DYNAMIC AND SEISMIC BEHAVIOUR OF A PIER-BRIDGE MODEL AT THE EUROPEAN TEST SITE

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INTRODUCTION

Although the effect of soil-structure interaction (SSI) on the dynamic response of typical residential or commercial structures and infrastructure (i.e. bridges, [1]) 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 inelastically and/or the soil conditions favor the appearance of SSI phenomena. For these reasons, 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 ([2], [7]).

This paper examines the behaviour of a model structure built at the Volvi European Test Site in Greece (http://euroseis.civil.auth.gr). As shown in figure 1, It is a small-scale representation of a single bridge pier and its foundation block with an overall geometry and mass distribution depicted in figures 2 and 3. The total mass is 185.3KN (41.7kips), 95.3KN (21.4kips) of which are concentrated at the deck level. This bridge pier model structure was recently built (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 [6], but of smaller dimensions and a different cross-section detailing (figure 4). 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. [2], and are not repeated here due to space limitation. This paper includes selective measurements of the dynamic response of this bridge-pier model, when it was subjected to man-made excitations of low or low-to-medium intensity. During the later sequence of tests the pier bridge model was damaged by developing cracking at its bottom region near the foundation footing. A number of numerical simulation predictions of the dynamic response of this structure are also included and discussed in this paper. 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-situ. In fact, this model structure is susceptible to soil-structure interaction (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 (i.e. a number of simple forced vibration tests as well as explosions).

