18th International Conference on Structural Mechanics in Reactor Technology (SMiRT 18)
Beijing, China, August 7-12, 2005
SMiRT18-K17-4

CYCLIC AND DYNAMIC RESPONSE OF A BRIDGE PIER MODEL LOCATED AT THE VOLVI EUROPEAN TEST SITE IN GREECE

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ABSTRACT

This paper presents results of the measured and predicted response of a bridge pier model-structures which has been erected at the Volvi-Greece European Test Site for Earthquake Engineering. The disadvantage at the Test Site is that one is unable to produce significant in-situ levels of ground motion, when desired, as can be generated by an earthquake simulator. However, one is having the advantage at the Test Site of realistic foundations conditions, which are present for model structures that are built there and are supported on the soft soil deposits in-situ. The current extension of the in-situ facility includes the possibility of subjecting large-scale model structures to low to 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 model with masonry infills b) A single bridge pier model, built for the purposes of the currently running Euro-Risk program, which is supported by the European Union. The variation of the dynamic characteristics of the 6-story 1/3 3-D frame model structure was measured over a period of ten years. So far, only one earthquake of moderate intensity has subjected the 6-story model structure to seismic loads and excited the permanent instrumentation system. The main objective of the recent tests, which are partially presented in this paper and involve the bridge pier model, is to include influences on the dynamic structural response arising from the flexible foundation support conditions. The bridge pier model was initially studied at the laboratory under cyclic horizontal loads that were applied simultaneously with vertical forces. Thus, the cyclic post-elastic behavior of this bridge pier model was recorded at the laboratory. Next, a series of low intensity excitations were performed at the test site over a period of two years. During this period, the pier model structure was in various configurations that included the presence or not of diagonal cables between the foundation and the deck as well the presence or not of extra mass at the deck apart from the concrete slab. 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 the pier bridge model, which are presented here in a summary form. Moreover, an extensive numerical simulation of the response was also performed, which includes the flexibility of the foundation. Good agreement can be seen when the measured values are compared with the corresponding numerical predictions.

Keywords: Bridge-Pier Model, Soil-Foundation Interaction, Measurements and Numerical Predictions, European Test-Site.

1. INTRODUCTION

1.1. Description of the Test Site.

The European test site is located at the Mygdonian valley in Greece (figure 1), which is bounded from North and South at the villages of Stivos and Profitis by sloping rocky formations, whereas thick layers of alluvia deposits form the central part. Figure 2 shows a transverse cross-section of the valley at its narrowest part. This location was selected as the best position to deploy a considerable number of accelerographs in order to record the distribution of the prototype earthquake ground motion along the valley and to identify influences arising from the variation of the stiffness of the various soil-layers. For this purpose, some of these instruments are located on the stiff rocky slopes at the ends of the valley, whereas the rest are placed at various locations within this section of the valley, as depicted in figure 2. In this figure, the various soil layers are also shown together with the values of their respective shear wave velocity (Vs) as found by Pitilakis (1995). As can be seen, at the central part of the valley, a number of accelerographs have been concentrated, both at the surface as well as at a certain depth in the alluvium soil layer. It is here that the model structures have been built.

The first model structure is a 6-story reinforced concrete frame model building with masonry infills (figure 2). This model structure was built at the Volvi test site in 1994. It is similar to a structure, with approximately the same dimensions but without masonry infills, which has been studied by Okada (1992) and co-workers at Chiba Field Station of the Institute of Industrial Science of the University of Tokyo in Japan. The second model structure is a single bridge pier and its foundation block (Pier B1). This bridge pier model was recently built (2004) and is similar to corresponding bridge piers that were tested at ELSA laboratories of the European Joint Research Center (Pinto 1996). The lay out at the Test Site is depicted in figure 3; apart from the structural models and the network of strong motion accelerographs this facility also includes a crane, a store house and a power generated hydraulic system (see also figure 4).

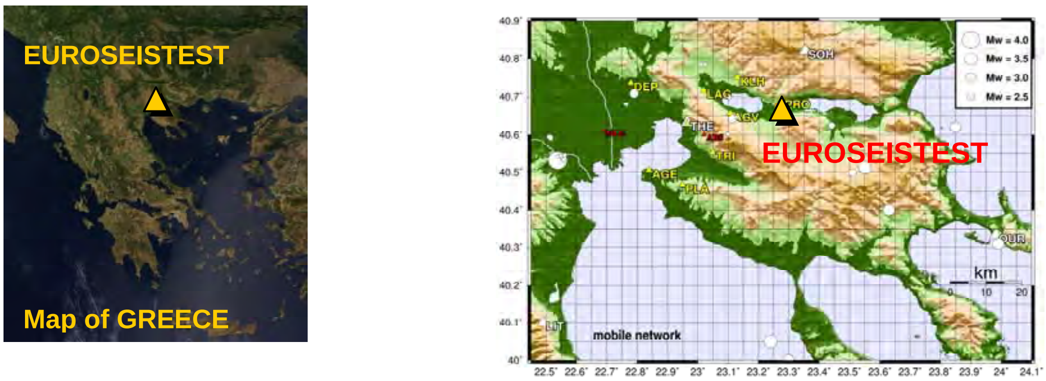


Figure 1. Location of the European Test Site at the Mygnonian Valley of Thessaloniki - Greece

