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STUDY OF THE DYNAMIC SOIL-STRUCTURE INTERACTION OF A BRIDGE PIER MODEL BASED ON STRUCTURE AND SOIL MEASUREMENTS

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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.

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 Soil Structure Interaction (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; b) use model structures in-situ to investigate the beneficial or detrimental role

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that the soil-foundation flexibility (SSI) has on the overall dynamic response; c) introduce structural yielding on the model structures and investigate the coupling between the structural yielding and the SSI effects; d) examine the nature and the effect of the waves transmitted by the oscillation of the superstructure to the foundation level and the surrounding soil; e) use the Aristotle University Laboratory facilities to verify post-elastic behavior of model bridge piers as well as effectiveness of repair techniques; f) use the in-situ measurements to validate empirical, analytical or numerical simulations of this soil-foundation-structural flexibility and damping on the dynamic and seismic structural response. Despite the disadvantages of being unable to produce significant in-situ levels of ground motion, when desired, as can be generated by an earthquake simulator, this is in part compensated for by the realistic foundation conditions, which are present for this model structure that is supported on the soft soil deposits. 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.

Figure 1. Overview of the Test Site facilities.

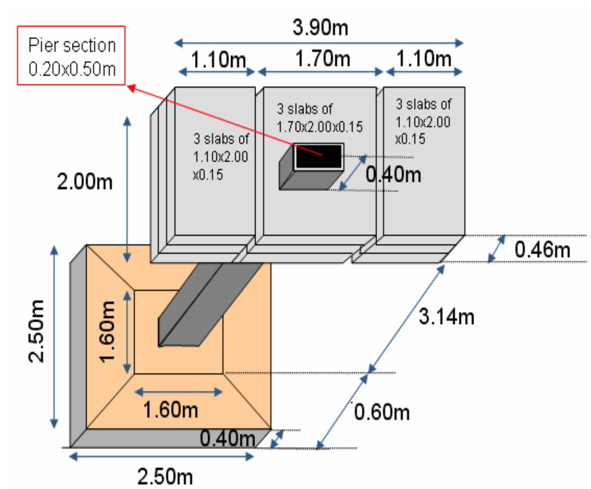


Figure 2. Basic geometry of the pier model.

Description of the Test Site Facilities

Overview

Two model structures have been built at the Volvi Test Site (Fig. 1). The latest model structure is a single bridge pier and its foundation block with an overall geometry and mass distribution depicted in Figs. 2 and 3. 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 (Fig. 4). 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 are not repeated here due to space limitation.

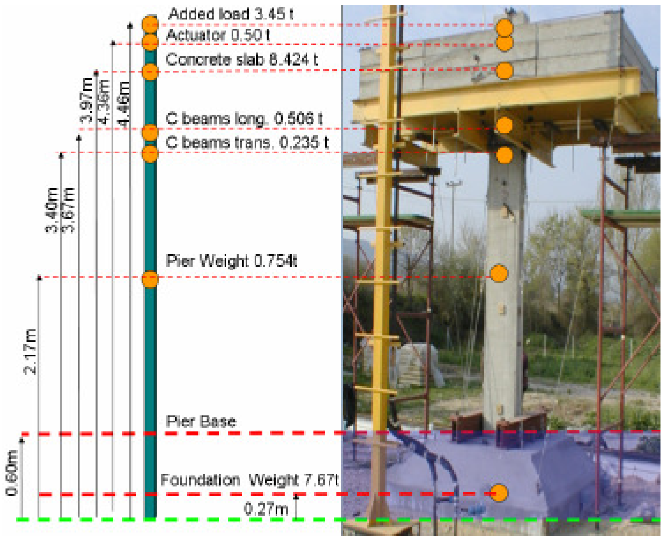


Figure 3. Mass distribution of the pier model.

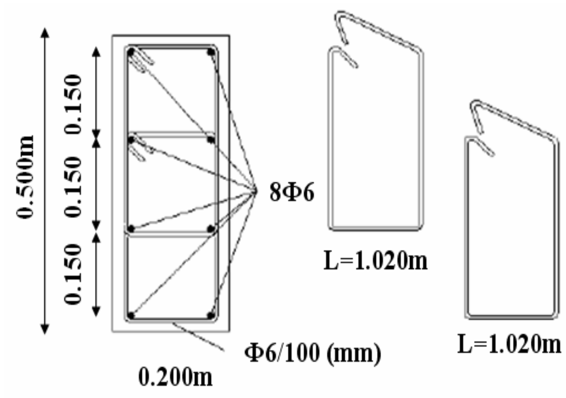


Figure 4. Basic cross-section of the pier model.

