SSI ISSUES IN TWO EUROPEAN PROJECTS AND A RECENT EARTHQUAKE

George GAZETAS
National Technical University, Athens, Greece

ABSTRACT : Several issues relating to soil-structure interaction (SSI) phenomena that are not presently fully resolved are critically addressed. They refer to (i) the dynamic response under kinematic and inertial loading of large pile groups in very soft soil; and (ii) uplifting–sliding of surface foundations, and SSI under large overturning moments that may lead to bearing-capacity type failures. The capabilities and limitations of the current state of the art in these topics are summarized. Two projects in Europe and a case history from the Izmit 1999 earthquake are used for illustrating the importance and the specific resolution of these issues in practice.

1. INTRODUCTION

There is little doubt that the science and "art" of SSI analysis is in a mature state : evidenced not only from the wealth in the related literature but also from the abundance of commercially available computer codes that can treat many of the practical problems, with at least "engineering" accuracy. Far less developed is the treatment of SSI in seismic code provisions, to the point that many codes do not contain detailed procedures or guidance for incorporating SSI effects in design. Furthermore, some real-life situations where SSI is judged as significant cannot be *routinely* handled with available technology. The paper outlines a few such situations using two European projects as examples. Emphasis is placed on highlighting unresolved issues.

2. LARGE PILE GROUP IN SOFT CLAY (The Case of the Brunsbuettel Nuclear Power Plant)

This is not an infrequent situation, as industrial facilities and infrastructure are often built in alluvial plains containing soft clays --- piled foundations are then an attractive solution. In Japan, this may be a most frequent situation. A particularly interesting case, involving a fully instrumented 64-pile foundation for a bridge pier on very soft clay, is the case of Ohba Ohashi near Fujisawa city (Tazoh et al 1988); its recorded seismic response has been the subject of theoretical investigations (Tazoh et al 1989, Gazetas et al 1994). An example from the eastern USA: the seismic re-evaluation of the historic Williamsburg Bridge, which involved theoretical investigation of the response of block foundations on a large number of piles penetrating through soft soils. Several other references can be found in the recent earthquake engineering literature, involving groups of very large number of piles.

The present case study refers to the seismic re-evaluation of a major critical facility in Germany, founded on a large embedded mat supported on 230 piles. The geometry of the group is sketched in **Fig. 1** (plan and section). The 1.3 m diameter piles pass through 9.5 meters of very soft saturated organic clay having S-wave velocity V_s = 80 m/s and mass density ? = 1.5 Mg/m^3. The piles are socketed in a stiff sandy layer having V_s = 330 m/s, into which they penetrate 6 meters.

The foundation transmits the loads of the superstructure through :

- a group of 230 piles, and
- a vertical sidewall of the embedded structure.

No force-transmitting contact is believed to exist between the base of the foundation mat and the soil that could be relied upon to transmit shear or normal loads onto the soil.

The foundation aspects of seismic SSI analysis were focused on obtaining : first, the dynamic impedances and, second, the seismic response of the pile group and embedded foundation to an earthquake with 0.11 g peak rock-outcrop acceleration. The seismic response was obtained with due consideration to both *kinematic* and *inertial* effects.

2.1 Methods of Computing Pile-Group Impedance

Three different methods, all based on *elastodynamic* theory, have been applied to obtain the dynamic impedances of the pile group :

(a) A rigorous semi-analytical method which accounts for the layered soil profile and the pile-to-pile interaction, including the "shadow" forming of the in-between piles (Kaynia, 1982). This method, however, requires extremely large computer capacity, and cannot handle the complete pile group. Thus, in order to compute the overall impedances the 230-pile group was divided into eight clusters of pile sub-groups, as shown in Fig. 1 ; evidently, the division is a "natural" one. Extensive comparisons were performed between rigorous and simplified methods for the sub groups.

(b) A superposition method, which generates rigorous interaction factors between any two piles in the group and uses them (internally) to compute the impedances of the whole group. The shadow--forming by the in-between piles cannot be taken into account. This method also requires a large computer capacity, since for the 230 piles in the group the number of different distances between individual piles is huge ; linear interpolation introduces errors stemming from the oscillatory nature of both the real and the imaginary parts of the interaction factors. For all these reasons, this method is used only to validate the superposition procedure, and then use it with the simplified approach (c) described below.

(c) A simplified interaction-factor method, which uses rigorous impedances of the single pile and semi-analytically derived interaction factors between two piles at different spacings. These factors are obtained using the aforementioned boundary-element-based results in conjunction with the wave-interference analytical solution of Dobry & Gazetas (1988) and Mylonakis & Gazetas (1998a and b).

A major potential drawback of the first two methods is that they can not treat soil nonlinearity. Only the simplified third method could very crudely account for nonlinear soil effects. For the specific project at hand, however, the small intensity of motion limits the role of soil nonlinearity.

2.2 Interaction Factors

The complex-valued interaction factors were obtained for a number of pile separation distances, from the closest ($s = 2$ m $\cong 1.7$ d) to the largest ($s = 70$ m $\cong 45$ d). **Fig. 2** plots the amplitudes of these factors in three directions.

We notice an unusual but explainable behavior in the interaction factors A_y at large distances, $s/d > 10$: the amplitude $\left|A_y\right|$ retains quite large values for frequencies between 3 and 8 Hz , and in fact it exceeds the amplitude for the smaller distance $s/d = 5$ for certain frequencies --- a behavior difficult to model analytically. The cross and rocking interaction factors are predictably small, and could be neglected for all but the closest piles ($s < 5d$).