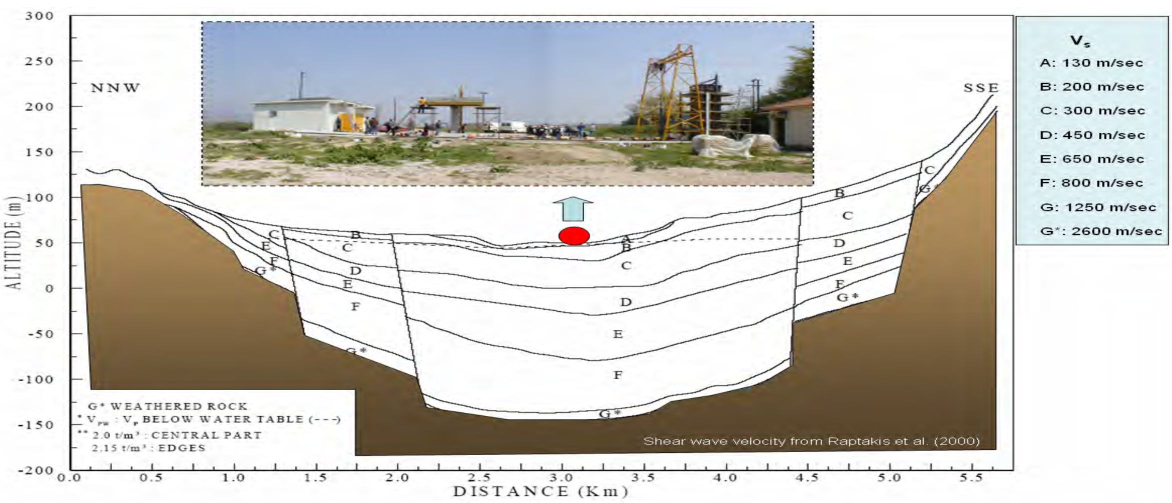


Figure 2. A transverse cross-section of the Volvi-valley at its narrowest part together with the European test-site