Dynamic Tests at the Test Site

A number of acceleration sensors were utilized as instrumentation for this bridge-pier model together with four pressure cells, which were placed 150mm under the foundation-soil interface to monitor the variation of the soil-pressure distribution during testing (Fig. 9). The proper operation of the instrumentation was carried out extensively. For this purpose, a number of testing sequences were performed starting from April 2004 till November 2004. After a series of trial tests, the final tests were performed during January, May and June 2005. Results from all tests will be presented and discussed in the following sections.

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 Fig. 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. The typical sequence includes a number of tests in each direction. Because the model structure was provided with diagonal supportive cables and struts, tests were performed with or without the supportive cables and struts in-place. Here, follows a selection of the most important aspects of the measured response that are presented and discussed. In Fig. 5a 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.

In-plane acceleration deck response

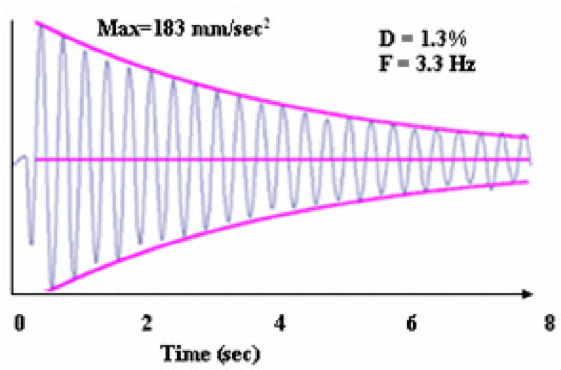


Figure 5a. Low intensity pull-out test. (structure with cables and no extra mass).

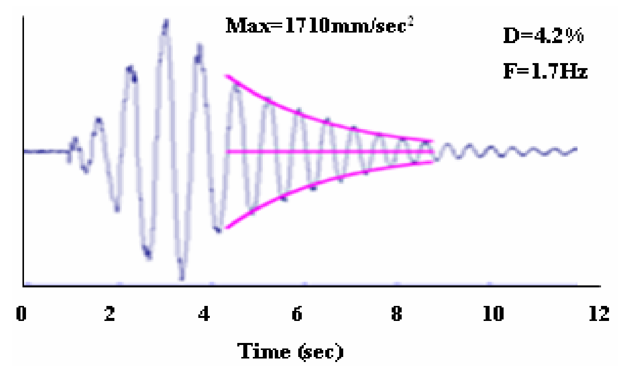


Figure 5b. Low-to-medium intensity forced vibration test that produced the pier damage (structure with cables and extra mass).

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

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. An indicative soil pressure measurement is illustrated in Fig. 6a, as it was measured by the four pressure cells located at the foundation-soil interface near each corner of the foundation block (Fig. 9).

The 2nd low-to medium intensity test that produced the Pier damage

During this test the diagonal cables and struts were active. Fig. 5b depicts the deck acceleration response during this test. By comparing this response with the corresponding response of the deck during the low intensity test (Fig. 5a) the severity of the forced vibration test can be seen in terms of deck horizontal acceleration. The soil pressures recorded during this test are depicted in Fig. 6b. As can be seen, the maximum pressure cell force is nearly 30% higher than the corresponding value that was attained during the previous test (Fig. 6a). Listed in Table 2 is the variation of the measured eigen-frequencies before and after the development of the damage at the pier base.

Pressure force measured by the pressure cells at the foundation-soil interface during the low-to-medium intensity tests. Model structure with cables and extra mass.

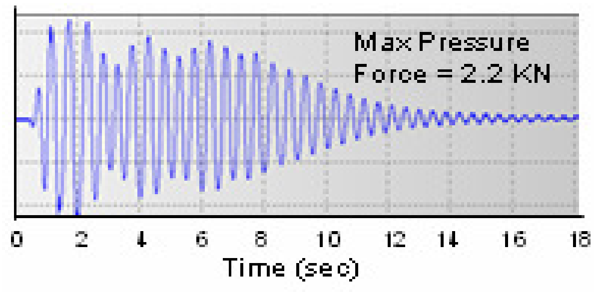


Figure 6a. During the 1st test.

