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# Numerical and experimental soil-structure-interaction of a bridge pier model at the Volvi-Greece European Test Site

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ABSTRACT: This paper presents the experimental and numerical actions that have taken place within the context of the preparation of the man-made excitations of a model bridge pier which is constructed at the Volvi – Greece European Test Site for the study of the dynamic pier-foundation-soil system interaction. In particular, a set of in-situ low level dynamic tests have been performed which were complemented by laboratory tests and preliminary finite element analysis in order to optimize the design of the structure towards the maximization of the SSI effects while retaining the system within the available force level capabilities that can be applied on site. Both experimental and numerical results have contributed to the optimal design of the structure and the preliminary prediction of its dynamic behavior.

## 1 INTRODUCTION

Although the effect of soil-structure interaction on the dynamic response of typical residential or commercial structures and infrastructure (i.e. bridges) has long ago attracted scientific attention, it is widely recognized that there is an urgent need for its experimental support and validation. This need is far more crucial in cases where the structure responds inelastically and/or the soil conditions favor the appearance of SSI phenomena. Along these lines, significant effort has been undertaken within the context of the Euroseis-Risk Project (http://euroseis.civil.auth.gr), for Earthquake Engineering, Engineering Seismology and Geotechnical Earthquake Engineering that has been continuously funded by the European Union for the last decade. This large physical laboratory (Test Site), is located 30 km distant from Thessaloniki.

One of the main objectives of the project is to utilize the facilities at the Aristotle University Laboratory as well as those at Volvi in order to:

- define soil flexibility and damping properties.
- use Model Structures in-situ to investigate the beneficial or detrimental role that the soil-foundation flexibility (SSI) has on the overall dynamic response.
- introduce structural yielding on the model structures and investigate the coupling between the structural yielding and the SSI effects.
- examine the nature and the effect of the waves transmitted by the oscillation of the superstructure to the foundation level and the surrounding soil.
- use the Aristotle University Laboratory facilities to verify post-elastic behavior of model bridge pier as well as effectiveness of repair techniques.
- 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 by the realistic foundation conditions, which are present for this model structure that is supported on the soft soil deposits in-situ (Pitilakis et al., 1999). In fact the structure is susceptible to SSI effects according to Eurocode 8 (CEN, 2002) criteria since the corresponding shear wave velocity Vs at the surface is approximately 100m/sec. The current extension of the in-situ facility includes the possibility of subjecting the model structures to low-medium intensity man-made excitations (i.e. a number of simple pull-out test) as well as explosions.

## 2 OVERVIEW OF THE EXPERIMENTAL AND NUMERICAL APPROACH

A series of preliminary activities were undertaken for the optimal design of the model structure and a subsequent set of parametric and sensitivity analyses were performed in order to ensure that a) the forces available on site are adequate to trigger soil-structure interaction phenomena b) the frequency of the man-made excitation is such that could optimize the presence of damping of the coupled soil-foundation-structure system and c) that the foundation dimensions would be designed in a way that would prevent the, unfavorable at this stage, effect of rocking while remaining within the scale of the rest of the model structure.

Along these lines, successive finite element analyses have been performed using alternative FE codes and the results of the optimization procedure have been compared with theoretical solutions. The experimentally observed response is compared with numerical simulations at various stages of the construction process. Having captured the main insitu dynamic characteristics of the model structure a level of agreement is established that allows for the numerical prediction under the scheduled pull-out tests and explosions.

## 3 DESCRIPTION OF THE SINGLE BRIDGE PIER MODEL

The current extension of the in-situ facility includes the possibility of subjecting the model structures to low-medium intensity man-made dynamic excitations. At this point in time the model structures that are built at the test site include: a) A 6-story Reinforced Concrete building with masonry infills b) A single bridge pier specimen. The first model as well as the general layout of the Test Site (Figure 1) in terms of geometry and soil profile has been described in detail in the companion paper (Manos et al. 2005).