Closed-form expressions were fitted to the above interaction functions. For instance, for the horizontal lateral motion :

$$A_x = A_h\,(0^o) \approx \frac{3}{4}\left(\frac{s}{d}\right)^{-0.70} \exp\left(\frac{-\left(i + \beta_S\right)\ 3\pi\left(f - 1\right) s}{V_{L\alpha}}\right) \qquad (1)$$

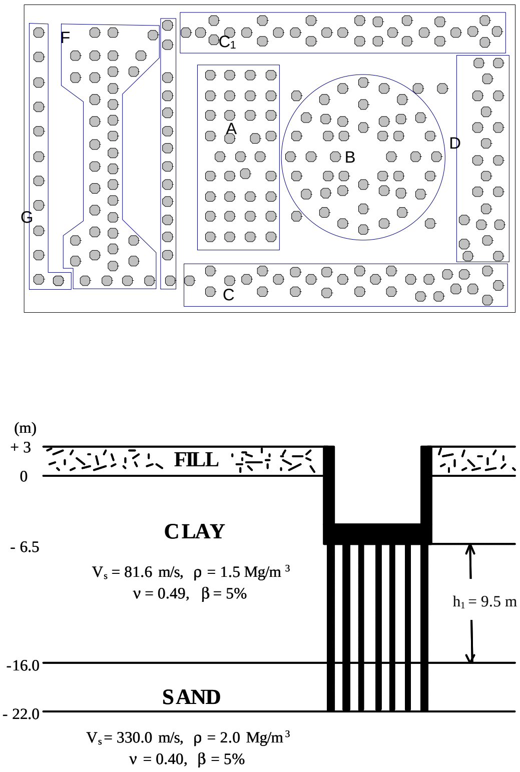

Figure 1. Plan and cross-section of the foundation (not in scale) and the soil properties used. Also shown in the plan is the division of the pile group in eight subgroups (A, B, C, C_1, D, E, F, G).

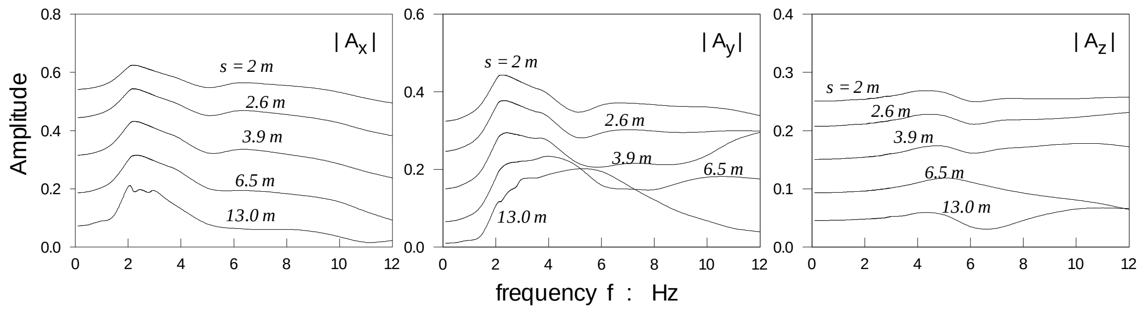

Figure 2. Amplitudes of interaction factors A_x , A_y , A_z for various pile distances, as a function of frequency.

One of the main issues facing the analyst trying to utilize of these results is the treatment of the interaction between fairly distant piles (say spacings s = 15d – 30d). There is a justified suspicion that the elastic wave-propagation solutions produce interaction factors which approach zero only asymptotically --- i.e. too slow, thereby overestimating (even if slightly) the real but unknown interaction factors.

Potential effects that may cause a smaller actual pile-to-pile interaction than the one computed by either the rigorous or the simplified solutions include : *(i)* the nonlinear soil response (near the pile cap); *(ii)* the unavoidable variability in the geometric and material parameters. [Recent research on this subject has shown that, indeed, both nonlinearity in the soil and randomness in material and geometric parameters tends to reduce the strong interaction produced by linear deterministic solutions.] ; *(iii)* the presence of the embedded foundation, the vertical sidewalls of which transmit an appreciable part of the lateral load, and which would thereby unavoidably interact with the pile group.

2.3 Pile Group Impedances

It could be intuitively believed that reasonably good estimates of the overall impedances of the complete 230-pile group would be obtained by simply adding algebraically the impedances of the eight individual subgroups, A through G. Such an addition would have been valid if the interaction among piles of different subgroups were negligible. It turns out that this is not the case, since the piles are placed extremely closely to one another (minimum spacing s = 1.54 d). In fact, the differences are functions of frequency. For group A+E the differences at low frequencies vary from about 15% for the lateral mode to 38% for the vertical mode stiffness. For group E+F+G these low-frequency differences exceed 60% for all vibration modes. In all cases the summation of the individual stiffnesses, $K_A + K_E$ or $K_E+K_F+K_G$ is larger than the stiffness of the whole group, K_{A+E} or K_{E+F+G}, i.e.

$$K_A+K_E > K_{A+E}$$
$$K_E+K_G+K_F > K_{E+G+F}$$

since the additional interactions between piles across the individual subgroup, tend to reduce the group stiffnesses.