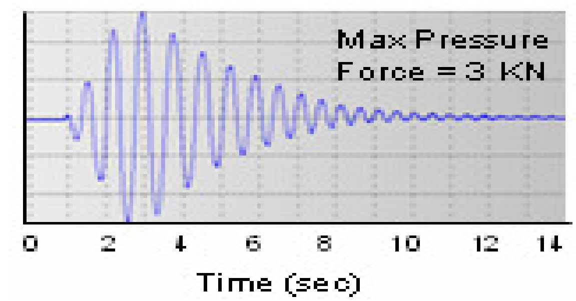


Figure 6b. During the 2nd test.

Moreover, the damping ratio value of the decaying part of the response for this test was equal to 4.2%, much larger than the one during the low intensity tests which was equal to 1.3% (Figs. 5a and 5b). This may be attributed to both the cracking and non-linear response of the pier as well as to an increase in the damping contribution from the foundation – soil response that arises from the higher intensity of the excitation. This test produced damage to the bridge pier model in the form depicted in Fig. 7. The non-linear response of the bridge pier model is also noticeable in Fig. 8, where the horizontal displacement at the middle of the concrete deck is plotted against the base shear force. As can be seen in Fig. 8, the response becomes nonlinear above a base shear value approximately equal to 15KN.

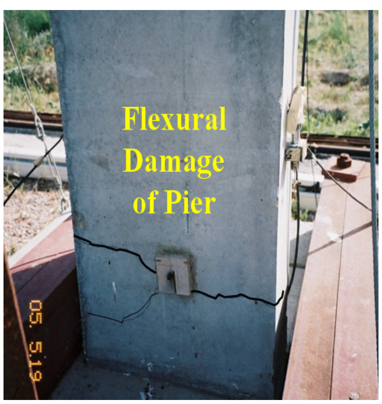


Figure 7. Flexural damage of the pier near its base after the 19th May 2005 experiment.

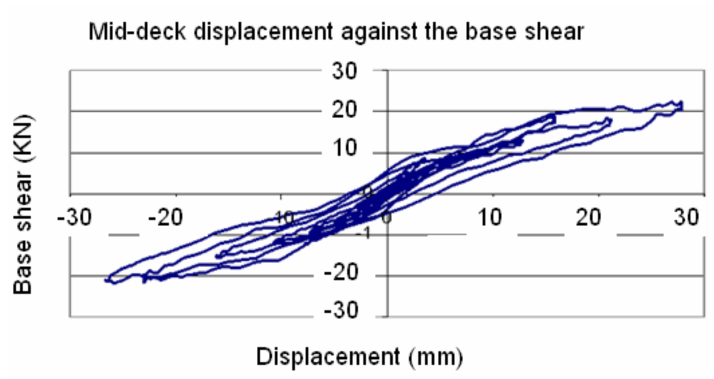
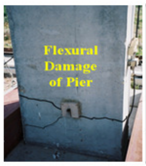


Figure 8. Base shear force - displacement response at the middle of the Deck 2nd low-to-medium intensity test, 19th May 2005). Model with cables and struts.

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	-

Numerical Prediction of the Soil-Foundation-Pier System

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 (Fig. 9)
- A 3D FE model (Fig. 10) 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).

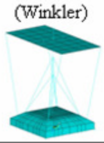
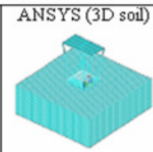
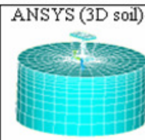
The last simulation, mobilizing this particular BEM approach, was based on the so called Thin-Layer Method while the soil was idealized as homogeneous infinite half space (Renault and Meskouris, 2005). The coupling of the pier-foundation-subsoil system was formulated with non-relaxed boundary conditions employing Greens functions which were based on the frequency domain formulation. For the models that soil flexibility was modeled with springs, the required stiffness matrix was calculated based on the theory of Mylonakis et al. (2002). The soil-related properties were determined from the measured response of the 6th story model structure that is placed nearby (Fig. 1) as well as from the available in-situ cross-hole measurements.