Figure 1: Overview of the Test Site facilities

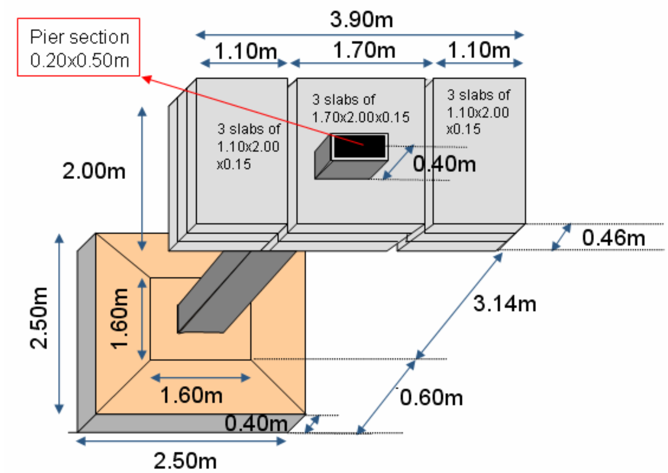


Figure 2. Basic geometry of the pier model

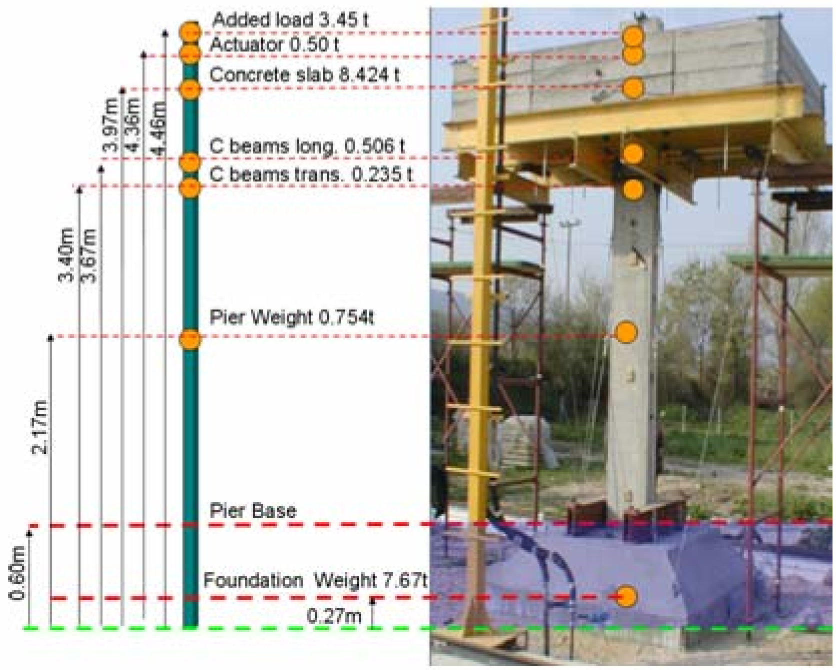


Figure 3. Mass distribution of the pier model

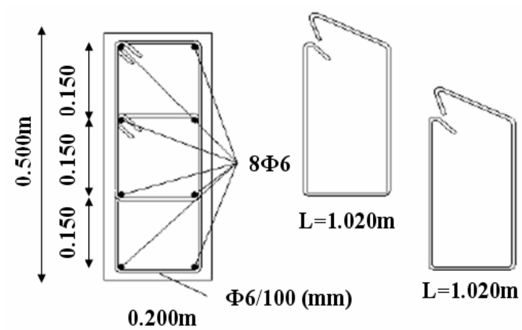


Figure 4. Basic cross-section of the pier model

DYNAMIC TESTS WITH THE BRIDGE PIER MODEL AT THE TEST SITE

A number of sensors of both the portable and permanent systems were extensively calibrated at the Laboratory of Strength of Materials of Aristotle University and then they were transported to the Test Site to be utilized as instrumentation for this bridge-pier model. Four pressure cells were placed at the foundation-soil interface to monitor the variation of the soil-pressure distribution during testing (Figure 13). The cabling of the permanent system was installed underground or in special plastic conduits. The installation of the sensors and cabling was performed at the Test Site and the checking of the proper operation of the permanent and portable sensors was carried out extensively. For this purpose, a number of testing sequences were performed starting from April 2004 till November 2004. Although some of this information is useful, the experiments performed during this period must be considered as preliminary. 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 of the permanent and portable instrumentation systems. Thus, prior to such testing, all these systems were checked to be operational in order to successfully record the dynamic response of the superstructure, the foundation and the surrounding soil. 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 (0.495 kips) in the in-plane direction and 1.4KN (0.315 kips) 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, tests were performed with or without the supportive cables in-place. Here, follows a selection of the most important aspects of the measured response that are presented and discussed. In Figure 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 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) [3]. 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

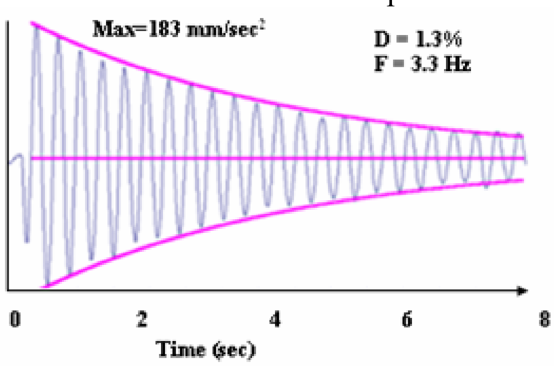


Figure 5a. low intensity pull-out test. (structure with cables and no extra mass)

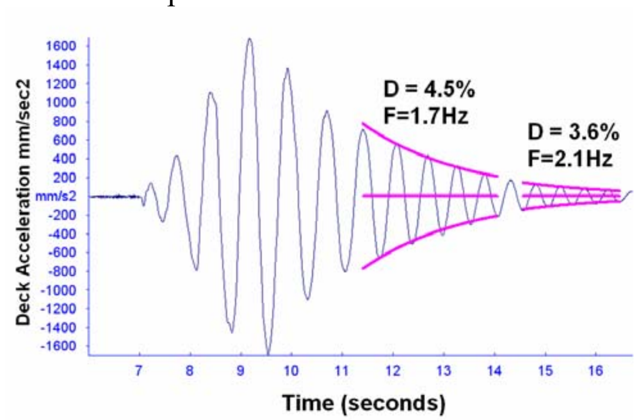


Figure 5b. low-to-medium intensity forced vibration test that produced the pier damage (structure with cables and extra mass)

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 (acc. of gravity)	
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 measurements is illustrated in Figures 6a and 6b, as was measured by the four pressure cells located at the foundation-soil interface near each corner of the foundation block (Figure 13).