The dynamic stiffness and damping for the whole pile group were obtained with the superposition method (using the curve-fitted interaction factors). Two types of analysis were performed : one considering the interaction between all pile pairs, and one imposing a cutoff pile-separation distance equal to 15d, beyond which all interaction factors were set equal to zero. It was found that imposing the s_{cutoff} leads to an appreciable change in the translational stiffnesses (but understandably small differences in the rocking impedances). Such large differences stem from the fact that there is a huge number of piles spaced at s > 15 d, so that accounting or not for their interaction makes a big difference.

Fig. 3 is aimed at convincing the unsuspected reader of the truth in the above statement regarding the number of piles, by displaying the histogram of the number of pile pairs having s/d falling within a certain range. It is seen that there are about 18,600 pile pairs within 15 < s/d < 25 alone! The total number of pile pairs exceeding 15d is about 35,000, nearly $^2/_3$ of the total number of pile pairs in the facility! That's why even a small mistake in computing the large-distance interaction factors has a substantial effect on group impedance. Hence, the role of even a moderate soil nonlinearity in reducing the interaction factors could be significant.

2.4 Contribution of Sidewalls

In this case, the base of the foundation mat cannot transmit any appreciable action to the soil. Because, in all likelihood, there is no *force-transmitting* contact between soil and foundation mat : the piles (being nearly end-bearing) are much stiffer vertically than the surrounding soft clay. It is certain that over the years this clayey layer has consolidated and separated from the mat. Thus, only the sidewalls of the facility offer additional stiffnesses and radiation damping to the foundation. Theoretical solutions to estimate the contributions of sidewalls, based on elasticity or Winkler models, have been developed and published, but note that these methods cannot easily account for any interaction between the deep/rigid sidewalls and the large pile group.

2.5 Foundation Input Motion (FIM)

Detailed free-field response analyses have been performed using the strain-compatible equivalent-linear iterative procedure. Furthermore, analyses for the kinematic response of the pile group were performed. These analyses have shown that the engineering approximation of neglecting modifications in the "effective input motion" that would

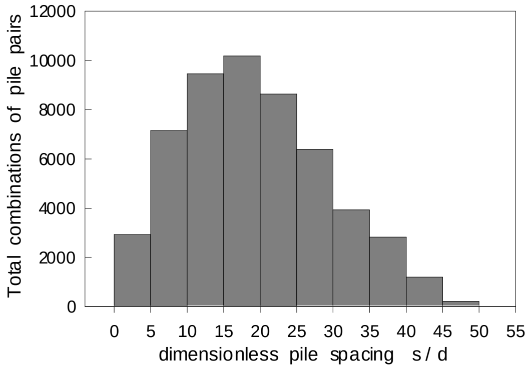

Figure 3. Histogram of the number of pile pairs with s/d falling within a certain range.

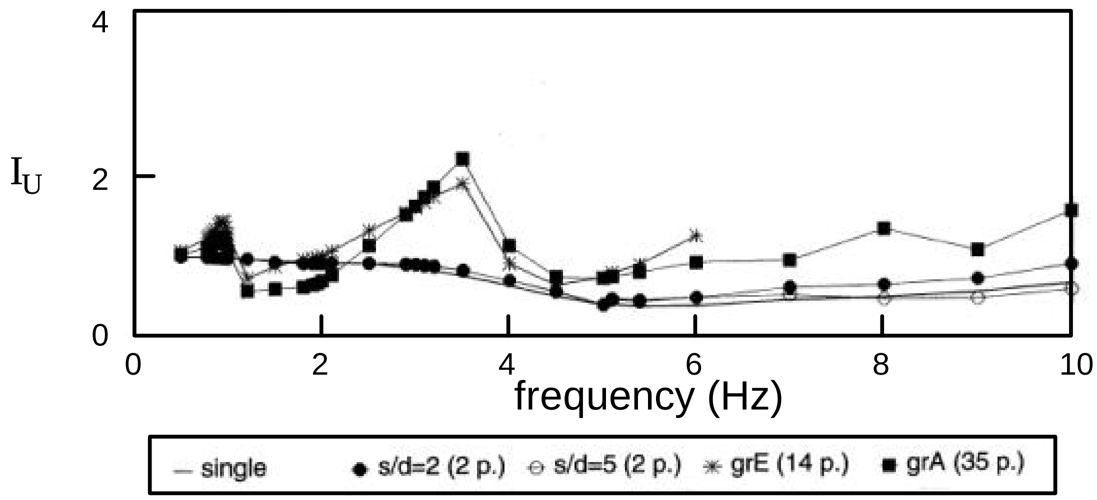

Figure 4. The kinematic interaction factor

arise from the presence of the pile group may be justified in a new design, where a healthy dose of conservatism is necessary. Such a conservative approximation was adopted in our analysis, but was hotly debated among the various parties involved. Because the uncertainty involved in this approximation is large (and crucial), given the difficulty in predicting the pile-to-pile interaction in such a huge group of closely-spaced piles . To give an idea of this uncertainty we plot in **Fig. 4** the *kinematic response* functions obtained for a single pile, a two-pile group with different spacings, group E (with 14 piles), and group A (with 35 piles). It is clear from the figure that for the natural frequencies of the flexibly-supported containment building ($f_1 \cong 1.10$ Hz, $f_2 \approx 3.5$ Hz), I_u could be taken anywhere between 0.50 and 2.0 --- a very wide range with significant repercussions on the fate of the piles : with $I_u > 1$ many piles would fail, while with $I_u = 0.5$ all piles would survive. And on top of all this, one has to think of the role to be actually played by the embedded wall, which has been neglected in this figure.

