MODEL BRIDGE PIER-FOUNDATION-SOIL INTERACTION IMPLEMENTING IN-SITU / SHEAR STACK TESTING AND NUMERUCAL SIMULATION

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ABSTRACT:

This paper presents results of the measured and predicted response of a bridge pier model structure which has been erected at the Volvi-Greece European Test Site. After an initial laboratory testing of the bridge pier model under cyclic horizontal loads and the study of its cyclic post-elastic behaviour, a series of low-to medium intensity excitations were performed at the test site for a period of two years. The deck acceleration response was recorded and was studied in the frequency domain in order to extract the most significant eigen-modes and eigen-frequencies for the various configurations of this pier bridge model. Moreover, an extensive numerical simulation of the response was also performed, including the flexibility of the foundation. The numerical simulation was also extended to include a volume of soil under the foundation in order to study the soil response when the pier was subjected to low intensity man-made excitations. Four pressure cells were placed in the soil under the foundation and measurements were obtained from these pressure sensors during the man made excitations, which were then correlated to the numerical predictions. A summary of the in-situ measurements of the bridge pier model response are presented and compared with the corresponding numerical predictions from a variety of numerical simulations that attempt either in a relatively simple or a relatively complex way to address the influence on the response that arises from the flexible foundation conditions. A similar testing arrangement, but on a smaller scale, was investigated at the Laboratory through a small-scale model pier. This testing arrangement also includes the flexibility of the soil with the implementation of a sand box with a shear stack arrangement. Accelerations are recorded both on the pier as well as within the soil body. Also in this case, numerical analysis is complementary utilized and the dynamic response of the bridge pier model – sand box system is simulated. Both the eigen-frequencies and the seismic response of the system in the time domain, that are derived numerically, are found to be in good agreement with the measured results.

KEYWORDS: Bridge Pier, Model, Soil-Foundation Interaction

1. INTRODUCTION

Although the effect of soil-structure interaction on the dynamic response of typical residential or commercial structures and infrastructure (i.e. bridges, Kawashima 2000) has long attracted scientific attention, it is widely recognized that there is an urgent need for further experimental support and validation. This need is far more crucial in cases where the structure responds in-elastically and/or the soil conditions favour the appearance of SSI phenomena. Towards this objective significant effort has been undertaken within the context of a number of projects that has been continuously funded by the European Union for the last decade [Manos 2004, Pitilakis 1999, http://euroseis.civil.auth.gr]. These projects were carried out mainly at a large "natural" laboratory (Volvi Euro-Test Site, located 30 km from Thessaloniki-Greece), which is unique in Europe and one of the few such Test Sites worldwide [Manos 2004]. The main objectives of this paper are to: a) define soil flexibility and damping properties and use model structures in-situ and in the laboratory (shear stack) to investigate the

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beneficial or detrimental role that the soil-foundation flexibility (SSI) has on the overall dynamic response b) introduce structural yielding on the model structures and, possible, investigate the coupling between the structural yielding and the SSI effects c) examine the nature and the effect of the waves transmitted by the oscillation of the superstructure to the foundation level and the surrounding soil and d) use the Aristotle University Laboratory facilities to verify post-elastic behavior of model bridge piers the effectiveness of repair techniques and utilize the shaking table and the shear stack apparatus to possible reproduce certain response features observed in-situ. e) use the in-situ and laboratory measurements to validate empirical, analytical or numerical simulations of this soil-foundation-structural flexibility and damping on the dynamic and seismic structural response. The in-situ model has the advantage of realistic foundation conditions, which are present for this model structure that is supported on the soft soil deposits. The shear stack model arrangement in the laboratory In fact, the structure is susceptible to SSI effects according to Eurocode 8 criteria since the corresponding shear wave velocity (Vs) at the surface is approximately 100m/sec. The current extension of the in-situ facility has made it possible to subject the model structures to low-to-medium intensity man-made excitations.