The latter model structure is a small scale representation of a single bridge pier (Figure 2). This type of structure has attracted research interest in the last decade, especially following the spectacular damage of bridges during the Northridge and Kobe earthquakes. Towards the objective of increasing our understanding on the earthquake behavior of bridge structures and in the framework of prenormative research of Eurocode 8 (CEN 2002) a series of pseudo-dynamic tests on 1:2.5 scaled bridge piers were conducted at ELSA Laboratoty of the Joint Research Center, Ispra (Pinto 1996). Moreover, shaking table tests on a 1:8 scale bridge model were carried out in the Structural Dynamic Testing Laboratory of ISMES, Seriate, Italy. Whereas the crosssection of the ELSA models was a hollow rectangular cross-section, more closely representing the crosssection of a prototype bridge pier, the cross-section of the ISMES piers as well as the ones to be presented in this work are of a monolithic prismatic cross-section, dictated by scaling considerations. The overall cross-section dimensions of the ISMES model piers and the model pier to be tested at the Volvi test site are quite similar. The cross-section of the Volvi model pier, together with the reinforcing details, is shown in Figure 1. Whereas the tests conducted both at ELSA and ISMES had the foundation block of the corresponding pier rigidly attached either on the strong reaction floor or on the shaking table platform, the Volvi pier foundation rests on the soil surface at the test site.

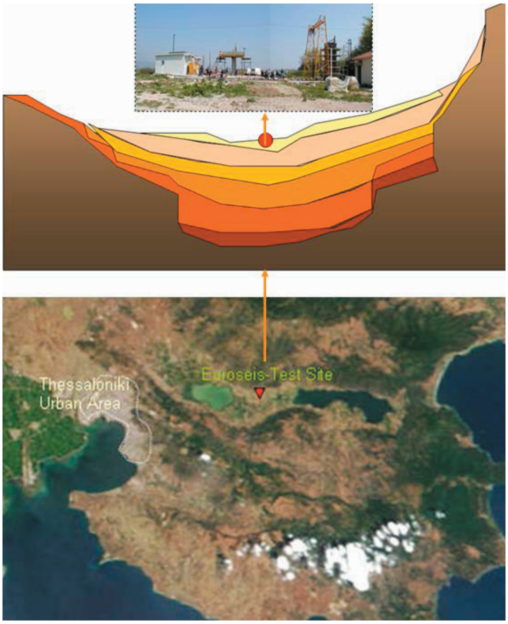


Figure 1. General layout of the Euroseis-Test Site

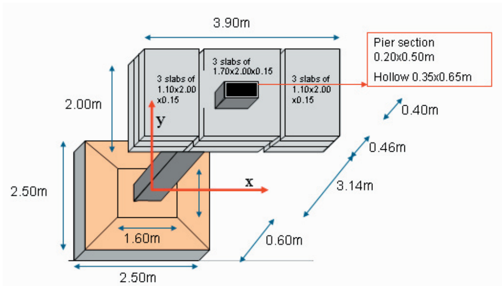


Figure 2. Geometry of the model bridge pier

## 4 LABORATORY TESTS WITH THE MODEL BRIDGE PIER

### *4.1 Cyclic Tests*

Two model bridge pier models with their foundation have been built according to the final design

(Model Pier A1 and A2 in Figure 3,4 and 5). These models are identical in all respects to the model Pier which was built at the test site. They differ in their height (they are1800mm high instead of 4000mm which is the height of the test site pier) and their foundation (1m x 1m x 0.3m) whereas the test site pier foundation is 2.5m x 2.5m x 0.5m. These models serve the purpose of ensuring the strength capacity of the identical model Pier structure which was built at the test site. They have been tested at the Strong Reaction Frame of the Laboratory of Strength of Materials under combined loading conditions resembling the ones at the test site. In this way the various features of the cyclic performance of these model structures could be observed and recorded at the Laboratory ensuring that they do not exhibit undesirable deviations from the ones predicted by the preliminary and final design in the framework of the objectives of this research program. The differences in height and foundation of these piers, from the pier to be built at the test site, have been introduced in order to accommodate them within the space limitation of the Strong Reaction Frame.

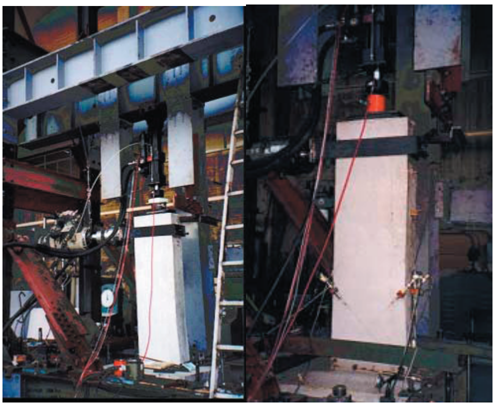

Figure 3. Pier A1 or A2 subjected to cyclic testing

Additionally to the aforementioned piers, a model pier structure (Model Pier B1) with a height of 4000mm and a foundation 1m x 1m x 0.3m was also built at the Laboratory (Figure 6). This pier has identical material characteristics (measured steel strength $f_y$ = 344.8MPa, $f_u$=470.9MPa, measured concrete strength at the bottom cross section 26MPa) as well as construction details for the concrete and reinforcing bar parts with those of models A1 and A2.