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. 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 modelling; thus, in 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 modelling (Fig. 9) was used in order to:

- extend the investigation towards the non-linear response of the pier without compromising the soil modelling 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 in the future, 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	

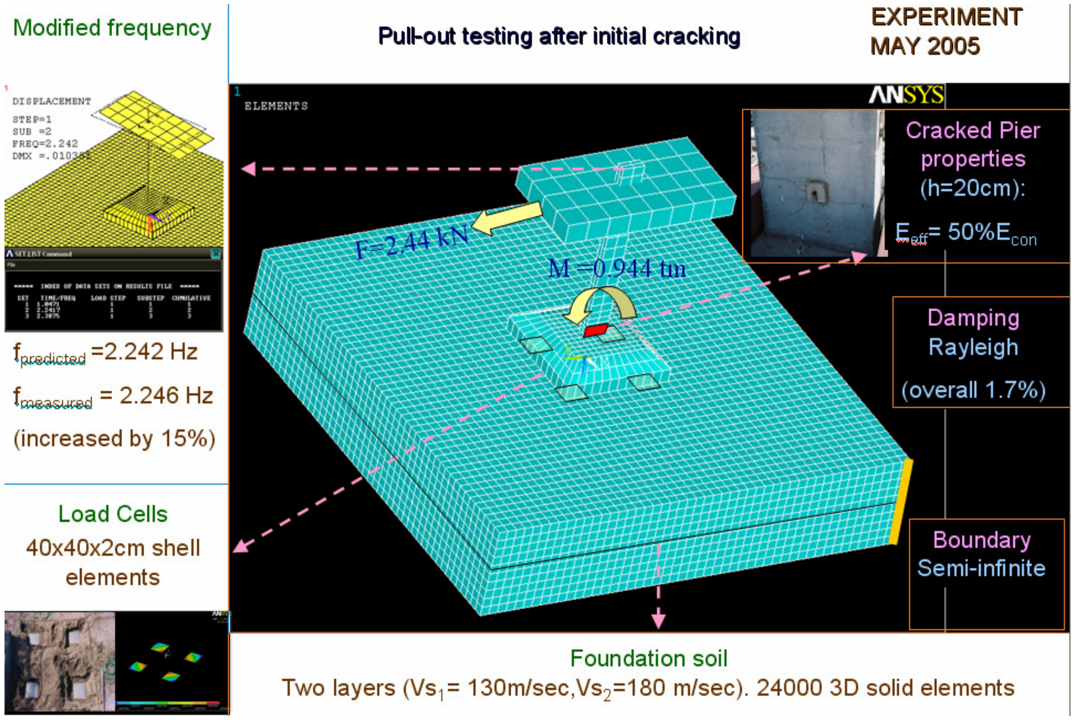


Figure 9. Overview of the FE model for the case of the post-cracking pull-out tests.

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 Fig. 9) 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 modelling approach is illustrated in Fig. 9. Characteristic numerical results are shown in Figs. 11-12 for two different vibration test sequences (20th Oct. 2004 and 13th May, 2005).

Numerical simulation of material (soil) and geometrical (soil-foundation interface) nonlinearities

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 Fig. 9; 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 Fig. 9).