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

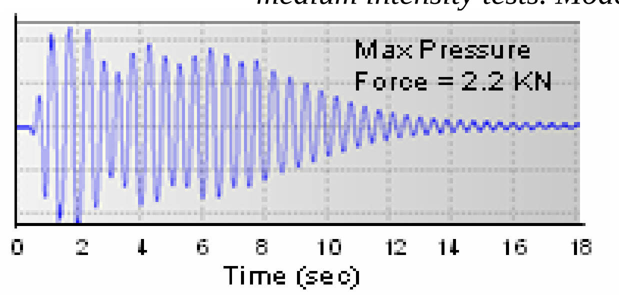


Figure 6a during the 1st test.

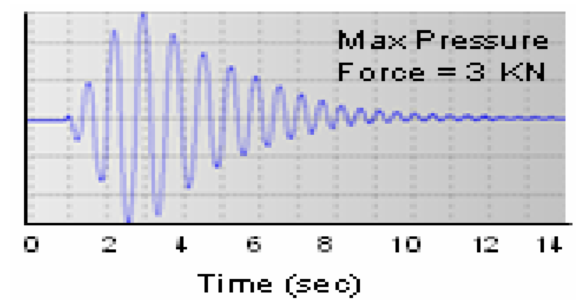


Figure 6b during the 2nd test

The 2nd test that produced the Pier damage

During this test the diagonal cables and struts were active on the model. Figure 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 (Figure 5a) the severity of the forced vibration test can be seen. The soil pressures recorded during this test are depicted in Figure 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 (Figure 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.

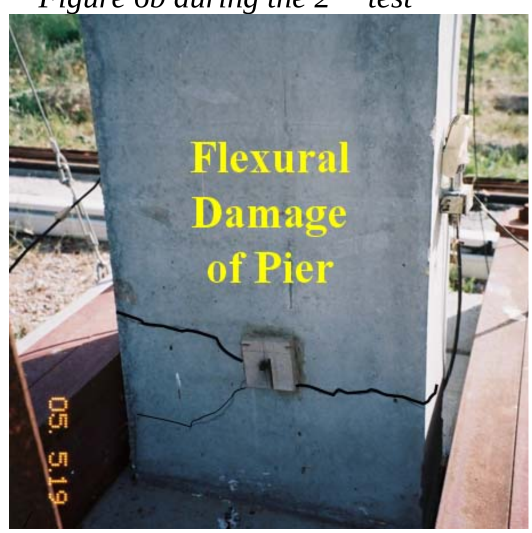


Figure 7: Flexural damage of the pier near its base after the 19th May 2005 experiment (cracking has been accentuated in this figure to become visible)

This test produced damage to the bridge pier model in the form depicted in Figure 7. The non-linear response of the bridge pier model is also noticeable in Figure 8, where the horizontal displacement at the middle of the concrete deck is plotted against the base shear force. As can be seen in Figure 8, the response becomes nonlinear above a base shear value approximately equal to 15KN (3.37kips).
Moreover, the damping ratio value of the decaying part of the response for this test was equal to 4.5% (f=1.7Hz) and 3.6% (f=2.1Hz), much larger than the one during the low intensity tests which was equal to 1.3% (Figures 5a and 5b). This may be attributed to both the cracking as well as to a possible increase in the damping contribution from the foundation – soil response; however, the later can not be simply extracted from the pressure measurements shown in figures 6a and 6b. An extensive study of the measured pier response is currently under way for the low-to-medium excitation tests, which may provide an insight to such contributions.