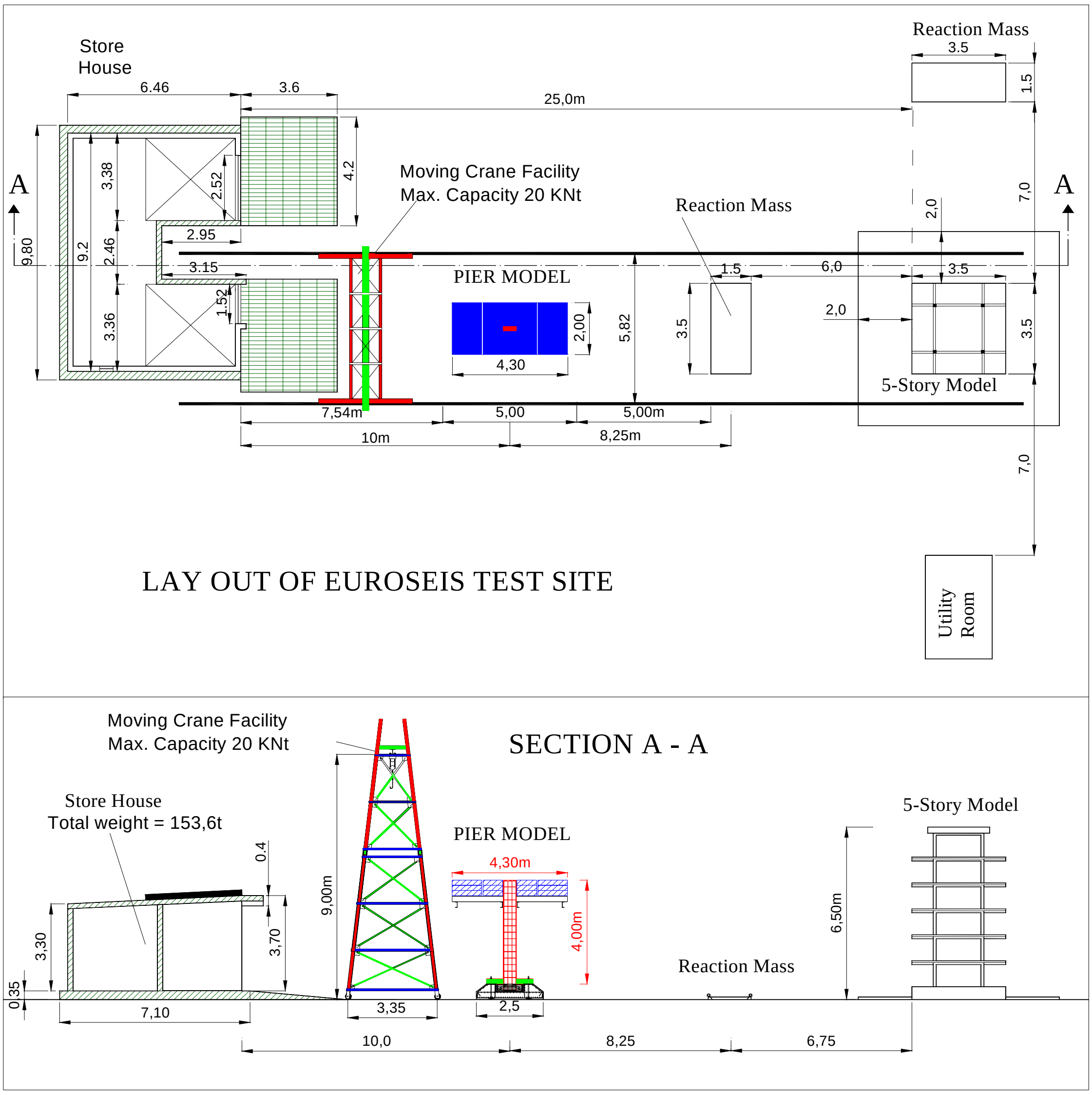


Figure 3. Lay-Out of the model structures and the facilities at the European test-site (Volvi-Greece)

Figure 4. Experiments with model structures at the European Test Site

4. THE SINGLE BRIDGE PIER MODEL

This structure is a small-scale representation of a single bridge pier. 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 pre-normative research of Eurocode 8 a series of pseudo-dynamic tests on 1:2.5 scaled bridge piers were conducted at ELSA Laboratory 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 cross-section of the ELSA models was a hollow rectangular cross-section, more closely representing the cross-section 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 European test site are quite similar. The cross-section of the Volvi model pier, together with the reinforcing details, is shown in figures 5 and 11. 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.

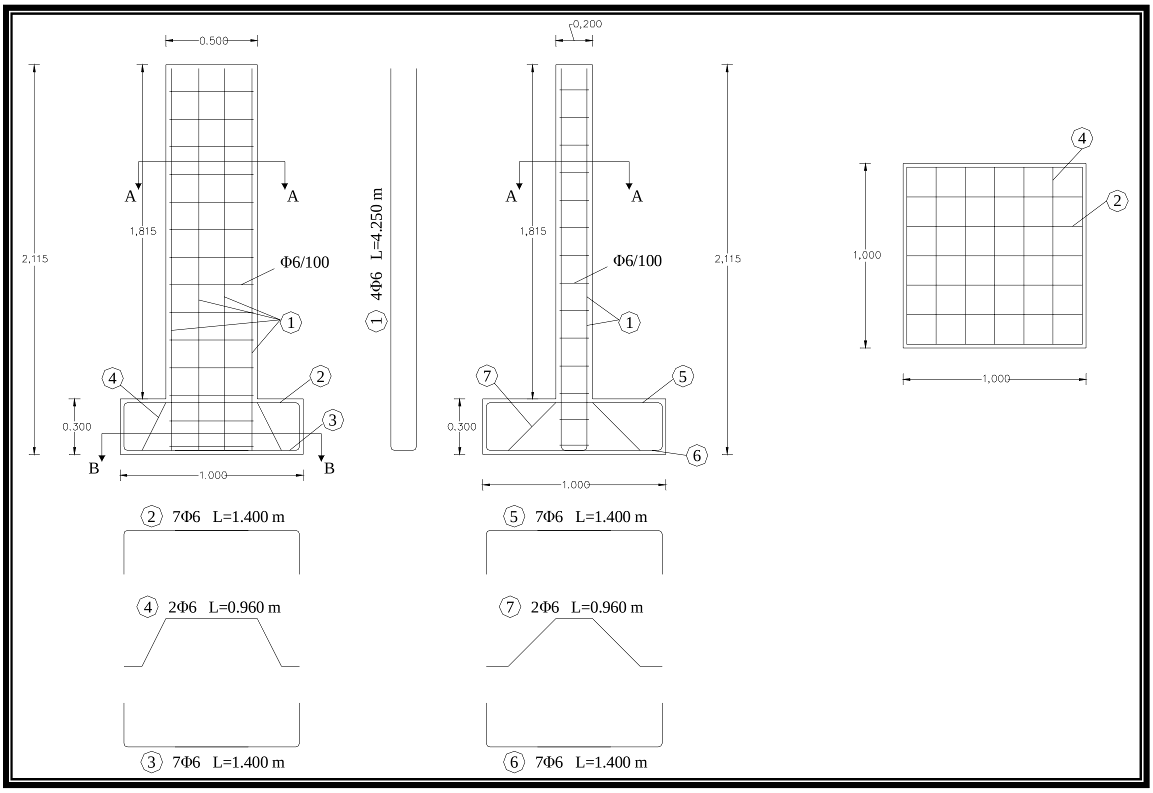


Figure 5. Structural details of Bridge Pier Model (see also Manos 2004)

3.1. Configurations of the Volvi model pier.

Two bridge pier models with their foundation blocks have been built at the Laboratory of Strength of Materials and Structures of Aristotle University, according to the final design (Model Pier A1 and A2). These models, A1 and A2, are identical in all reinforcing details, cross-sectional dimensions and material properties to the model pier, which has been built at the Volvi test site. They differ only in the following:

The height of piers A1 and A2, tested at the laboratory, is 1800mm whereas the height of the Volvi pier (B1) is 4000mm. The height is measured from the top of the foundation block to the top of the pier.