2.6 Internal Forces due to the Inertial Loads of the Superstructure

A series of dynamic analyses were performed to obtain floor response spectra at the various levels of the major critical facility. In those analyses, an artificial earthquake time history was used as base excitation of the elastically supported structure. Two different supporting springs and dashpots were considered, corresponding to the two sets of impedance curves, *with* and *without* cutoff frequency, respectively. The results of that study also showed that the fundamental period of the structure-foundation system is about 1 second, and that the influence of the higher modes is negligible. Time histories of total shear force and overturning moment transmitted onto the foundation were obtained for each case for both directions, x and y.

Fig. 5 displays for the 230 piles their share of the total shear force at the fundamental frequency of the system. In the sequel, the response of two particular piles is examined :

- Pile 227 represents the perimeter piles that carry a very large part of the load (about 1% of the total shear, compared to the average of 0.43%)
- Pile 40 represents the central piles that carry a very small part of the load about 0.2% of the total shear).

For piles 227 and 40 the variation with frequency of the shear pile-head forces is presented in **Fig. 6 (a)**. The change with frequency of the load taken by the two piles is thus clearly seen. For the seismic excitation, the time histories of shear forces Q at the head of these two piles are portrayed in **Fig. 6 (b)**. In addition to the significant differences in amplitudes, it is important to notice the differences in the frequency content of the forces in pile 227 versus pile 40.

Results for the bending moments of a pile, due to vertically propagating shear waves that are consistent with the base input acceleration history, are summarized in **Fig. 7** ; they are in the form of distribution with depth of the peak (in the time domain) values of moment, and the largest (in the frequency domain) values of the moment-to-acceleration transfer functions. The latter occur at or about, the fundamental frequency of the soil deposit.

It is worth noting that the largest peak values occur at the head of the pile (due to the restriction of rotation by the rigid base slab) and at the interface between the two soil layers, at a depth of 9.50 m (stemming from the great difference in soil stiffness). On the basis of these results, it was concluded that some of the piles would develop plastic hinges during the design earthquake.

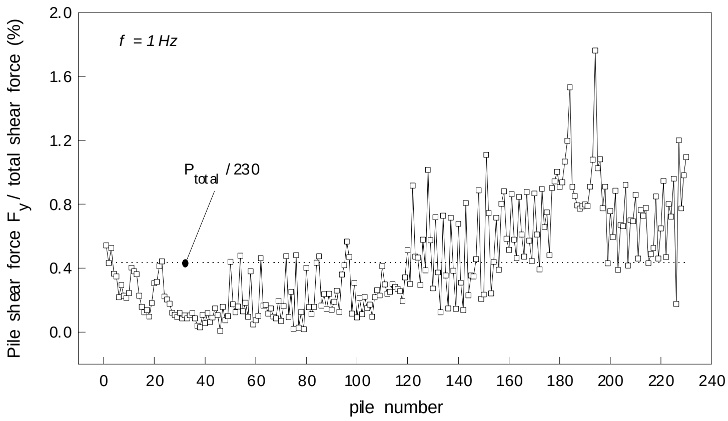

Figure 5. Distribution of the total shear foundation force on each pile at frequency f = 1 Hz.

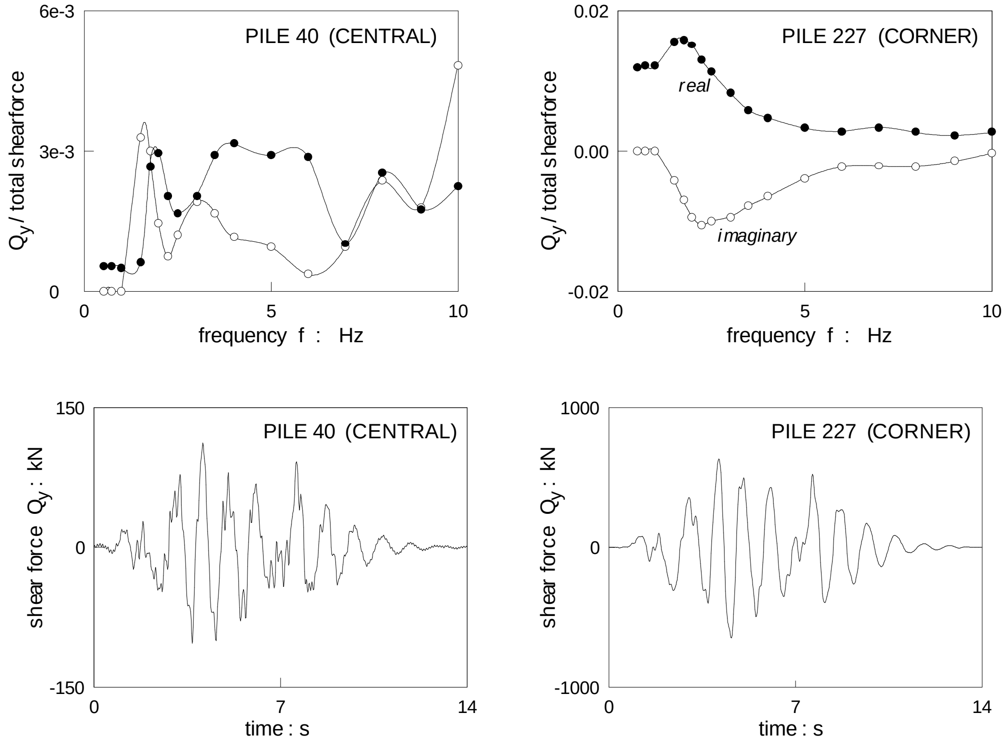

Figure 6. (a) Frequency domain and (b) Time domain variation of pile-head shear force in piles 40 and 227.