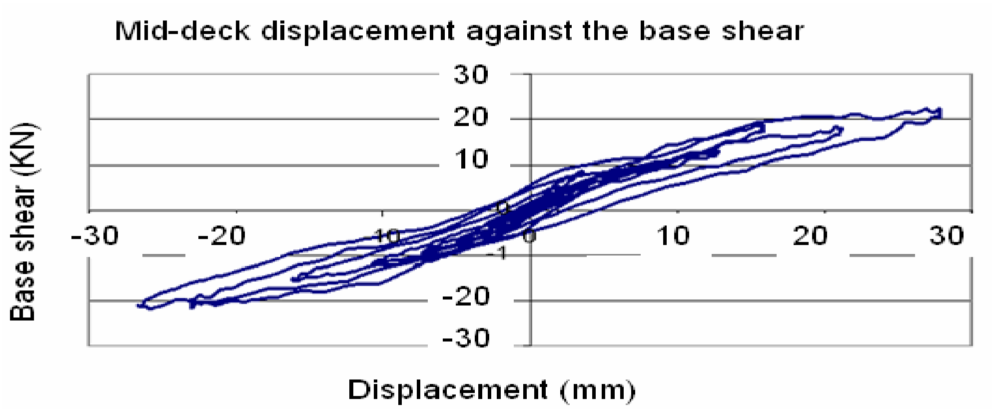


Figure 8: Base shear force - displacement response at the middle of the Deck 2nd low-to-medium intensity test, 19th May 2005). Model with cables and struts

Table 2: Summary of Measured Response

PRIOR TO CRACKING	With cables and struts	Without cables or struts
Out-of-plane	1.929 Hz	1.120 Hz
In-plane	2.800 Hz	2.600 Hz
AFTER CRACKING	**With cables and struts**	**Without cables or struts**
Out-of-plane	1.709 Hz	1.099 Hz
In-plane	2.539 Hz	2.246 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 behaviour 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 13)
- 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 [4].
- 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 [8]. 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. [5].

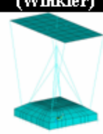
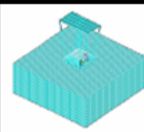
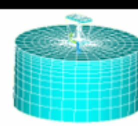
Currently, the optimum approximation (in terms of model complexity and degree of accuracy) of the above seven alternative FE numerical approaches is judged only on the agreement between measured and predicted pier response in terms of a) eigen-frequencies and eigen-modes and b) maximum amplitude of the low-intensity displacement and acceleration deck response.

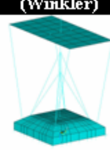
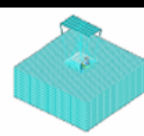
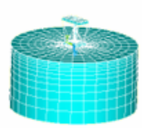
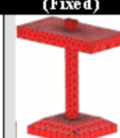
Numerical predictions of the Pier response prior to cracking

The properties of the materials (concrete, steel and soil), employed in the various numerical simulations for the pier model prior to cracking, were based on extensive laboratory and in-situ measurements [3]. In particular, the soil-related properties were determined from the measured response of the 6th story model structure that is placed nearby (Figure 1) as well as from the available in-situ cross-hole measurements specifically taken at the Euro-Test Site.

Table 3 summarizes the numerically predicted translational eigen-frequencies for the bridge pier model prior to cracking. The corresponding measured values are also listed at the same table. Reasonably good agreement can be seen between the observed and measured values. This comparison of the numerical and measured dynamic response of the bridge pier model prior to cracking is also shown by the plots of Figures 9 and 10. Figure 9 depicts the deck horizontal acceleration response of the pier during a low intensity pull out test performed on the 13th May 2004. This deck response was measured both in the in-plane and out-of-plane directions. During the 13th of May 2004 tests there was no extra mass at the deck of the pier and all cables and struts were active. Figure 10 includes similar results from a low intensity pull-out test performed on 20th of October 2004. The deck in-plane horizontal as well as the vertical acceleration responses, measured at the edge of the deck (x-x axis at symmetry), are depicted in this figure. Reasonably good agreement can also be seen between these measured and predicted results. 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 modelling 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 modelling 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).