The foundation block of piers A1 and A2 is 1m x 1m in plan with a thickness of 0.3m. On the contrary, the foundation block of the Volvi pier is a properly formed single footing, having a bottom plan with dimensions 2.48m x 2.48m and a top plan 1.62m x 1.62m. The total thickness of this foundation block is 0.6m.

Models A1 and A2 served the purpose of ensuring the strength capacity of the Volvi model pier structure. 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 Volvi pier have been introduced in order to accommodate these models (A1 and A2) within the space limitation of the strong reaction frame at the laboratory (figure 6).

Figure 6. Models A1 and A2 tested at the laboratory and model pier B1 ready for transportation at the Test Site

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

Model Code Name	Dimensions		Foundation (m)	Measured Concrete Strength (at the bottom cross-section)		Measured Steel Strength	
	Height (m)	Cross-section (cm)		28days	14months	Yield (Mpa)	Ultimum (Mpa)
A1	1.8	20 x 50	1.0 x 1.0 x 0.3	26.0Mpa	--	344.8	470.9
A2	1.8	20 x 50	1.0 x 1.0 x 0.3	26.0Mpa	--	344.8	470.9
B1	4.0	20 x 50	2.5 x 2.5 x 0.6	28.0Mpa	37.0Mpa	344.8	470.9
B2 (Rep)	4.0	20 x 50	2.5 x 2.5 x 0.6	25.0Mpa	34.0Mpa	344.8	470.9

A model pier structure (Model Pier B1, right end of figure 6) with a height of 4000mm and a foundation block 1m x 1m x 0.3m was also built at the Laboratory. Pier B2 represents an identical to B1 pier, which was damaged during transportation and then repaired. Pier B1 was transported intact at the Test Site. It has almost identical material characteristics and construction details for the concrete and reinforcing bar parts with those of models A1 and A2 (left and center part of figure 6). An extended foundation footing of 2.5m x 2.5m x 0.6m, built separately from the model pier B1, has a hollow part in its top with the appropriate dimensions so that the 1m x 1m x 0.3m block of pier B1 could be totally encased. This extended foundation was also built at the Laboratory of Strength of Materials of Aristotle University and was transported to the test site. The main advantage of the composite foundation scenario is that building time was gained in this way during the bad weather of the winter months. Furthermore, the compatibility of the various fixtures for attaching the constructed part of Model Pier B1 with that of the extended foundation was tested inside the laboratory. Finally, the damage and subsequent failure of pier B1 will be dealt with in the future in-situ by means of the total replacement of the damaged upper part of this model pier (B1) with its foundation block of 1m x 1m x 0.3m with a new identical part, which could again be totally encased in the hollow top part of the extended foundation footing (see figures 10a and 10b).

- A steel platform has been designed and constructed that was rigidly attached at the top of the Volvi pier B1. This steel platform represents the deck of the pier together with concrete slabs that were rigidly attached to this steel platform. The steel platform and the concrete slabs provide the necessary weight in order to apply the desired level of axial load on the cross-sections of the pier. In addition, this concentrated mass generates horizontal forces of desired amplitude at this "deck" level. This is utilized during the artificial excitation tests as well as in the eventuality of an earthquake event that will excite the bridge pier model structure at the test site. The R.C. slabs which are fixed at the model B1 pier bridge deck, have been constructed at the laboratory and have been transported to the test site. They 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. The total weight of the steel platform and the R.C. slabs is approximately 9 tones. The axial load of the pier cross-section adjacent to the foundation at the bottom of the pier is approximately 100KN. The overall dimensions of the deck are 2000mm x 4000mm. The model pier is shown in figures 7a and 7b as it has been erected two years ago at the Test Site. In these figures the in-plane and out-of-plane directions of this model structure are also indicated. The in plane direction coincides with the long side of the cross-section of the pier and the deck; the out-of-plane direction coincides with the narrow side of the cross section of the pier and the deck (see also figures 10a and 10b).

Figure 7a and b. Erection of the model pier at the Test Site.

4. OBTAINED RESULTS FROM PIERS A1 AND A2 TESTED AT THE LABORATORY

Pier models A1 and A2 have been tested at the strong reaction frame at the laboratory. They were subjected to combined cyclic horizontal and vertical loads. The horizontal load was applied through a predetermined cyclic variation of the applied horizontal displacement at a location 1.42m from the bottom cross-section of the pier. This cyclic displacement was applied with a progressively increasing amplitude, up to a maximum amplitude of 20mm, with consecutive sinusoidal cycles having a frequency of 1Hz. The aim was to keep the vertical load constant approximately at 95KN during the application of the horizontal cyclic displacements. For this purpose, two different schemes were tried at the laboratory. In the first scheme, which was used for model A1, two one-way active hydraulic jacks without any electronic control were utilized, together with a system of accumulators. However, this scheme did not successfully maintain the vertical load constant at its predetermined level. 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 scheme of applying the vertical load utilized a two-way hydraulic actuator electronically controlled. This was used in model A2 as well as in all subsequent tests with the repaired piers A1 and A2. Again, a variation of the vertical load could be observed during the cyclic variation of the horizontal displacement at the top of the pier. However, this variation was of relatively smaller amplitude than the corresponding variation observed during the