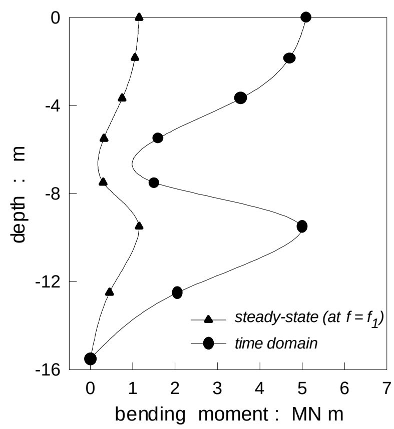

Figure 7. Envelopes of the peak pile kinematic bending moments along a pile with depth, in frequency and time domains.

2.7 The Crucial Unresolved SSI Issues

It must have become clear from the above presentation that a number of issues arise in solving problems similar to the one presented here, under strong seismic excitation. They include :

- the treatment of pile-to-pile interaction in a group of very large number of closely spaced piles
- the assessment of the effect of soil nonlinearity and lateral soil heterogeneity on the interaction factors, and on the relevance of superposition for obtaining the overall foundation impedance
- the assessment of the kinematic effects under the above conditions and the reliable estimation of the "Foundation Input Motion"
- the role of the deeply embedded foundation : its interaction with the piles, and its effect on the kinematic response
- the consequences of structural yielding of a number of piles in a large pile group.

Reliable answer to all these questions cannot be easily obtained within the current state of the art.

3. UPLIFTING and SLIDING of SURFACE FOUNDATIONS UNDER STRONG EXCITATION (The Case of Rion-Antirrion Bridge Foundation, and the Overturning in Adapazari)

Shallow mat foundations were and are often chosen for buildings and bridges on stiff and strong soils. In some critical offshore facilities, surface foundation is sometimes an unavoidable solution even on very poor soils. And in many parts of the world, in regions with soft soils and high water table, shallow mat foundations may be the only economically feasible solution.

In many of these cases, "modern-day" strong seismic shaking (i.e., with PGA values in excess of 0.40 g and spectral values S_a more than 1 g over a wide period range), would tend to cause partial cyclic uplifting and sliding of the foundation. The consequences of such phenomena on the response of the foundation and the superstructure can be significant. Bearing capacity failures or excessive foundation tilting and sliding may ensue. Evaluation of foundation behavior under strong excitation leading to such *"near-failure"* conditions has only recently been receiving research attention. This is in contrast with the huge worldwide research effort in developing methods of SSI analysis for shallow, embedded, and deep foundations responding in the linear or moderately nonlinear range, during the last three decades.

The present chapter outlines the relevance of the above nonlinear phenomena on an active critical project, and summarizes observations from Adapazari during the 1999 Izmit Earthquake.

3.1 The Foundation of the Rion-Antirrion Piers

This major 2.2 km long bridge is now at the height of its pier construction in western Greece, linking Peloponnesus to the Mainland. For nearly ten years, a number of international competitions had taken place, and two basic solutions had emerged : a suspension bridge (1300 m main span with two pylons-piers of total height of 180 m) and a four-pylon cable-stayed bridge (560 m each main span, total pylon-pier height of 230 m). The bridge being constructed is the latter. Despite many differences, one thing the two types of bridges would have in common : the pier-foundation structure. Regardless of the span of the piers and the nature of the super-structure, a 90 m diameter surface foundation was necessary, supporting a heavy 80 m high pier.

Several factors rendered the solution of a surface foundation an almost mandatory choice, and contributed to the need for unusually large (even by suspension-bridge standards) piers : the large water depth (nearly 60 m, almost over the whole length of the straits) ; the design requirement for the pier to withstand a full-speed collision with a tanker ; the poor soil conditions (relatively-loose soil layers down to at least 50 m depth, and bedrock located at depths perhaps exceeding 500 m) ; and finally, the very strong seismic design ground shaking (PGA $\approx$ 0.50 g, maxSa ≈ 1.25 g for T ≤ 1 sec *). [Recall that the much bigger Akashi-Kaykio bridge (span of 1991 m) has two piers of 80 m diameter (slightly embedded, however).]* The tributary weight of the superstructure was in all cases only a small fraction of the dead weight of the pier. Thus, the seismic oscillation of the pier itself and the response of the supporting soil were the factors controlling the design of pier-foundation-soil system.

Several extensive geotechnical exploration programs were carried out over a period of years, administered by the Public Works Laboratory of Greece (1987, 1992), and by the bridge contractor, GEFYRA SA (1996). The program involved at least 10 of the most reputable geotechnical laboratories in Europe and comprised state-of-the-art insitu and laboratory testing. Extensive summaries of the findings can be found in GCG (1992) and Pecker & Teyssandier (1998). Some general trends are as follows :

(a) To a depth of at least 80 m the soil profile comprises layers of mixed cohesive and cohensioless soils, with significant interbedding at both the macroscale and microscale.

(b) Cohesionless units have substantial gravel and fines content ; they are of moderate density, which only occasionally becomes high.

(c) Only relatively thin zones appear to be potentially liquefiable ; shear stresses transmitted into these zones seem to be limited by the transient strength of weaker underlain silty clay or clayey silt layers.