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

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

	Measured	LUSAS	ANSYS (Winkler)	ANSYS (3D soil)	ANSYS (3D soil)	Wolf	ANSYS (Fixed)	Theoretical Solution (Fixed)
Out of Plane								
No extra Mass With cables	1.830 Hz	1.765 Hz	1.953 Hz	2.363 Hz	1.952 Hz	1.924 Hz	1.991 Hz	
Extra Mass Without cables	1.120 Hz	1.218 Hz	1.199 Hz		1.199 Hz	1.196 Hz	1.212 Hz	1.156 Hz
Extra Mass With cables	1.929 Hz	1.485 Hz	1.654 Hz	2.014 Hz	1.654 Hz	1.645 Hz	1.687 Hz	

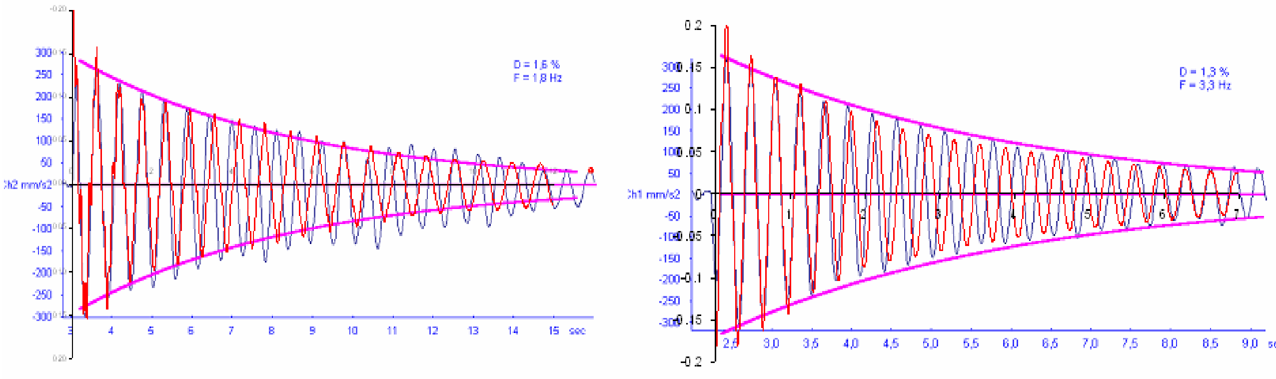


Figure 9: Low-intensity pull-out test 13th May 2004. Left: Deck horizontal acceleration response. Out-of-plane direction. Right: Deck horizontal acceleration response. In-plane direction (Numerical predictions in red color).

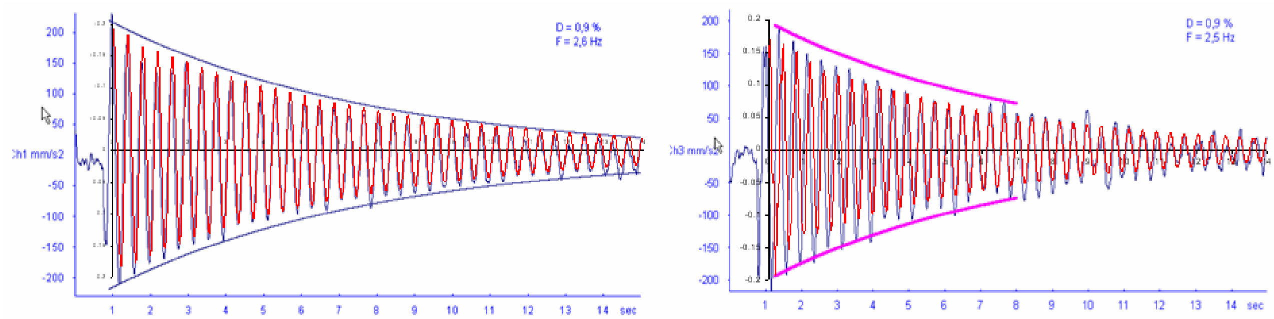


Figure 10: Low-intensity pull-out test 20th October 2004. Left: Deck horizontal acceleration response. In-plane direction. Right: Deck vertical acceleration response. (Numerical predictions in red color).