1.1. Model structure at the Volvi – European Test Facility

The model structures built at the Volvi Test Site (Figure 1) is a single bridge pier and its foundation block with an overall geometry and mass distribution depicted in figures 2. The total mass is 185.3KN, 95.3KN of which are concentrated at the deck level. This bridge pier model structure was built during March 2004 at the Euro-Test Site and can be considered as a reduced scaled model of a number of corresponding bridge pier specimens that were tested at ELSA laboratories of the European Joint Research Center [Pinto, 1996], but of smaller dimensions and a different cross-section detailing . Because of these differences no comparison of the observed behavior between the Volvi and the ELSA pier specimens is attempted here. More information regarding the material parameters, the reinforcement distribution and the testing of identical pier models at the laboratory are described in Manos et al. (2004 and 2006a) and are not repeated here due to space limitation.

Figure 1: Overview of the Test Site facilities

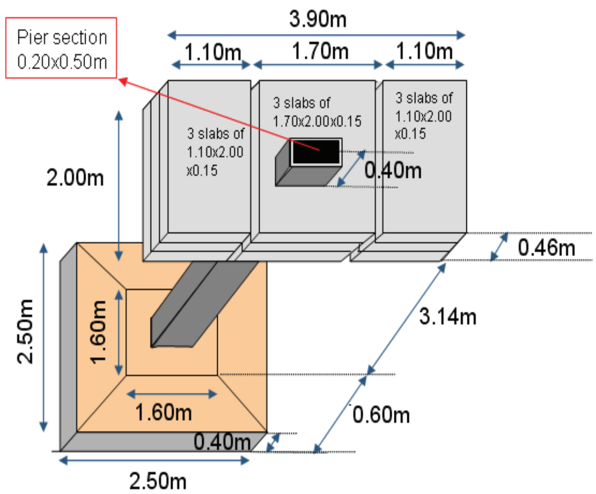


Figure 2. Basic geometry of the pier model

2. EXPERIMENTAL RESULTS

2.1 A typical testing sequence of low - intensity pull-out tests.

A typical testing sequence includes relatively low-intensity free vibration tests of the pier model. This is accomplished by introducing a controlled force on the deck of the bridge pier model thus displacing it from its original equilibrium condition. The sudden release of this applied force caused the free vibration of the model structure which was recorded by the sensors. The force was applied at the deck either coinciding with the in-plane axis (the strong direction of the pier cross-section depicted in figure 4) or with the out-of-plane axis (the weak direction of the pier cross-section). The amplitude of this force did not exceed 2.2KN in the in-plane direction and 1.4KN in the out-of-plane direction. In Figure 3a the measured in-plane deck response from a low-intensity pull-out test is depicted (D being the damping ratio and F the dominant response frequency). The full set of the measured response [Manos 2005] includes measurements for various instruments and their components (i.e. displacements, accelerations, soil pressures), direction of excitation (in-plane, out-of-plane) and configurations of the structure (inclusive or not of the cables and the additional mass). The measured horizontal deck acceleration response is listed in Table 1 together with the dominant frequency of this response, as found from analyzing the measured signals in the frequency domain.

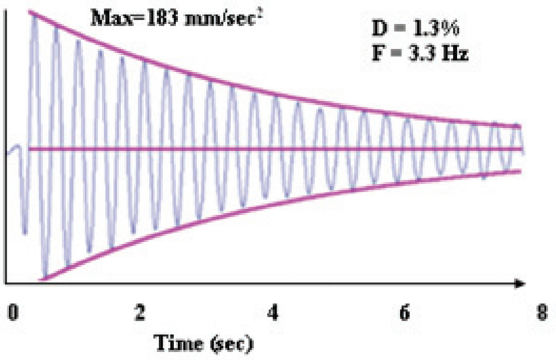


Figure 3a. low intensity pull-out. In-plane acceleration deck response est.

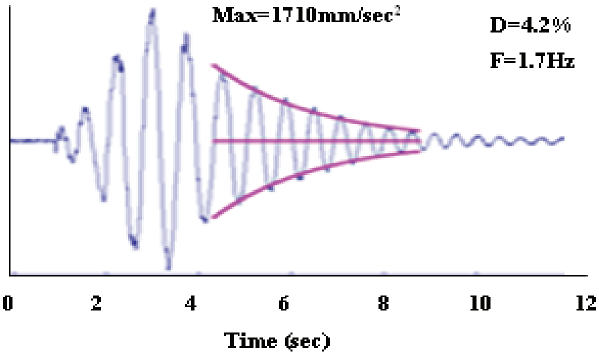
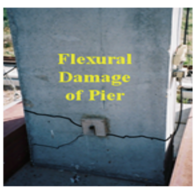