Table 1. Geometric and material characteristics of the current project pier models.

| Model Code Name | Height [m] | Foundation |
|---|---|---|
| A1 | 1.8 | 1.0m x 1.0m x 0.3m |
| A2 | 1.8 | 1.0m x 1.0m x 0.3m |
| B1 | 4.0 | 2.5m x 2.5m x 0.6m |

Figure 4. Damaged pier A1 after being tested and slabs to be fixed at the deck of pier B1 in-situ

The dimensions of the three models are summarized in Table 1. The pier was transported to the test site. An extended foundation of 2.5m x 2.5m x 0.6m was also built at the Laboratory of Strength of Materials of Aristotle University and transported at the test site, hence this way building time was be gained during the bad weather winter months. Moreover, in this way the compatibility of the various fixtures for attaching the constructed part of Model Pier B1 with that of the extended foundation were tested inside the Laboratory.

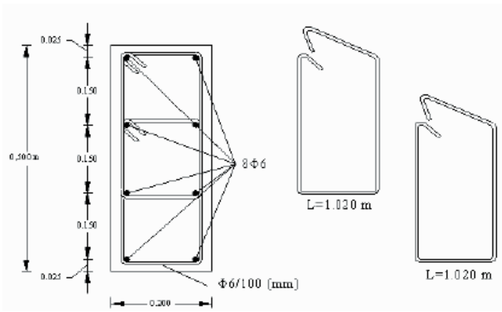


Figure 5. Dimensions and reinforcing details of Volvi pier cross-section (Models A1, A2 and B1)

A steel platform has been designed and was attached onto Pier B1 at the test site. This steel platform represents the deck of the pier. On top of this platform weights in the form of concrete slabs were be fixed, that provided the necessary weight in order to apply on the cross-sections of the piers the desired level of axial load. Moreover, these added masses will also generate horizontal forces of a desired amplitude at this “deck” level. The R.C. slabs that were fixed at the model B1 pier bridge deck are of two types. The first type (6 pieces) is of 1.1m x 2.0m x 0.15m whereas the second type is 1.7m x 2.0m x 0.15m (Figure 4). The total weight of the steel platform and the R.C. slabs is approximately 9 tones. Pier models A1 and A2 have been tested at the strong reaction frame at the laboratory. They were subjected to combined horizontal and vertical loads. The vertical load had to be kept constant during the application of the horizontal cyclic displacements. For this purpose two different ways were tried at the laboratory. In the first way, which was used for model A1, two hydraulic jacks were utilized.

Figure 6. Dimensions and reinforcing details of Volvi pier cross-section (Models A1 and A2)

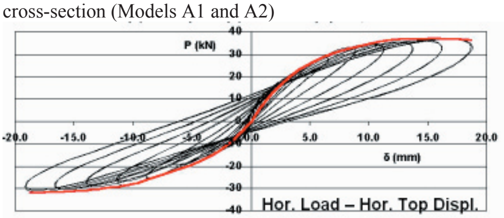


Figure 7. Observed cyclic behavior of pier A2 tested at the strong reaction frame (Horizontal Load – Horizontal Displacement H = 1.42m).

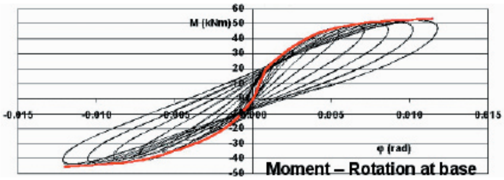


Figure 8. Observed cyclic behavior of pier A2 tested at the strong reaction frame (Moment – Rotation).

These jacks are only one-way active jacks without any electronic control. The level of vertical load is kept constant through a system of accumulators. This way of applying the vertical load did not successfully maintain constant the vertical load. A variation of the vertical load could be observed during the cyclic variation of the horizontal load. This variation of the vertical load increased as the amplitude of the horizontal displacement at the top of the pier was increased. The second way of applying the vertical load utilized a two-way hydraulic actuator electronically controlled. This was used in model A2. Again a variation of the vertical load could be observed during the cycling variation of the horizontal displacement at the top of the pier.