(d) At mudline, a sandy gravel layer of medium to low density exists; its thickness varies for the sites of the four piers, from 5 to 20 meters.

(e) Cohesive units are predominantly silts or clayey silts of low plasticity ; carbonate content is high but only acts as a cement below 35 to 40 m beneath sea bed. Where evidence of stress history was retained, OCRs of 1.5-2.0 are inferred with higher values in previous drying or wave compacted crusts.

(f) Shear wave velocity values (obtained mostly from in-situ seismocone measurements, and also from resonant column tests and bender-element triaxial tests) are compiled in the attached **Fig. 8.**

(g) No borehole or in-situ test extended deeper than 100 m depth ; this was judged adequate in view of the rapidly increasingly stiffness and strength below 50 m depth. Bedrock was not encountered, and geologic estimates indicate that its depth may be of the order of several hundred meters.

3.2 Seismic Analysis of Pier

The small coupling between the oscillations of the pier and of the pylon-cables-deck of a suspension or cable-stayed bridge would require a model such as that sketched in **Fig. 9** for a suspension bridge. Neglecting such coupling altogether would provide acceptable accuracy for design calculations. On the other hand the *"added" hydrodynamic masses* along the underwater part of the pier are very significant.

Dynamic impedances in swaying and rocking were obtained with the SASSI finite-element code, assuming full contact between the footing base and the soil, and treating nonlinearity in a very crude 1-D equivalent-linear way. Large overturning moments and shear forces (of the order of 40,000 MNm and 1,200 MN, respectively) are obtained from such analyses. Comparing with the vertical (normal) effective force transmitted to the base, of about 1000 MN, it becomes clear that significant uplifting and some sliding would occur. Simplified analyses of the effect of possible sliding at the interface (using a lumped-parameter model) showed that this would imply only a minor permanent displacement, along with a beneficial reduction in the developing pier acceleration and the resulting overturning moment (Younis & Gazetas 1996). The danger, however, with such sliding of a huge footing on not strong enough soil would be to take place not at the interface, but rather through the soil --- a bearing-capacity type failure, the deformational consequences of which, even if of only limited extent, would be unacceptable. To preclude this possibility the geotechnical designer of the cable-stayed bridge (Pecker, 1998) developed an innovative *soil "nailing"* technique to improve the uppermost 20 m of natural soil (**Fig. 10**).

Partial uplifting during the design earthquake was both unavoidable and desirable, as it reduced the vibration amplitudes, and hence the developing shear force and overturning moment. Properly modeling the footing-soil

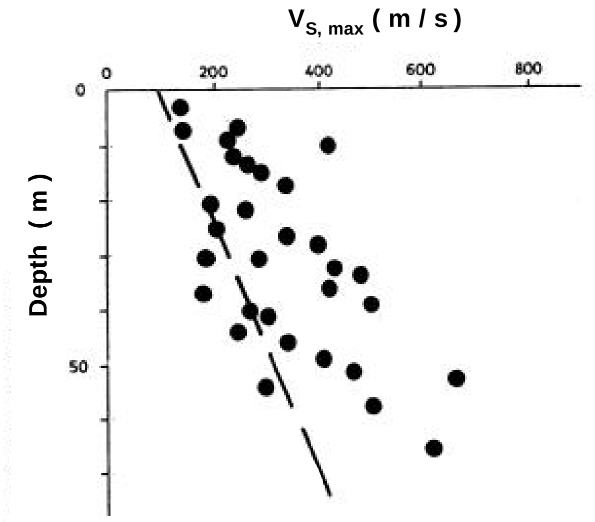

Figure 8. Compilation of shear wave velocity measurements under the piers.

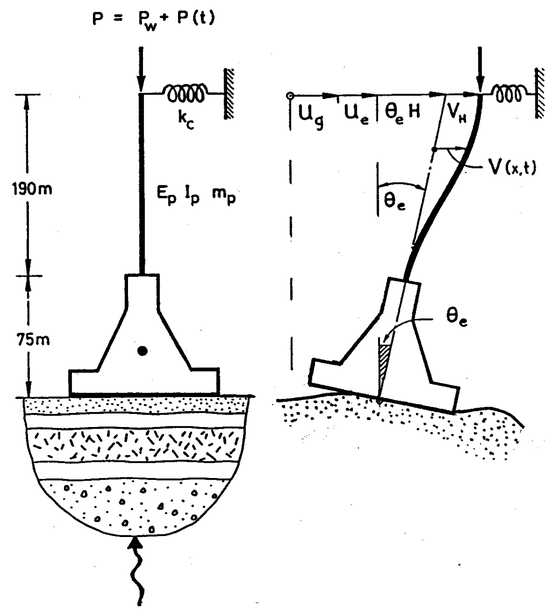

Figure 9. Sketch illustrating the SSI analysis for a pier of a hypothetical suspension bridge (Gazetas, 1988). Its analysis is similar to the one conducted for the piers of the cable-stayed bridge that is presently being constructed.