Figure 3b. low-to-medium intensity forced vibration test that produced the pier damage. In-plane acceleration deck response

Table 1: Summary of Measured Response		Pull- Out Tests x-x and y-y, 6th April 2004, Structure with no Extra Mass and Cables		
		From FFT	Peak deck acceleration in g	
Channel No.	Response	Frequency Hz	Max	Min
11 in-plane	Deck Accel.	3.29	0.01826	-0.01745
12 out-of-plane	Deck Accel.	1.83	0.01389	-0.01493

Table 2. Summary of Measured Response

INITIALLY	With cables and struts	Without cables or struts
Out-of-plane	1.929 Hz	-
In-plane	2.800 Hz	2.600 Hz
Torsional	-	-



AFTER CRACKING	With cables and struts	Without cables or struts
Out-of-plane	1.709 Hz	1.099 Hz
In-plane	2.539 Hz	2.344 Hz
Torsional	2.783 Hz	-

2.2. Low-to-medium intensity testing sequence.

A series of low-to-medium intensity forced vibration tests were performed which produced non-linear response of the pier. The frequency of excitation for these tests was varied in the range 1.5Hz to 2.0Hz. During this test the diagonal cables and struts were active. Figure 3b depicts the deck acceleration response during the 2nd low-to medium intensity test that produced the Pier damage. By comparing this response with the corresponding response of the deck during the low intensity test (Figure 3a) the severity of the forced vibration test can be seen in terms of deck horizontal acceleration. Listed in Table 2 is the variation of the measured eigen-frequencies before and after the development of the damage at the pier base.

3. NUMERICAL PREDICTIONS

3.1. Overview of the alternative FE approaches

Different finite element (FE) numerical simulations of the bridge-pier model were constructed within the framework of the numerical analysis aiming to provide an ascending level of modeling complexity and an optimum balance between model simplicity and accuracy. Simple and more advanced spring/damper models were investigated together with finite element approaches and a coupled Boundary Element / Finite Element Method (BEM/FEM) formulation and their relative advantages were assessed. In particular, the following FE models were used for the simulation of the static/dynamic and linear/non-linear behavior of the bridge pier:

- A simple frame-type model with appropriate mass distribution and flexible support, which has the potential capability of simulating the development of a plastic hinge by the appropriate coupling of plastic rotations with soil flexibility.
- A 3-D FE model with equivalent cube-type foundation supported on springs of non-uniform properties with the use of LUSAS code.
- A complete 3-D model with concrete cracking/crashing capabilities supported on geometrically non-linear (compression only) springs with the use of the FE code ANSYS.
- A complete linear elastic 3-D model with a detailed representation of the additional C220 connecting steel beams as well as of the cables that attach the deck to the foundation with the use of the FE code ANSYS.

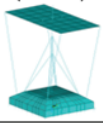
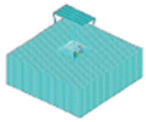
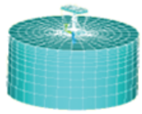
- Two alternative 3-D models inclusive of the surrounding soil as modeled with solid elements (Figure 4)
  - A 3D FE model (Figure 5) accounting for both pier uplift at the soil-foundation interface (i.e. through uni-lateral springs resisting only compression forces at the soil-foundation interface) and material non-linearity for the soil (using Von-Mises material law for the soil solid elements).
  - A 3-D pier model supported on elastic foundation that is simulated using the Cone theory developed by Wolf.
  - A complete 3-D far field-soil-pier model preformed within the framework of a comprehensive FEM/BEM approach [Manos et al., 2005a].

Currently, the optimum approximation (in terms of model complexity and degree of accuracy) of the above seven alternative FE numerical approaches is judged only on the agreement between measured and predicted pier response in terms of a) eigen-frequencies and eigen-modes and b) maximum amplitude of the low-intensity displacement and acceleration deck response. Table 3, summarizes the calculated natural frequencies (herein only the first two that are of interest are presented) together with the measured values. The influence of the foundation flexibility is further evaluated, utilizing the pressure cell measurements, which are not presented here due to space limitation. Moreover, the non-linear response of the bridge pier model in the presence of SSI effects in-situ must be further studied. There was a preliminary calibration of the various numerical simulations (Table 3), related to the properties of the cables, concrete and soil, based on the measured values. However, this calibration was used for all types of numerical simulations that were tried. In this way, the complexity of the various simulations and the resulting accuracy is not influenced by this calibration. When the accuracy of the obtained numerical results is judged on the basis of the complexity of the numerical simulations it can be concluded that the more complex models did not necessarily resulted in more accurate results. The increasing numerical complexity was mainly due to the soil modeling; thus, it can be concluded that the refinement in soil modeling did not necessarily resulted in more accurate predictions. This must be attributed to the fact that the less complex models with Winkler-type foundation could be easily calibrated whereas for the more complex soil modeling this was not the case, as it represented a multi-parametric problem (choice of soil volume and number of layers and their properties, types of boundary conditions).