However, this variation was of smaller amplitude than the corresponding variation during the tests where the vertical load was applied in the first way, and was kept at relatively low levels even for large horizontal displacement amplitudes at the top of the model pier. The axial compressive load of the pier cross-sections, resulting from the application of the vertical load, was approximately equal to 10tones.

The observed cyclic behaviour of pier A2 in terms of the horizontal load-horizontal displacement (P-δ) and moment-rotation (M-θ) curves, is illustrated in Figures 7 and 8.

## 5 NUMERICAL INVESTIGATION OF THE DYNAMIC RESPONSE OF THE PIER

### *5.1 Overview of the FE approach*

Different Finite Element models were constructed aiming to provide an ascending level of modeling complexity in order to obtain the optimum balance between model simplicity and accuracy. In particular the following FE models were used:

a) A simple frame-type model with appropriate mass distribution and flexible support with the use of the FE code SAP2000, which can be extended to account for the development of a plastic hinge by the appropriate coupling of plastic rotations with soil flexibility (Kappos & Sextos, 2001).
b) A 3D spring supported model with equivalent cube-type foundation with the use of FE code LUSAS.
c) A complete 3D model supported on compression only non-linear springs and concrete cracking/crashing capabilities for subsequent implementation with the use of the FE code ANSYS.
d) A complete linear elastic 3D 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 (Figure 9).
e) A complete 3D model supported on soil (solid elements) implemented within a comprehensive FEM/BEM approach (Manos et al., 2005).

For the models that a spring support is adopted for the representation of soil flexibility, the required stiffness matrix is calculated with the use of the computer code ASING (Sextos et al., 2003) and the well known theory of Gazetas et al. (2002). These values are summarized in Table 2.


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Figure 9. Complete FE model inclusive of the additional steel section for the pier-deck connection and the restraint cables for the foundation-deck connection.

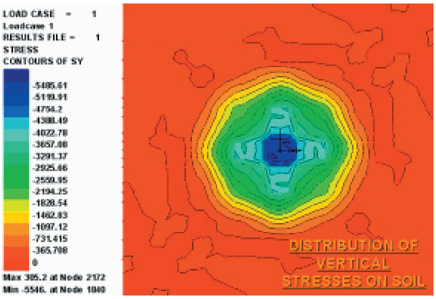
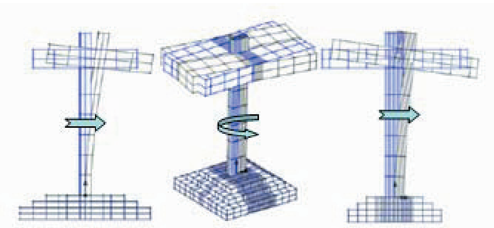



Figure 10. Distribution of expected vertical stresses on soil



out of plane(y-y) rotational in-plane (x-x)

Figure 11. The three first modes of vibration.

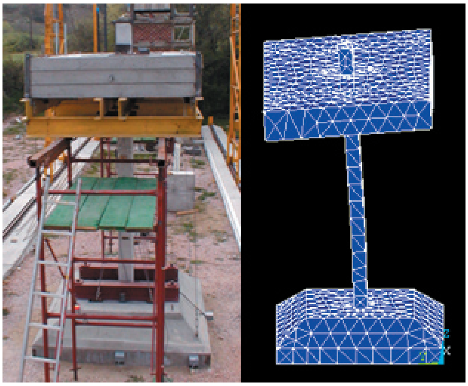


Figure 12. 3D FE model and 1st natural period with the use of the commercial FE package ANSYS

Table 2. Static stiffness matrix of the model pier-foundation-soil system

| Degree of freedom | Stiffness |
|---|---|
| Horizontal $K_{xx} = K_{yy}$ | 91000 kN/m |
| Vertical $K_{zz}$ | 105000 kN/m |
| Rocking $K_{rx} = K_{ry}$ | 520000 kNm/rad |
| Torsion $K_{rz}$ | 950000 kNm/rad |