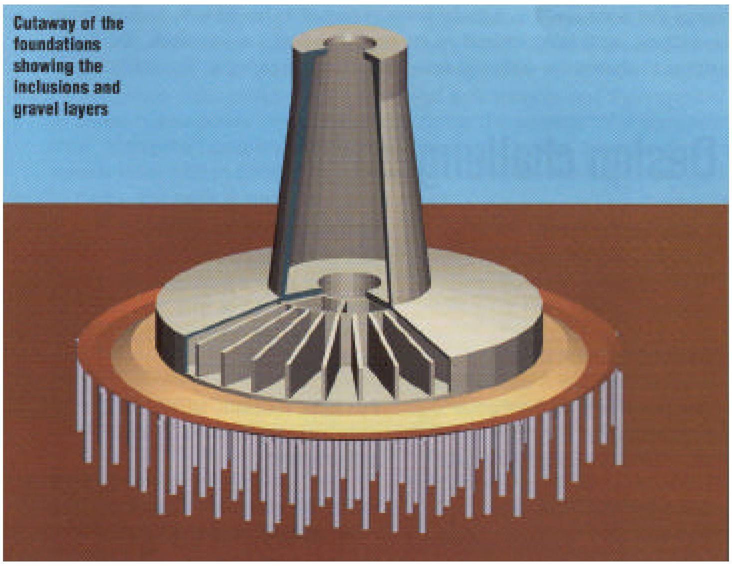

Figure 10. The pier foundation and the soil "nailing" concept of the under construction bridge (Pecker, 1998)

interaction to account for partial time-dependent uplifting, is not an easy task in a nonlinear soil "environment". Phenomena occurring in such a case include :

- reduced (but time-and-frequency dependent) foundation stiffness, especially in rocking
- reduced radiation damping in the uplifting phase and increased damping in the "*slapdown*" phase
- approaching or even reaching the ultimate capacity of the foundation, with the development of a shallow yield surface.

Possible ways of solving this problem, as described by Pecker (1998), include :

- Development of a global finite-element model including both the soil and the structure. A realistic nonlinear constitutive soil model must be used. Owing to this constraint, to computer limitations and also to the fact that development of a global model requires state-of-knowledge expertise in a number of fields related to SSI, such an approach is seldom used. In addition, it is not well-suited for the development of design which requires that various alternatives be tested before achieving a final design.
- Development of an uncoupled procedure which involves the following steps :
  (i) The static global moment-rotation relation of the foundation is obtained for monotonic loading. This can be accomplished rigorously with a 2-D finite-element multi-yield-surface plasticity modeling of the soil ; less rigorously with a simpler elastoplastic model ; and even simpler by empirically combining the foundation-soil elastic stiffness with the foundation-soil ultimate moment capacity from limit analysis.
  (ii) The cyclic behavior curve is obtained using improved Masing-type or Iwan-type or Bouc-Wen-type rules for unloading and reloading.

In the final design of the pier, both approaches were followed by the designers (Pecker & Teyssandier, 1998). They also implemented a global fully-coupled dynamic model. This model was subjected simultaneously to a vertical and horizontal motion specified at a hypothetical rock outcrop. A nonlinear constitutive model was used for the in-situ soil layers and for the reinforced soil block. Special contact elements connected the mat and the soil ; they could not transmit tensile forces, thereby allowing separation between soil and foundation. Transmitting boundaries provided at the bottom of the mesh simulated the underlying halfspace and prevent spurious wave reflections.

As a result, an ultimate moment capacity of about 30,000 MNm was computed. The existence of this limit reduced the developing acceleration levels, and thereby the transmitted shear force to about 800 MN. All this would imply a limited dynamic and residual horizontal and tilting deformations for the pier. The *soil "nailing"*, with steel pipe inclusions of 2 m in diameter, 20 m in length, and 5 m in spacing, have been shown to ensure that such deformations would occur while the soil retains its integrity.

3.3 Tilting and Overturning of Buildings in Adapazari, 1999 Izmit Earthquake

An unprecedented opportunity to see an actual manifestation of some of the issues raised in the above discussion of the design of the Rion-Antirrion pier foundation has been provided by the M_S 7.5 Izmit (Kocaeli) 17-August-1999 Earthquake.

One of the most significant aspects of this earthquake was the extensive foundation–related failures observed in an area covering several city blocks near the center of Adapazari. Such failures led to settlement, tilting, and complete overturning (toppling) of buildings, which otherwise retained their structural integrity (see photo in **Fig. 11**). Liquefaction of shallow soil layers was evident on the ground surface, but not in abundance, with few sand boils observed in the free field. Significant permanent tilting and overturning was only observed in relatively ***slender*** and *laterally free* buildings, while wide and/or contiguous buildings experienced very small (and usually only vertical) displacements.

Soil profiles, based on SPT and CPT data in a characteristic location where two buildings overturned, reveal the presence of a number of alternating sandy-silt and silty-clay layers, from the surface down to a depth of at least 15 m. From the empirically–interpreted CPT (q_c, f_s) results [Robertson & Wride, 1997], the distribution with depth of fines content (FC) was found to fluctuate from less 10% to more than 60%, with abrupt changes occurring every 30 cm or less.. The q_c values also fluctuate between about 0.40 and 5 MPa.