In simulating the pier response during the low-to-medium as well as during the low-intensity tests after cracking, the complex soil modeling (Figure 4) was used in order to:

- extend the investigation towards the non-linear response of the pier without compromising the soil modeling refinement by using a spring-type soil representation.
- be able to extract the response values at the volume of the soil around and under the pier foundation (e.g. pressure cells), when needed.
- be able to extend this type of analysis, by accounting for the non-linear response of the soil itself.

Table 3: Numerical dynamic characteristics of the pier for the two translational vibration modes

In plane	Measured 	LUCAS 	ANSYS (Winkler) 	ANSYS (3D soil) 	ANSYS (3D soil)- 	Wolf	ANSYS (Fixed) 	Theoretical Solution (Fixed)
No extra Mass With cables	3.290	3.315	3.244	3.301	3.246	3.202	3.445	
Extra Mass Without cables	2.600	2.681	2.611		2.611	2.583	2.762	2.890
Extra Mass With cables	2.800	2.815	2.817	2.954	2.818	2.778	3.007	

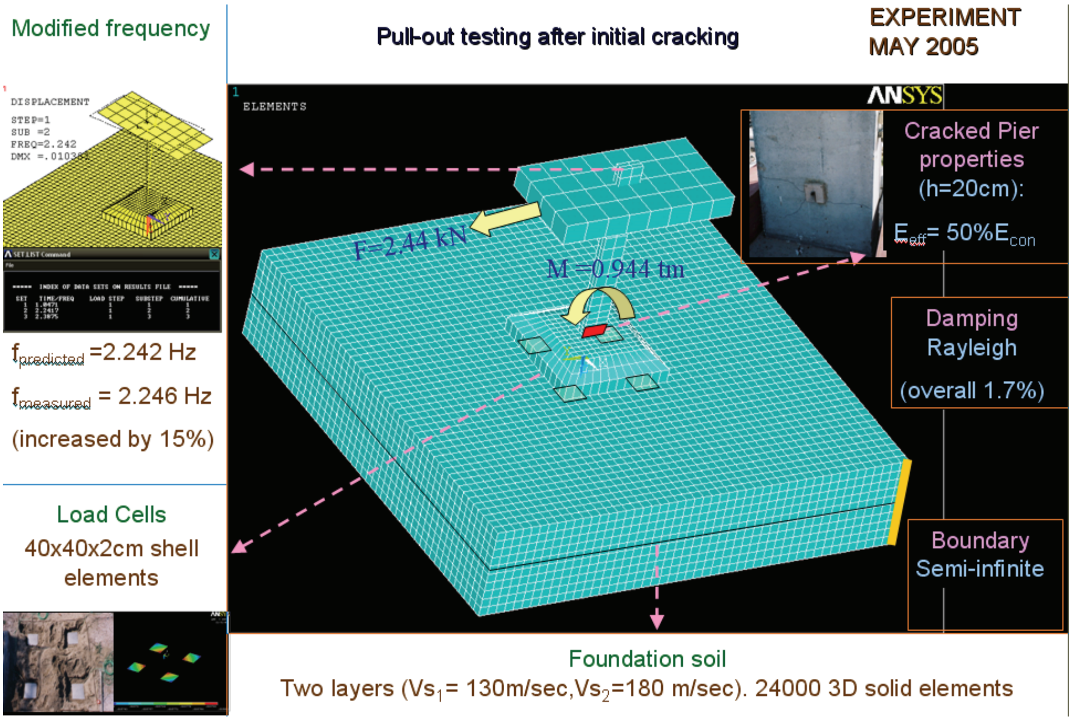


Figure 4: Overview of the FE model for the case of the post-cracking pull-out tests