Numerical simulations of the cracking behaviour of the pier

In this case, the numerical simulation attempts to capture the bridge pier cracking behaviour, when a horizontal force of sufficient amplitude is introduced at the deck level (in-plane direction). Based on an experimental investigation, performed at the Laboratory with an identical bridge pier model, the amplitude of this horizontal force was gradually increased up to 10kN and then it was suddenly removed, thus introducing in this way a non-linear free vibration dynamic response of the pier. The cracking and crushing capabilities of ANSYS Finite Element software for simulating the non-linear concrete behaviour were utilised through a special purpose element (SOLID65) which can accommodate such non-linear material behaviour, based on a William-Warnke failure criterion (Figure 11). In this analysis, the soil was modeled by linear springs located at the soil-foundation interface. This simplification in simulating the soil flexibility was adopted in order to compensate for the complexity of the non-linear dynamic analysis of the superstructure. At the right hand side of Figure 12, the numerically predicted cracking pattern near the base of the pier is illustrated for this 10kN dynamic excitation. The fine small circles shown at the base of the pier (right hand side of Figure 12) indicate the location and orientation of the predicted cracking pattern. This analysis was numerically stable and the predicted cracks extend up to a height of 22 cm from the pier base having a mainly horizontal orientation. In this way, the predicted cracking pattern is in good agreement with the actual cracking behaviour of the pier illustrated at the left hand side of Figure 12. However, it must be pointed out that this actual cracking pattern was obtained from a forced vibration test, performed on the 19th May at the Test Site (see Section on low-to-medium intensity testing sequence).

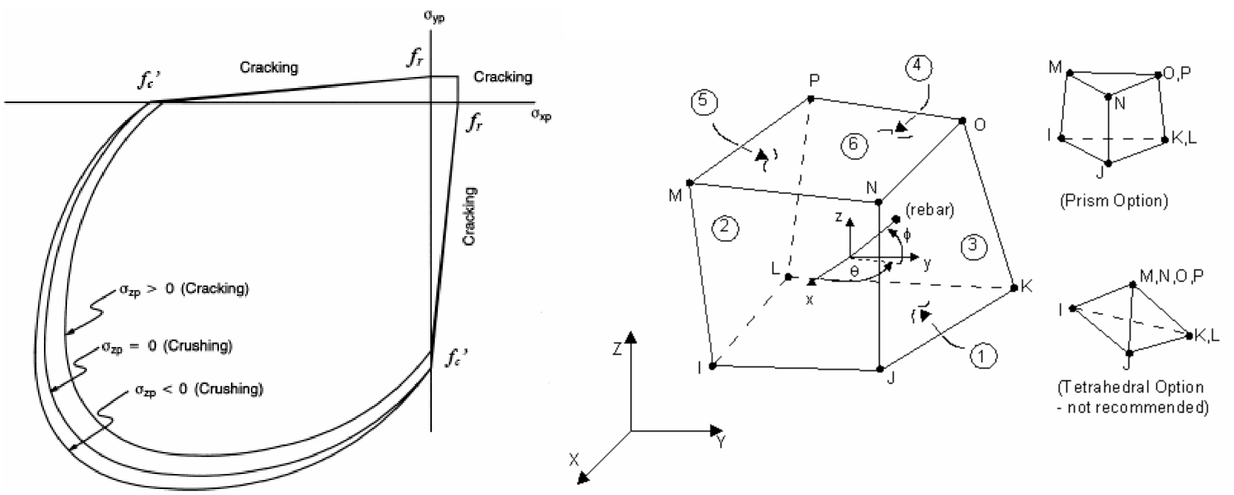


Figure 11: Failure criterion implemented into the SOLID65 special concrete element of ANSYS