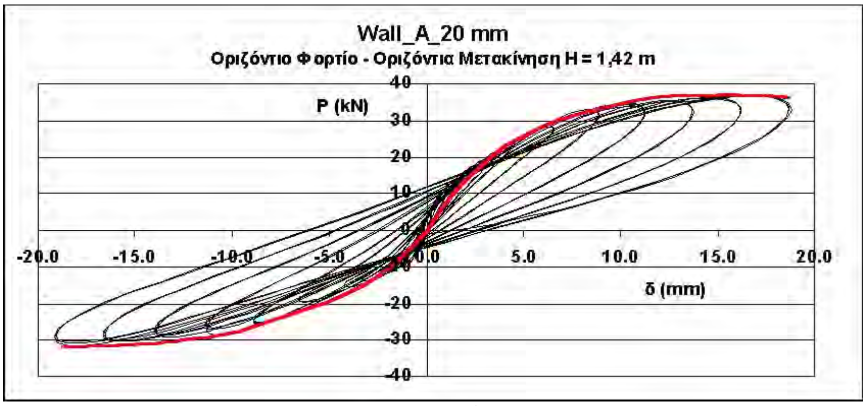
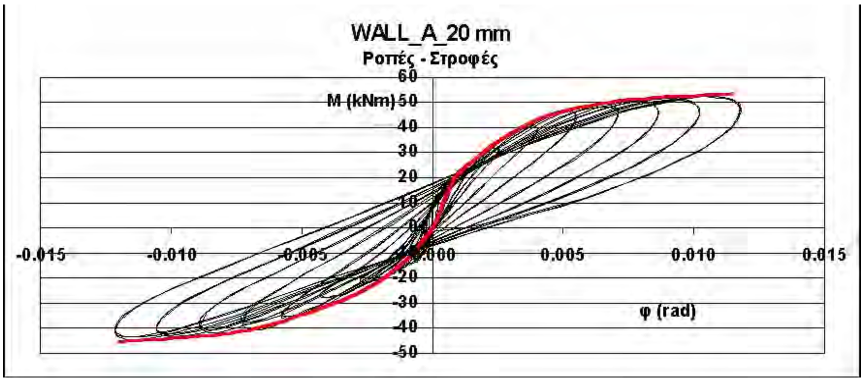


Figure 8a Cyclic response in terms of horizontal load – horizontal displacement at the top of the pier



tests with pier A1. The initial level of axial compressive load was equal to 95KN.

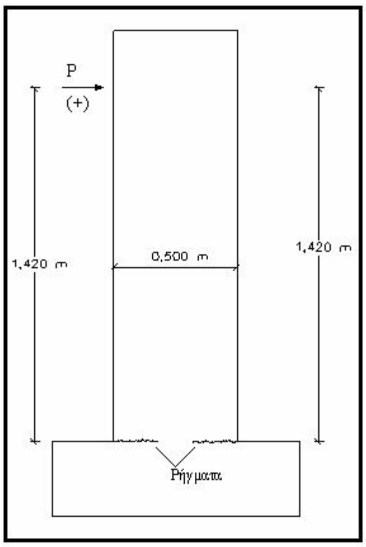


Figure 9. Observed damage, pier A2

Figures 8a and 8b depict the Pier A2 obtained cyclic response. The top plot (8a) is the observed behavior in terms of the variation of the applied top horizontal cyclic displacement versus the horizontal load sustained by the pier. The bottom plot (8b) is the variation of the bending moment that develops at the bottom cross-section of the pier versus the rotation at this cross-section. Finally, figure 9 depicts the corresponding flexural damage that developed at the bottom cross-section of the pier A2. Both the measured load-deformation cyclic response together with the observed damage for pier A2 demonstrates that the non-linear flexural response at the bottom of the pier is the predominant mode of response for this structure. This was one of the objectives that was aimed at the preliminary and final design stage, which was successfully fulfilled. As a result it is expected that the model pier B1 that has been erected at the European test site will have similar levels of strength and predominant mode of non-linear response with the ones observed at the laboratory during the sequence of cyclic tests for piers A1 and A2.

5. MEASURED RESPONSE OF THE BRIDGE PIER MODEL AT THE TEST SITE

The geometry of this model is depicted in figures 10a and 10b. In figure 10a the model structure is without diagonal cables whereas in figure 10b diagonal cables were added to connect the corners of the deck with the corners of the foundation block. This was a precaution taken in order to avoid premature damage of the model structure. The scaffolding that is shown in both these figures was placed for safety purposes and it was removed when this pier was tested; thus the scaffolding did not participate in the structural response.

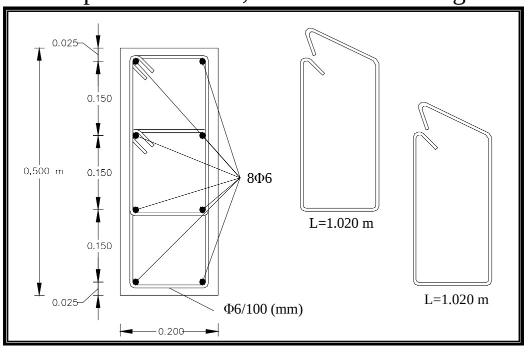


Figure 11. Cross section of the model pier.