3.2. Numerical simulation of low-intensity tests after pier cracking

A second set of FE analyses was also performed (involving only the 3D soil model presented in Figure 4) for the pull-out tests that followed the initial pier cracking stage. In order to account for the concrete cracking mode at the base of the pier in a simplified way, an effective modulus of elasticity equal to the 50% of the un-cracked section was used along the lowermost 30cm of the pier, to equivalently match the (modified by 15%) frequency of the cracked pier. The pressure cells were simulated using 2-D elements embedded within the solid mesh; the solid element properties correspond to the actual two-layer soil profile. Through sensitivity analyses, it was ensured that the outer soil-volume boundary conditions do not affect the numerically predicted response. An overview of the modeling approach is illustrated in Figure 4.

3.3. Numerical simulation of material (soil) and geometrical (soil-foundation interface) non-linearities

As a next step, it was attempted to simulate the geometric non-linearity arising from the possibility of the foundation to uplift during pier rocking (i.e. detachment at the foundation-soil interface), utilizing the elastic model of Figure 4; moreover, this next step includes the possibility for the soil to develop plastic deformations due to soil compression. For the first type of (geometrical) non-linearity, a set of unilateral springs were used resisting only compression; these were placed at the soil-foundation interface. The material non-linearity was implemented by adopting a Von-Mises failure criterion for those soil solid elements under 3-D compression. The corresponding soil regions were approximately identified from a preceded linear analysis for the same mesh and the same loading sequence (push-over applied at the deck of the pier, see figure 4).

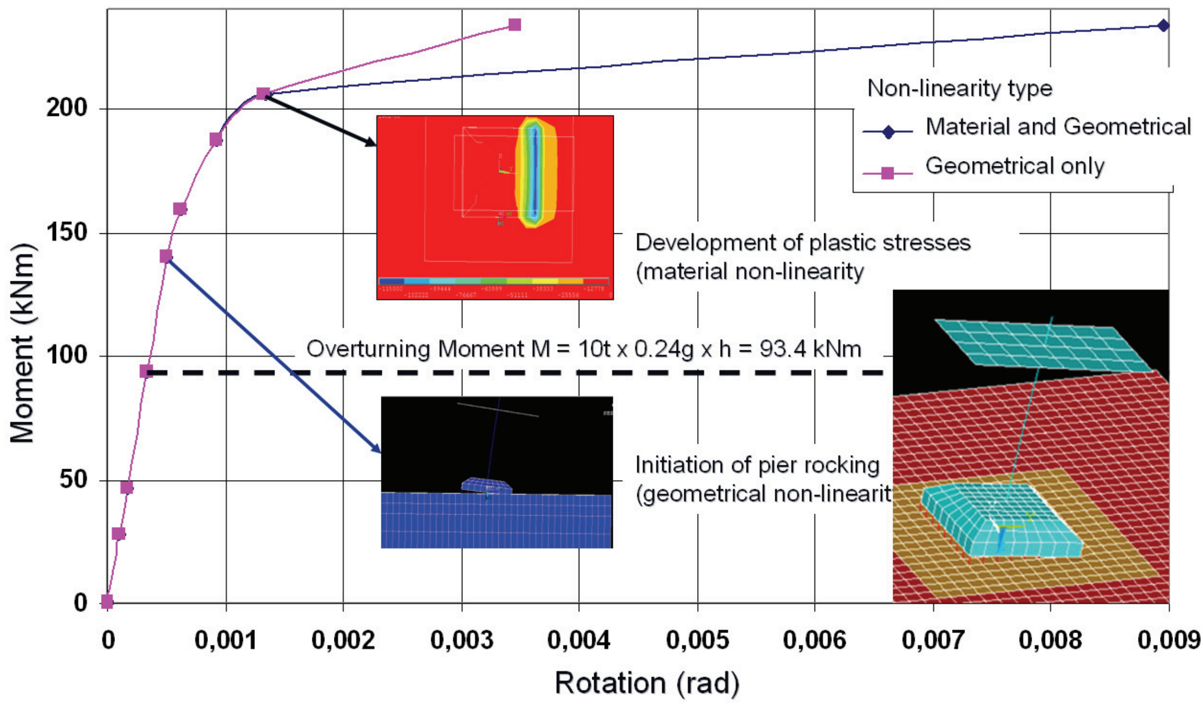


Figure 5: Numerically derived Moment-Rotation curves of the pier foundation accounting for geometrical and material non-linearities.