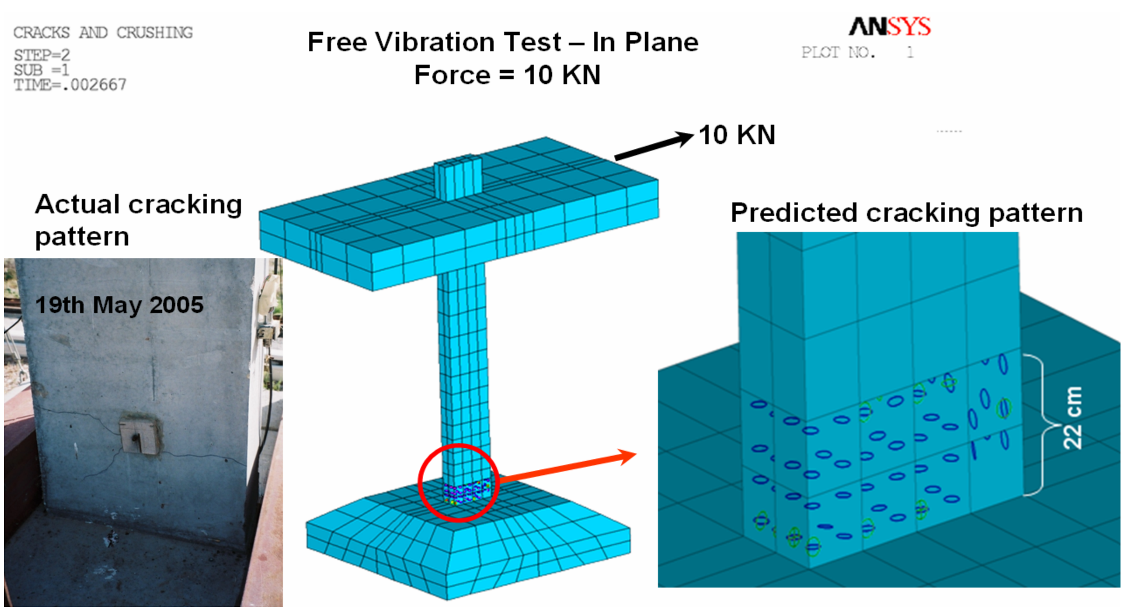


Figure 12: Numerical prediction of the pier non-linear response using the cracking and crashing capabilities of ANSYS for concrete

Numerical simulation of low-intensity tests after pier cracking

A second set of FE analyses has also been performed for the pull-out tests that followed the cracking of the pier during the 19th of May 2005 experiments. In this case, the complete soil volume was introduced under the foundation to account for the soil flexibity (Figure 13). Through sensitivity analyses, it was ensured that the outer soil-volume boundary conditions do not affect the numerically predicted response. In this case, the concrete cracking at the base of the pier was simulated in a simplified way, through an effective modulus of elasticity set equal to the 50% of the original value used for the uncracked pier. This lower modulus of elasticity value was used along the lowermost 200mm of the pier, based on the observed cracking pattern at the Test Site. This was sufficient to equivalently match the modified by 15% frequency of the cracked pier as it was measured during a specific test sequence (Table 2). The pressure cells, which were embedded under the foundation (see Section on the Dynamic Testing and Figure 13) were simulated using 2-D elements integrated within the solid mesh; the solid element properties correspond to the measured shear wave velocity of the uppermost two layers of soil profile at the particular location of the Test Site. An overview of this overall modelling approach is illustrated in Figure 13. On the top left hand side of this figure the predicted and measured translational in-plane eigen-frequency values are also listed. As can be seen, following the assumptions described above for this numerical simulation, a good agreement was achieved.