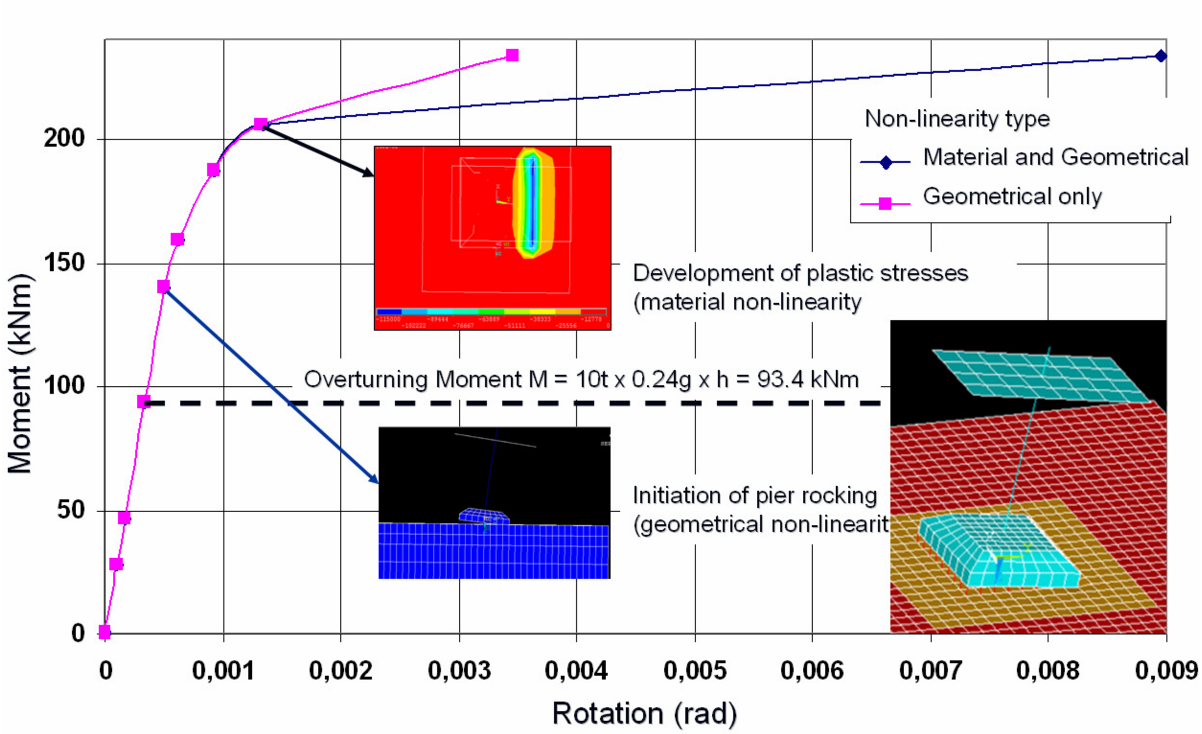


Figure 10. Numerically derived Moment-Rotation curves of the pier foundation accounting for geometrical and material nonlinearities.

Fig. 10 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 140kN.m. 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 (see Fig. 7). The inclusion of only the geometric non-linearity leads to a rather modest non-linear Moment-Rotation response for overturning moment values larger than 180kN.m. When the numerical simulation includes both the geometrical (uplift) and the material (soil) non-linearities, for overturning moment values higher than 200kN.m 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, see Fig. 7). 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.

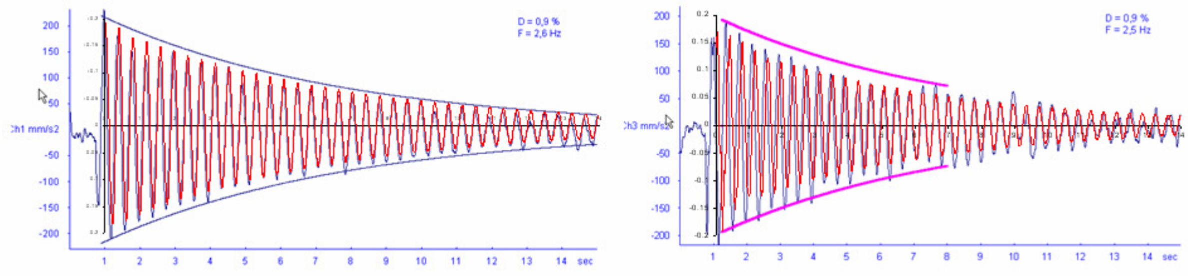


Figure 11. Low-intensity pull-out test 20th October 2004 (left) Deck horizontal response (right) Deck vertical response (Numerical predictions in red color).

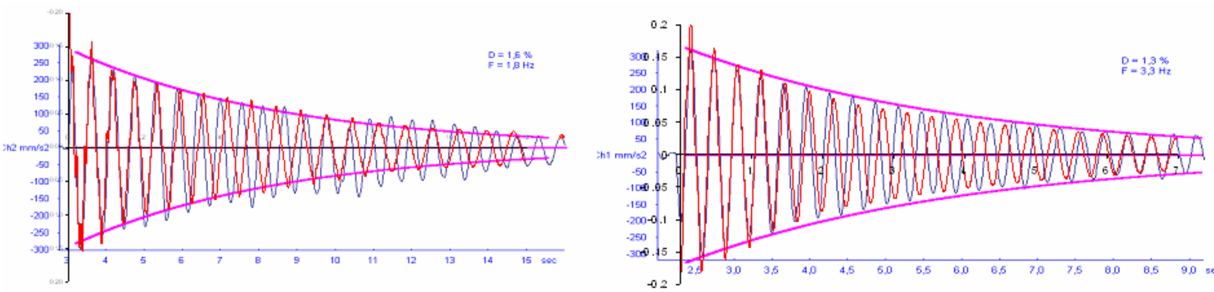


Figure 12. Low-intensity pull-out test 13th May 2005 (left) Deck horizontal response – Out-of-plane direction (right) Deck horizontal response – In-plane direction.