Figure 5 illustrates the corresponding, numerically derived, Moment-Rotation response curves of the pier-foundation system. One of these curves represents the pier rocking response when only the geometric non-linearity is included while the second response curve was obtained when both the geometric as well as the material non-linearity are included in the simulation. At the same figure the horizontal dotted line marks the upper limit of the overturning moment applied to this system during the experimental sequence in-situ. As can be seen in this figure, the geometrical and material non-linearities exercise a noticeable influence on the predicted response for overturning moment value approximately equal to 140kNm. This is at least 60% higher than the maximum overturning moment applied during the experimental sequence in-situ. It must be pointed out that for this phase of the experimental campaign this was intended in order to ensure the initiation of failure at the base of the pier. The inclusion of only the geometric non-linearity leads to a rather modest non-linear Moment-Rotation response for overturning moment values larger than 180kNm. When the numerical simulation includes both the geometrical (uplift) and the material (soil) non-linearities, for overturning moment values higher than 200kNm it results to a rather large rocking response, which indicates the eventual overturning of the pier. This is believed to represent a realistic representation of the expected actual behavior under the condition that the non-linear behavior of the soil volume is well simulated with the assumed material law. Such levels of overturning moment values were not reached experimentally because the pier response was dictated by non-linear mechanisms which developed at the pier itself (plastic hinge formation at the bottom of the pier). The level of overturning moment required for such a soil-foundation non-linear response to develop is more than twice the maximum level applied so far. This is one of the future objectives for the in-situ experimental facility.

4. THE PIER MODEL ON THE SHEAR STACK TESTED AT THE LABORATORY

Two shear stack apparatus were constructed and tested at the laboratory of Strength of Materials and Structures of Aristotle University. The first was constructed with wooden panels having at their interior interface with the volume of sand a soft layer of synthetic foam of 25mm thickness (figure 6a). The second was constructed, as it was done by Taylor (1996) with aluminium hollow cross-sections forming seven horizontal square closed frames that were joined with six 13mm soft layers of synthetic foam (figures 6b and 6c). Both shear stack

Figure 6a. 1st Shear stack with wooden sections.

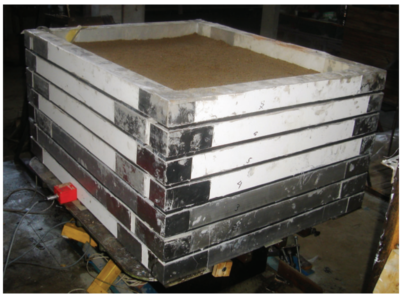


Figure 6b. 2nd Shear stack with aluminum sections.

Fig. 6c. Model pier and 2nd shear stack tested at the Laboratory.

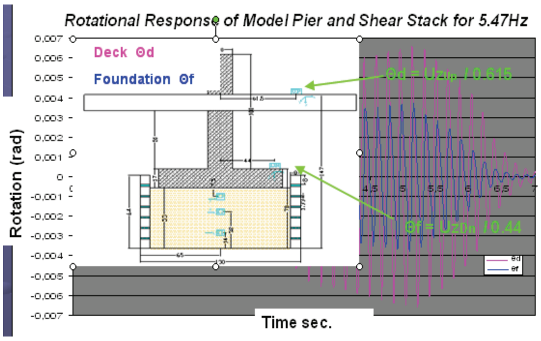


Figure 6a. Rotational (rocking) response of the pier model at the foundation and deck levels

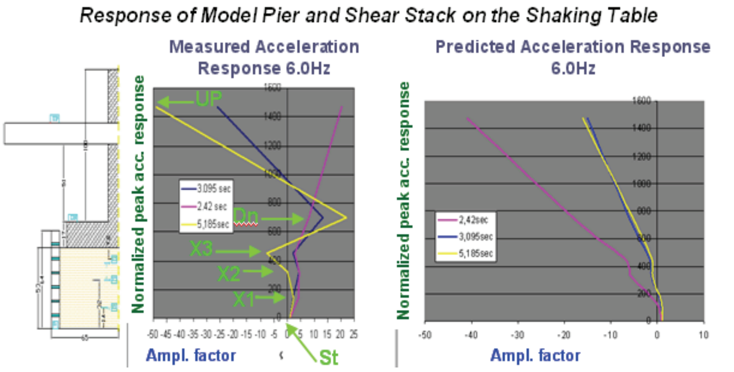


Figure 6b. Measured and predicted acceleration response of the shear stack with the model pier on top.