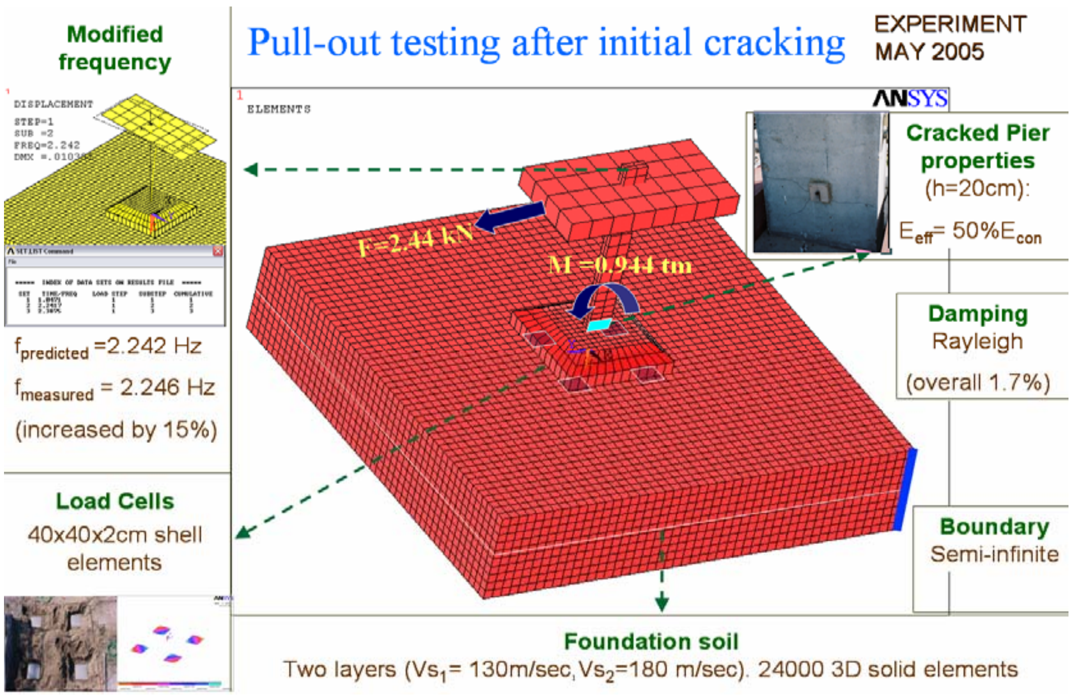


Figure 13: Overview of the FE model for the case of the post-cracking pull-out tests

The success in portraying the soil flexibility by the 3D volume, also used in this simulation, has been already validated by the good agreement between measured and predicted in-plane eigen-frequency values for the pier behaviour prior to cracking (see Table 3, rows 1 and 3). Thus, it can be concluded that the agreement between the measured and predicted in-plane eigen-frequency values for the pier after cracking, presented in this section, is mainly due to the way the degradation of the stiffness due to cracking was introduced (decrease by 50% of the original value for the Young Modulus of the concrete), as a simple way to approximately account for the observed cracking. Moreover, the height of the pier region where this concrete Young Modulus reduction was applied was also dictated from the observed cracking pattern. Consequently, this arbitrary decrease of the concrete Young Modulus was quite successful to match the observed lowering of the system stiffness. However, one should be cautious to generalise this conclusion by this one case and for this level of intensity (low intensity tests).

DISCUSSION OF RESULTS AND CONCLUSIVE REMARKS

Observed dynamic behaviour 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.

- In conclusion, 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.

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.

Observed dynamic behaviour during the low-to-medium intensity tests.

- The non-linear behaviour 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 [2]. 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 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 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 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).

Numerical predictions of the linear and non-linear dynamic response of the pier

- 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 for simulating the soil resulted in good agreement between numerical predictions and experimental measurements.
- The arbitrary decrease by 50% of the original value for the Young Modulus of the concrete in order to match the observed lowering of the system stiffness after cracking was reasonably successful. However, one should be cautious to generalise this conclusion by this one case and for this level of intensity (low intensity tests).
- The numerically predicted cracking pattern obtained by the described non-linear numerical simulation is in good agreement with the actual cracking behaviour of the pier.

ACKNOWLEDGMENTS

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