Evaluation of the Soil Behavior Utilizing the Measurements

Foundation rocking stiffness

Having measured the dynamic response of the pier and its foundation under low and low-to-medium excitations as well as having obtained a good agreement between the measurements and the numerical predictions, it was deemed interesting to compare the experimentally derived foundation rocking stiffness with the widely used expressions available in the literature. Along these lines, the rocking stiffness of the pier-foundation-soil system was measured equal to:

$\mathbf{K_{r,exp} = M/\theta} = 9.44\text{kN.m}/2.72\text{x}10^{-5}$ rad = 347000 kN.m/rad (1)

and is compared to the theoretical stiffness which is equal to (Mylonakis et al., 2002):

$\mathbf{K_{r,th} = 0.45GB3/(1\text{-}v)}$ = 261000 kN.m/rad (2)

It is observed that the two values do not differ by more than 30%, a discrepancy which is acceptable given the idealizations of the analytical approach (i.e. assumption of a uniform soil half-space, linear soil response, complete contact at the foundation-soil interface).

Soil pressure response

Fig. 13 depicts the cell pressure versus the applied overturning moment relationship that is derived from both the available measurements, at 150mm depth from the soil surface, and the numerical predictions at the same level. As can be seen from this comparison the agreement is reasonably good. The pressure cells measurements (expressed in terms of Force) were also compared with the numerical predictions of the refined FE model depicted in Fig. 9 in which the pressure cells where modelled as distinct shell elements with their actual stiffness and thickness, embedded within the 3-D soil FE mesh. This reasonably

good agreement can also be seen when the measured pressure cell maximum force per unit overturning moment (0.253 kN/tm) is compared with the corresponding numerical prediction (0.243 kN/tm) obtained for the low-to-medium excitation test, which is depicted in Fig. 14. Because the numerical model results in a non-linear soil-stress distribution under the foundation (see bottom right corner of Fig. 13), this numerical force value was derived from an averaging process utilising the numerical predictions at the pressure cell integration points of the F.E discretization.
Obtaining instead a pressure force value, based on the commonly used assumption of linear soil stress distribution under the foundation when it is subjected to an overturning moment, the predicted pressure force value is 50% larger than the value measured by the pressure cells in-situ (Fig. 15). This fact suggests that this linear distribution assumption is a simplification and that the non-linear distribution, predicted by the refined 3-D numerical simulation corresponds with reasonable accuracy to the actual state of soil-stress under the foundation. Furthermore, one can potentially utilise these pressure cells measurements in order to assess the level of the soil stresses that resulted from the low-to-medium pier excitation and conclude indirectly whether the soil behaviour underneath the foundation remained elastic.