A very simple way to excite this model structure was to use a system of cables in order to displace the deck in a controlled way and then to suddenly release it, thus introducing free-vibration conditions to the model. Instrumentation was provided that could measure the acceleration and displacement response of the deck, the pier and the foundation block. This type of excitation was introduced in two different directions. The x-x direction, which corresponds to the strong in-plane direction of the pier (cross-section A-A, figures 10, 11) and the y-y direction, which is the weak out-of-plane direction of the pier (cross-section B-B, figures 10, 11).

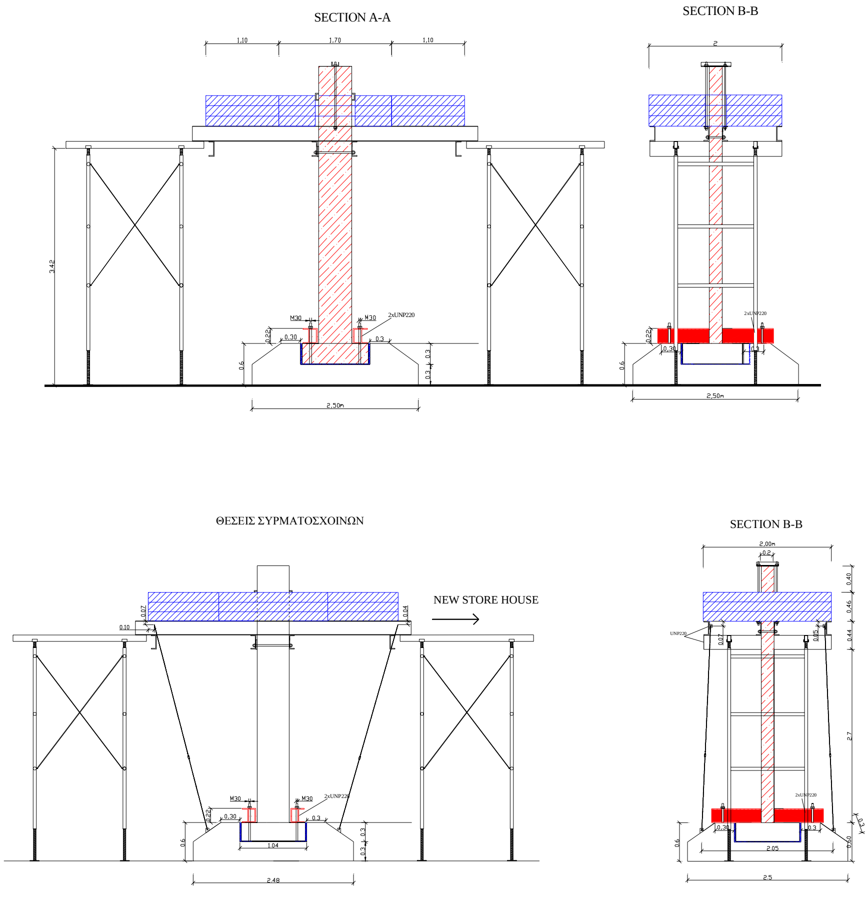


Figure 10a and 10b. Model pier without and with diagonal cables

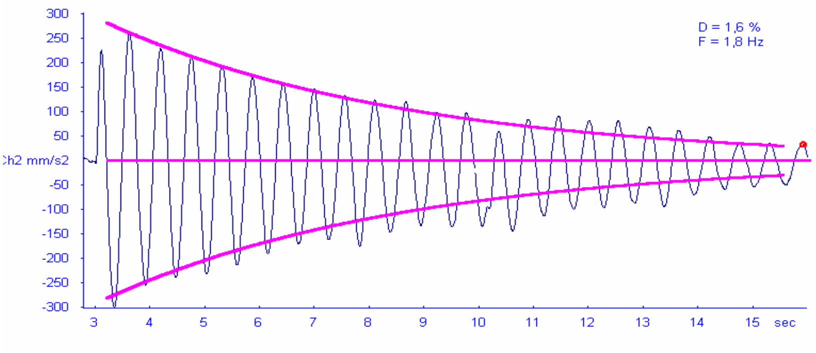


Figure 12a. Deck Response from Out-of-Plane Pull-Out Test   Structure with cables and no extra mass 13th May 2005 (the measurements must be multiplied by 0.5)

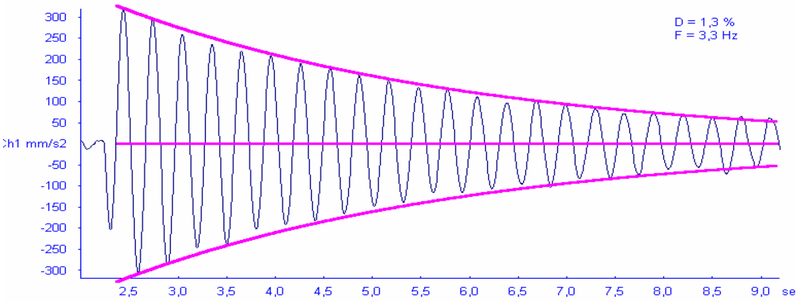


Figure12b. Deck Response from In-Plane Pull-Out Test , Structure with cables and no extra mass 13th May 2005 (the measurements must be multiplied by 0.5)