Table 3. Dynamic characteristic and static response of the fixed-base pier

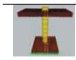
| Finite Element Model | Static Displ. (cm) | Static Rotation ($^o$x$10^{-3}$) | Mode y-y [Hz] | Mode rot. [Hz] | Mode x-x [Hz] |
|---|---|---|---|---|---|
|  SAP2000 | 4.91 | 10.09 | 1.36 | 2.33 | 3.15 |
|  LUSAS | - | - | 1.34 | 2.44 | 3.03 |
|  ANSYS | 4.56 | 7.87 | 1.33 | 2.41 | 3.30 |
| FEM/BEM | | | 1.31 | 2.46 | 2.99 |
| Analytical | | | 1.31 | 2.33 | 3.22 |

Table 4. Dynamic characteristic and static response of the flexibly supported pier

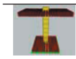
| Finite Element Model | Static Displ. (cm) | Static Rotation ($^o$x$10^{-3}$) | Mode y-y [Hz] | Mode rot. [Hz] | Mode x-x [Hz] |
|---|---|---|---|---|---|
|  SAP2000 | 5.62 | 11.70 | 1.35 | 2.27 | 2.95 |
|  LUSAS | - | - | 1.34 | 2.44 | 2.88 |
|  ANSYS | 5.01 | 8.65 | 1.27 | 2.30 | 3.16 |
| FEM/BEM | | | 1.23 | 2.46 | 2.27 |

Table 5. Volume and weight of the pier

| Pier part | Volume ($m^3$) | Weight (kN) |
|---|---|---|
| Foundation | 3.03 | 75.75 |
| Pier section | 0.36 | 8.88 |
| Deck | 3.60 | 90.00 |
| Additional C220 connecting steel beams | | 7.06 |

### *5.2 Comparative study of the numerically derived dynamic and stiffness characteristics of the pier*

The implementation of the aforementioned alternative FE models gives an insight of the expected dynamic characteristics and stiffness of the overall deck-pier-foundation-soil system. Figure 10 illustrates the distribution of the soil stresses expected to be developed at the soil-foundation interface due to the vertical (self) load of the pier only. Moreover, Figures 11 and 12 as well as Tables 3 and 4 summarize the calculated natural frequencies (herein only the first three that are of interest are presented) together with the derived horizontal rotation and deck rotation for the case of a man-made horizontal force equal to 1770kN applied at the center of the deck mass and along the stiff axis (in plane) of the pier. The corresponding particle and total weight of the system is summarized in Table 5.

What can be observed in general from the numerical analysis results is that, there is good agreement between the calculated natural frequencies for both the fixed base and the flexibly supported system despite their different modeling complexity. This fact essentially implies that for this stage, all modeling approaches are acceptable and provide confidence that the dynamic behavior of the pier is well understood and represented. In particular, Models a), b) and c) are both effective and not extremely expensive in terms of computational time, while Models d) and e) are more complex but provide additional capabilities.

Moreover, it is interesting to see that the influence of the foundation flexibility appears to be more noticeable when the BEM/FEM approach was followed. This subject will be dealt with, in more detail in the future with the help pf the experimental observations in-situ. The above are well suited to be valid for low intensity excitation. For medium intensity excitations that are also planned at the test site, inelastic behavior may arise from either the pier or the soil foundation inelastic response.

It has also to be noted the apparent result that when the pier is flexibly supported, its period increases. Whether dynamic soil-structure interaction is beneficial or detrimental though, is a complicated and multi-parametric issue that has to be studied in depth since soil damping and the relative stiffness of the system together with the frequency content of the excitation itself strongly affect the overall dynamic response of the system.

Therefore, with the use of the well constrained laboratory results and the preliminary numerical analyses presented herein as well as the measurements to be obtained in-situby the series of the artificial excitations, the dynamic SSI effects can be studied effectively and the knowledge gained to be extrapolated for the final stage of the study of the inelastic dynamic response of the pier-soil system.

## 6 CONCLUSIONS

Towards the identification of the dynamic pier-foundation-soil system interaction a model bridge pier has been constructed at the the Volvi – Greece European Test Site. With the use of experimental means (in the laboratory and in-situ) as well as numerical computations, the design of the model structure is optimized and its dynamic behaviour is predicted. Along these lines, it is concluded that the model structure is very well controlled both experimentally and numerically and that an acceptable level of confidence has been established in order to proceed to the final set of artificial excitations up to failure.

## ACKNOWLEDGEMENT

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