apparatus were tested on the shaking table either empty or with sand (figure 6b). At the final stages of testing the model pier with its foundation block was placed on top of the sand (figure 6c). Instrumentation was provided to monitor the acceleration response of the sand volume as well as that of the pier either at the foundation block or the top of the deck levels. The acceleration response of the volume of sand was monitored by embedding the acceleration sensors at three levels at the centre of the sand volume; the first very close to the bottom near the steel platform of the shaking table, the second at mid-height and the third very near the surface of the sand volume underneath the foundation block.

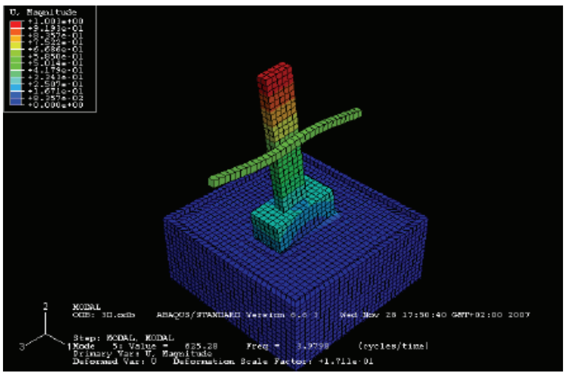


Figure 7. 3-D Non-linear numerical simulation of the shear stack with the pier on top.

Figure 6a depicts the rotational (rocking) response of the pier model both at the foundation and the top of the deck levels as it was derived from the acceleration measurements for a test with sinusoidal base excitation equal to 5.47Hz. Figure 6b depicts a profile of the variation of the acceleration response of both the sand and the pier model in terms of amplification factors along the height of the sand-pier model assembly (taking as unit the response at the base). The measured response in this figure is compared with predictions made on the base of a linear 2-D numerical simulation that represented the shear stack -sand-pier model assembly. As can be seen in this figure there certain similarities comparing the measured and predicted response as well as certain discrepancies. The later are more pronounced at the foundation block to sand interface and they may be attributed to non-linear response mechanisms that develop at this interface. Currently, an additional numerical model is studied (figure 7); apart from being a 3-D representation of the shear-stack sand-pier model assembly this numerical model includes the capability of numerically approximating two important non-linear mechanisms at the foundation to sand interface using a pushover loading sequence; the first non-linear mechanism is the possible separation between the foundation block and the sand volume whereas the second addresses the possibility of plastification of the volume of sand at regions of stress concentration (see also Maugeri 2000), caused by the foundation block during separation.

5. SUMMARY AND CONCLUSIONS

- A lowering of the fundamental translational eigen-frequency values by almost 10% was observed for the low-intensity tests after the cracking of the pier in-situ, when these values are compared with the corresponding eigen-frequency values for similar low intensity tests the equivalent maximum damping ratio before cracking. In both cases (before and after pier cracking), the corresponding stiffness variation includes influences from the flexible foundation conditions; however, these are not dominant.
- A noticeable increase was observed in the in-situ model response damping ratio values from 1.6% (before cracking, low-intensity tests) to 4.2% (during cracking, low-to-medium intensity tests). This must be attributed to the cracking of the pier as well as to the soil-foundation interaction during this intense shaking. The observed cracking pattern is in agreement with similar cracking patterns observed at the laboratory.
- The numerical simulation of the in-situ bridge-pier model dynamic response during the low-intensity tests, both before and after cracking, was quite successful.
- Reasonably good agreement can also be seen when the measured in-situ foundation flexibility, in terms of rocking stiffness and maximum pressure cell force per unit overturning moment, is compared with corresponding either analytical or numerical prediction obtained from refined FE 3-D simulations of the structure and the soil.
- The measurements of the shear-stack sand-pier model assembly at the laboratory shaking table were used to check and tune simple and complicated numerical simulation tools that include the capability of certain non-linear response mechanisms at the foundation block to soil interface. This investigation is still under way.

Acknowledgements
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