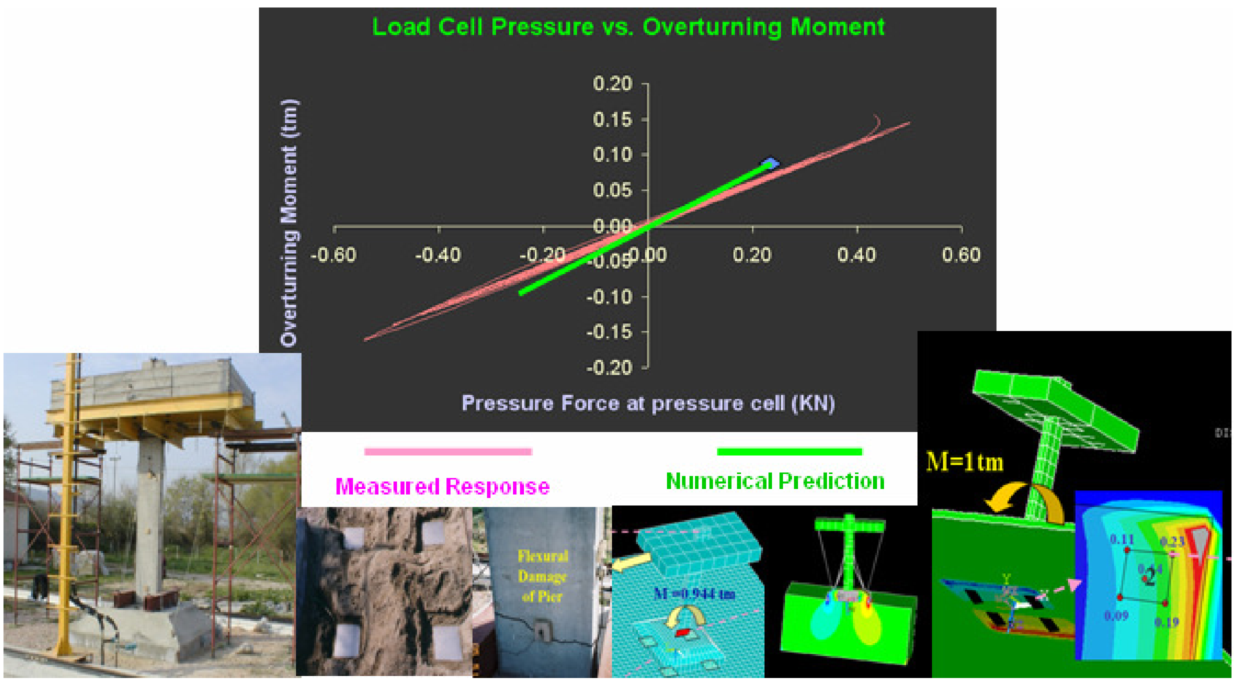


Figure 13. Variation of load cell pressure with overturning moment. Test of 8th June (after pier cracking). Pier with cables and struts.

Discussion of Results and Conclusive Remarks

Dynamic behavior during the low-intensity tests.

Stiffness variation

Pre-cracked condition. The measured eigen-modes and eigen-frequencies in the in-plane and out-of-plane directions for the pier with the extra lead mass had initial values equal to 2.80Hz (in-plane) and 1.93Hz (out-of-plane), respectively (pier with diagonal struts and cables). These eigen-frequencies became 2.60Hz (in-plane) and 1.12Hz (out-of-plane), when the diagonal struts and cables were removed.

Post-cracked condition. The measured eigen-modes and eigen-frequencies in the in-plane and out-of-plane directions for the pier with the extra lead mass had values equal to 2.54Hz (in-plane) and 1.71Hz (out-of-plane), respectively when the diagonal struts and cables were in place. These eigen-frequencies became 2.34Hz (in-plane) and 1.10Hz (out-of-plane), when the diagonal struts and cables were removed. Again the corresponding stiffness includes influences arising from the flexible foundation conditions.

Damping ratio variation

Pre-cracked condition. For the low intensity tests, the measured damping values, extracted from the decay of the free vibration response, are in the range of 0.9% to 1.6%.

Post-cracked condition. The measured damping values, extracted as above, are in the range of 1.2% to 1.9% a modest increase from the pre-cracked condition.

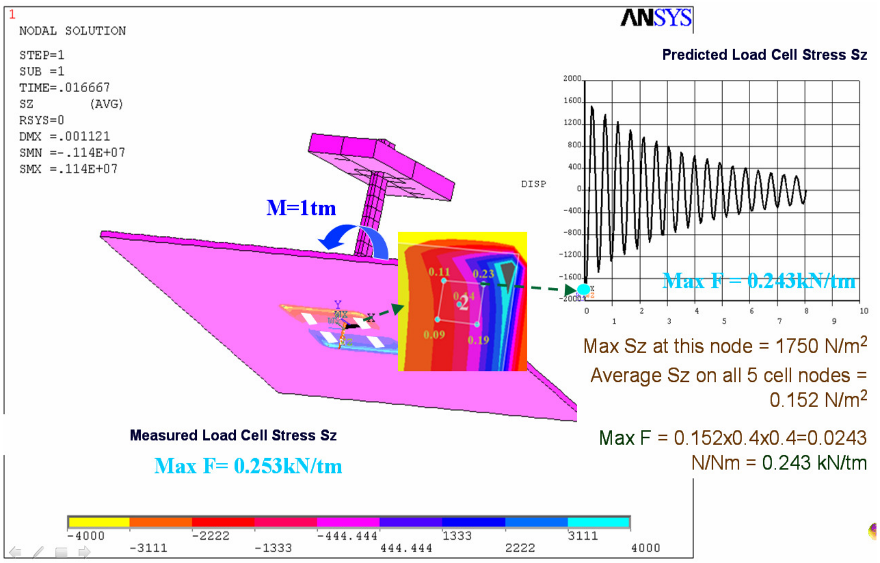


Figure 14. Comparison between the numerically predicted maximum pressure force developed at the load cell and the measured response.