The peak ground surface acceleration (PGA) is not known, but preliminary (1-D) wave propagation analyses suggest that it might have been as low as 0.15g – 0.20 g , with several significant cycles of motion. In fact, indirect comparative evidence shows that PGAs in the studied region could not have been much higher than the above values. *[By contrast the neighboring districts which were structurally devastated but with no sign of foundation-soil distress, accelerations must have been higher.]* With such levels of acceleration and the aforementioned CPT stratigraphy liquefaction is computed to have been of rather limited extent. Sandy-silt layers of total thickness of ≈ 1m located between 2.0 and 4.0 meters depth must have liquefied. The small amount of water expelled by such a small–thickness layer, covered by about 2 m of clayey–silty fill, barely reached the surface *[recall Professor Ishihara's liquefaction-appearance charts]* ; hence the scarcity of sand boils [see also Dobry 1989]. But its effect on the stability of the foundation was significant. Several other silty inter-layers at larger depths were also probably in the verge of liquefaction. As their total thickness was not more than 3-5 meters, and their stiffness was appreciable (indicated by $q_c \approx$ 4-5 MPa or $N_{SPT} \approx$ 15-20), liquefaction of these layers, even if it occurred, would not cause any dramatic effect on the surface in the free-field. But the developed porewater pressure, and the subsequent decline in soil resistance, could have exerted some small effect on stability of foundations.

The above pattern of liquefaction or strength reduction can explain at least qualitatively: **(a)** why many buildings "escaped" with only vertical settlements [≈ 0.50 m on the average]; **(b)** why several moderately slender buildings suffered substantial tilting; and **(c)** why the slender buildings overturned. It is noted that all these buildings were founded on mat foundation at –1m, just above the water table; but they differed substantially in foundation width and building height. What is of importance is the ratio of building height to foundation width: the "***aspect***" ratio. Specifically, it was observed that

Figure 11. Settlement and overturning of buildings in Adapazari.

- *buildings of aspect ratio $H/B < 1$ did not experience any visible tilting, even if they were free laterally*
- *buildings with aspect ratio of about H/B » 1.5 experienced tilting of about 5 degrees, and*
- *buildings with aspect ratio of $H/B > 2$ toppled, if of course they were free laterally.*

.

The interpretation is straightforward, although computationally difficult: disproportionately large overturning moments were generated in the very slender buildings; these would have led to rotational (bearing capacity) sliding failure, even if the building had experienced moderate levels of acceleration, of only about 0.15 g. For instance, with $H/B = 2$ and this structural acceleration only about ½ of the foundation base width will have retained its contact with the soil.

Such a rotational failure could not possibly have occurred in buildings either having small aspect ratios ($H/B \approx 1$), or being surrounded by other buildings. On the other hand, buildings with moderate aspect ratios ($H/B \approx 1.5$) experienced seismic overturning moments which were sufficient to cause partial uplifting of the foundation, but not so large as to topple the building.

4. CONCLUSION

Despite the enormous progress in understanding the phenomena and analysing the effects of SSI, several unresolved issues of practical significance are beginning to emerge. The paper has outlined two engineering projects, one involving a large number of piles constituting the foundation of an existing critical structure, and the second referring to the huge foundation for the piers of a major bridge designed to significantly uplift during a strong design earthquake. For the second case, a qualitative analogy has been drawn with the tilting and overturning of slender buildings in Adapazari, during the 1999 Izmit Earthquake. Future research will improve the state of knowledge on these topics.

5. REFERENCES

Dobry R. (1989). “Some basic aspects of liquefaction during earthquakes”. *Earthquake Hazards and the Design of Constructed Facilities in the Eastern US,* Annals of the N.Y. Academy of Sciences, pp. 172-182.

Gazetas G. (1988). “Analysis of SSI for the pier of a suspension bridge for Rion—Antirrion”. Report to Ministry of Public Works, Greece.

Gazetas G., Fan K., Tazoh T. & Shimizu K. (1993). “Seismic response of the pile foundation of Ohba Ohashi bridge”. *Proceedings of the 3rd International Conference on Case Histories in Geotechnical Engineering*, pp. 1803-1809.

Gazetas G., Hess P., Zinn R., Mylonakis G., & Nikolaou A. (1998). “Seismic response of a large pile group”. *Proceedings of the 11th European Conference on Earthquake Engineering*, Topic 3, Paris.

Geotechnical Consulting Group – GCG (1992). “Rion—Antirrion fixed link: additional offshore geotechnical investigation. Evaluation and interpretation of in-situ and special laboratory test programmes”. Report to Public Works Laboratory of Greece

Japan Society of Civil Engineers (1999). The 1999 Kocaeli Earthquake, Turkey – Investigation into the damage to civil engineering structures. Report of Earthquake Engineering Committee.

Pecker A. & Teyssandier J.P. (1998). “Seismic design for the foundations of the Rion—Antirrion Bridge”. *Journal of Geotechnical Engineering,* Vol. 131. pp. 4-11.

Pecker A. (1998). “Capacity design principles for shallow foundations in seismic areas”. *Proceedings of the 11th European Conference on Earthquake Engineering,* Topic 3, Paris, pp. 303-315.

Robertson P. K. & Wride C. E. (1997). “Evaluation of cyclic liquefaction potential based on the CPT”, *Seismic Behavior of Ground and Geotechnical Structures*, P. S. Seco e Pinto, Ed., A. A. Balkema, 269-279.

Tazoh T., Dewa K., Shimizu K., & Shimada M. (1984). “Observations of earthquake response behavior of foundation piles for road bridge”. *Proceedings of the 8th World Conference on Earthquake Engineering,* Vol. 3, pp. 577-584.

Tazoh T., Shimizu K., & Wakahara (1988). “Seismic observations and analysis of grouped piles”. Shimizu Tech. Res. Bull. No. 7, pp. 17-32.

Younis C.J. & Gazetas G. (1996). “Dynamic response of a tower-pier system on viscoelastic foundation with frictional interface”. *Engineering Structures,* Vol. 18, No. 7, pp. 546-557.