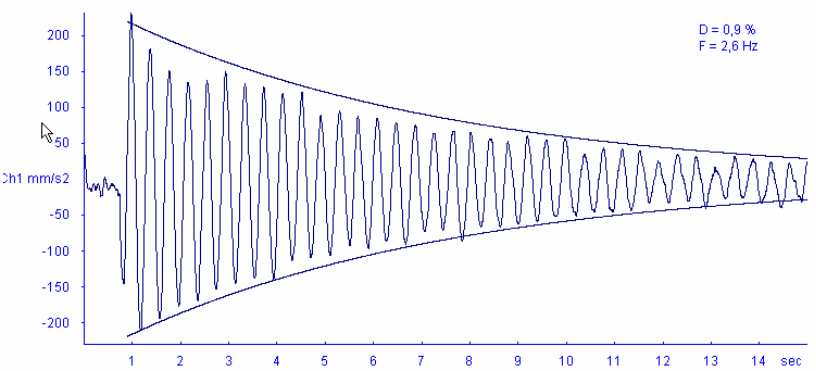


Figure 13a. Deck In-Plane Horizontal Response Pull-out Test x-x, Structure with lead mass and no cables, 20th October 2004

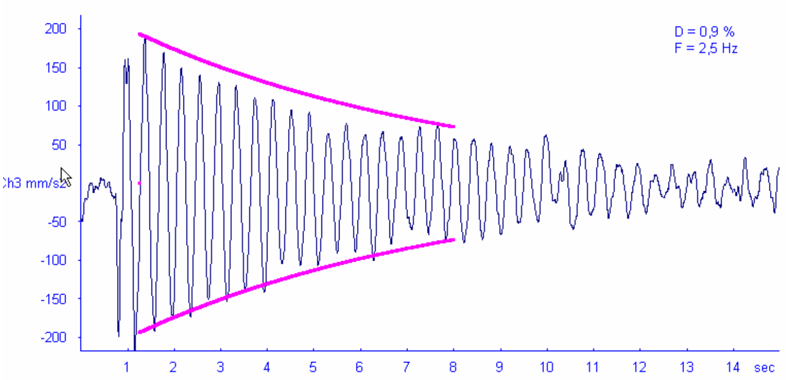


Figure 13b. Deck Rocking Vertical Response, Pull-out Test x-x, Structure with lead mass and no cables, 20th October 2004.

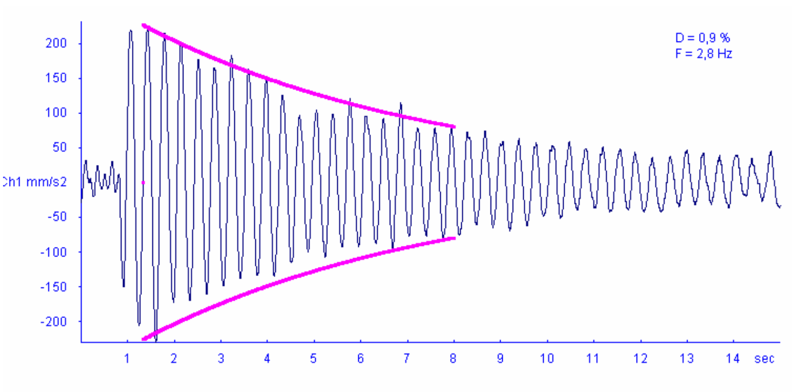


Figure 14a. Deck In-Plane Horizontal Response, Pull-out Test x-x, Deck with lead mass+A and cables, 21st October 2004

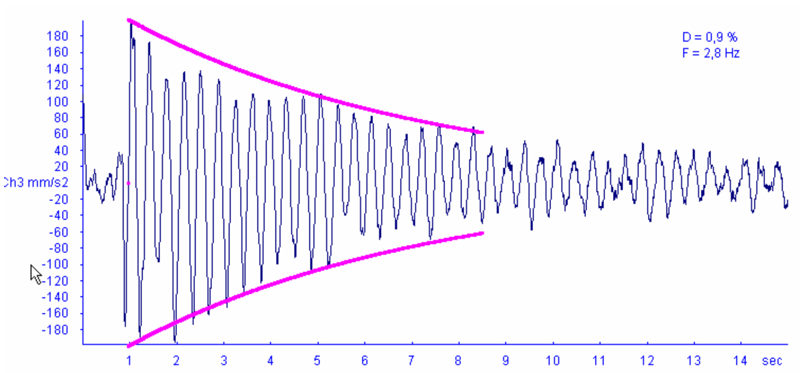


Figure 14b. Deck Rocking Vertical Response, Pull-out Test x-x, Deck with lead mass+A and cables, 21st October 2004