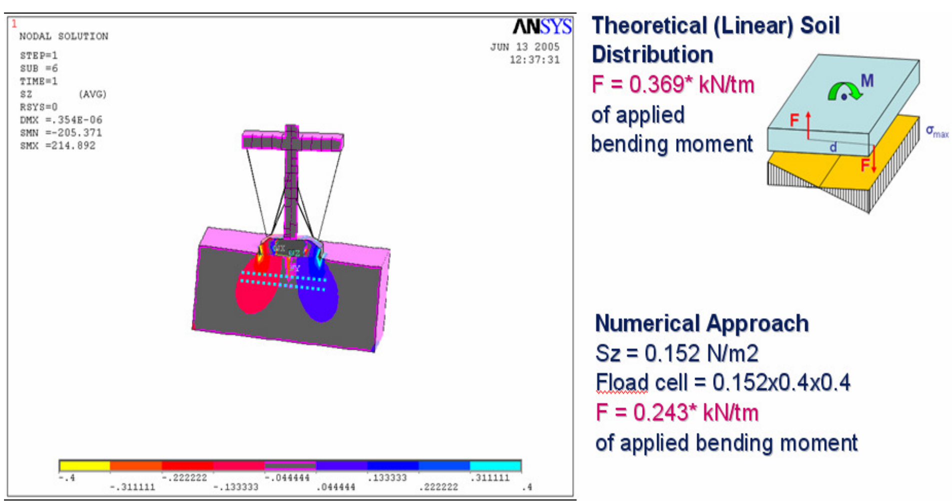


Figure 15. Comparison between the simplified theoretical approach for the distribution of the soil stresses at the foundation level and the numerical prediction of the refined 3D FE model.

Numerical Simulation

The numerical simulation of the bridge-pier model dynamic response during the low-intensity tests, both before and after cracking, was quite successful. The tried numerical simulations included various simple or relatively complex approaches in order to approximate the soil-foundation flexibility. For this particular application almost all the tried approaches resulted in good agreement between numerical predictions and experimental measurements. Moreover, a good agreement was observed between the numerically predicted maximum pressure force that was developed at the load cell in time and the maximum response measured.

Dynamic behavior during the low-to-medium intensity tests

- The non-linear behavior of the studied bridge pier model at the Test Site has already been observed on replica models, which were built for this purpose and tested at the laboratory of Strength of Materials and Structures of Aristotle University (Manos, 2004). It was found that for both tests, in the laboratory as well as at the Test Site, the predominant mode of response was the flexural mode, which led to the corresponding flexural type of damage. This damage, both at the Laboratory and the Test-Site models, was concentrated, as expected, at the model pier base, and it is in agreement with the predicted limit-state aimed at, according to the design of this model structure.
- A noticeable increase was observed in the equivalent maximum damping ratio values from 1.6% (before cracking, low-intensity) to 4.2% (during cracking, low-to-medium intensity).
- The non-linear response of the bridge pier model is noticeable when the horizontal displacement at the middle of the concrete deck is plotted against the base shear force for the 2nd low-medium intensity test (the most intense test). Moreover, the non-linear response of the bridge pier model together with the soil-foundation interaction in-situ must be further analyzed.

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, when these values are compared with the corresponding eigen-frequency values for similar low intensity tests 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 equivalent maximum 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 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 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.

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References

Chiras, S., 2005. ‘Inelastic dynamic response of bridge piers under real soil conditions. Application at the Euroseis-Test Site’, *MSc Thesis*, Dept. of Civil Engineering, Aristotle University Thessaloniki.

Kawashima, K., 2000. “Seismic performance of RC bridge piers in Japan: an evaluation after the 1995 Hyogo-ken nanbu earthquake”, *Prog. Struct. Engng Mater.* 2000; 2: 82–91.

Manos, G.A., Kourtides, V., Yasin, B. and Soulis, V.J., 2004. “Dynamic and Earthquake Response of Model Structures at the Volvi – Greece European Test Site” *13th WCEE*, Vancouver, Canada, No.787.

Manos, G., Renault, P. and Sextos, A., 2005. Investigating the design implications of the influence between neighboring model structures at the Euroseistest site, *Proceedings of EURODYN 2005.*

Mylonakis, G., Gazetas, G., Nikolaou, S. & Chauncey, A., 2002. ‘Development of Analysis and Design Procedures for Spread Footings’, Technical Report, MCEER-02-0003.

Pinto A. V., editor, 1996. Pseudodynamic and Shaking Table Tests on R.C. Bridges. *ECOEST PREC*8* Report No. 8, November 1996.

Pitilakis, K., et al., 1999. Geotechnical and geophysical description of EURO-SEISTEST, using field, laboratory tests and moderate strong motion records. *Jrnl. of Earthq. Eng*. Vol. 3 (No. 3): 381-409.

Renault, P. and Meskouris, K., 2005. ‘A coupled BEM/FEM approach for the numerical simulation of bridge structures’, EURODYN 2005, 1267-1272.