unsafe design.

(2) Averaging response spectra of motions recorded on soft soil without proper normalization of periods may lead to errors.

(3) Ductility demand in fixed-base structures is not necessarily a decreasing function of structural period, as suggested by traditional design procedures. Analysis of motions recorded on soft soils have shown increasing trends at periods higher than the predominant period of the motions.

(4) Soil-structure interaction in inelastic bridge piers supported on deformable soil may cause significant increases in ductility demand in the piers depending on the characteristics of the motion and the structure. However, inappropriate generalization of ductility concepts and geometric considerations may lead to the wrong direction when assessing the seismic performance of such structures.

(5) The role of soil in the collapse of the Hanshin Expressway was double and detrimental: *First*, it modified the incoming seismic waves and the resulting surface motion became detrimental for the bridge (amplification of spectral accelerations in the range $T \approx 0.8$ to 1.3 secs). *Second*, the presence of compliant soil at the foundation resulted to an increased natural period of the bridge which moved to a region of stronger response. For the Fukiai record, the increase in seismic demand due to SSI was dramatic. Of course, both phenomena might simply worsen an already dramatic situation for the bridge due to its proximity to the fault and inadequate structural design.

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