Table 2. Summary of Measured Response		Pull- Out Tests x-x and y-y, 6th April Structure with no Extra Mass and Cables		
		From FFT	Peak (Time history) in g (acc. of gravity)	
Channel No.	Name of Resp.	Frequency Hz	Max	Min
Ch11 x-x	Accx-x (Deck)	3.29	0.01826	-0.01745
Ch12 y-y	Accy-y (Deck)	1.83	0.01389	-0.01493

Figures 12 to 14 depict the measured response of the deck of the bridge pier model for various configurations of the structure as it was recorded during a series of pull-out tests. Figure 12a depicts the out-of-plane deck horizontal acceleration response when the excitation was an out-of-plane y-y pull-out test. Similarly, figure 12b depicts the in-plane deck horizontal acceleration response when the excitation was an in-plane x-x pull-out test. These two tests were performed with the pier model having no extra mass at its deck and with the diagonal cables being present. Figure 13a depicts the in-plane deck horizontal acceleration response when the excitation was an in-plane x-x pull-out test. Similarly, figure 13b depicts the rocking deck vertical acceleration response when the excitation was again in-plane x-x pull-out test. These two tests were performed with the pier model having extra mass at its deck and no diagonal cables. Figure 14a depicts the in-plane deck horizontal acceleration response when the excitation was an in-plane x-x pull-out test. Similarly, figure 14b depicts the rocking deck vertical acceleration response when the excitation was again in-plane x-x pull-out test. These two tests were performed with the pier model having extra mass+A at its deck and with the diagonal cables being present. Finally, Table 2 depicts basic response quantities as they were measured during similar pull-out tests in the in-plane (x-x) and the out-of-plane (y-y) direction. The measured horizontal deck acceleration response is listed in this table together with the dominant frequency of this response, as it was found from analyzing the measured signals in the frequency domain.

6. NUMERICALLY PREDICTED RESPONSE OF THE BRIDGE PIER MODEL AT THE TEST SITE

In what follows, the bridge pier model at the Test – Site was numerically simulated, assuming elastic behavior of all its parts as well as assuming flexible supports at the foundation-soil interface. The flexibility of the supports was based on measurements of the response of the 6th story model structure that is placed nearby and it was studied extensively during the past 10 years (Manos 1997, 1998, 2000). The numerical simulation was based on the following assumptions:

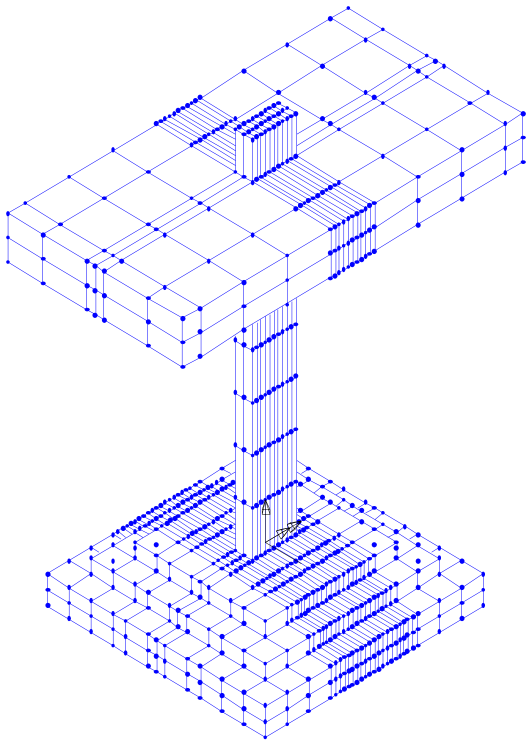


Figure 15. Employed mess in the numerical simulation.

The elastic modulus of all the reinforced concrete parts of the pier and the extended foundation block was assumed equal to 32Gpa. The distribution and position of the masses of the various parts of the pier model, such as the foundation, the pier, the longitudinal and transverse steel girders and the concrete slabs forming the deck, were based on exact measurements of the corresponding weights and dimensions of this model. The pier and the deck of this bridge model during some of the tests were linked to the foundation by a number of steel cables. This was done in order to avoid any premature accidental damage of this model structure. Although these cables were not altering in any significant way the total structural stiffness they were included in the numerical simulation. The employed mesh is depicted in figure 15. The model structure was simulated with two distinct boundary conditions at the foundation-soil interface. The first conditions assumed absolute fixity at this interface whereas the second condition attempted to simulate a flexible soil-foundation interface by utilizing a number of springs with stiffness properties that were taken from a previous experimental study with the 6th story structure (Manos 2000).

The results of this numerical investigation are depicted in figures 16 to 18 . The employed mesh together with the obtained first three eigen-modes and eigen-frequencies, are depicted in these figures. In all the presented simulations the model pier was assumed to have flexible foundation conditions. The following table summarizes these results. Together with the predicted eigen-frequencies the corresponding measured values are also listed in this table

Table 3. Summary of predicted and measured response

Structural Configuration	Measured Response (eigen-frequency)		Predicted Response (eigen-frequency)	
	In-plane (Hz)	Out-of-plane (Hz)	In-plane (Hz)	Out-of-plane (Hz)
No extra Mass With cables	3.29	1.83	3.315	1.765
Extra Mass Without cables	2.60	-	2.681	1.218
Extra Mass With cables	2.80	1.929	2.815	1.485

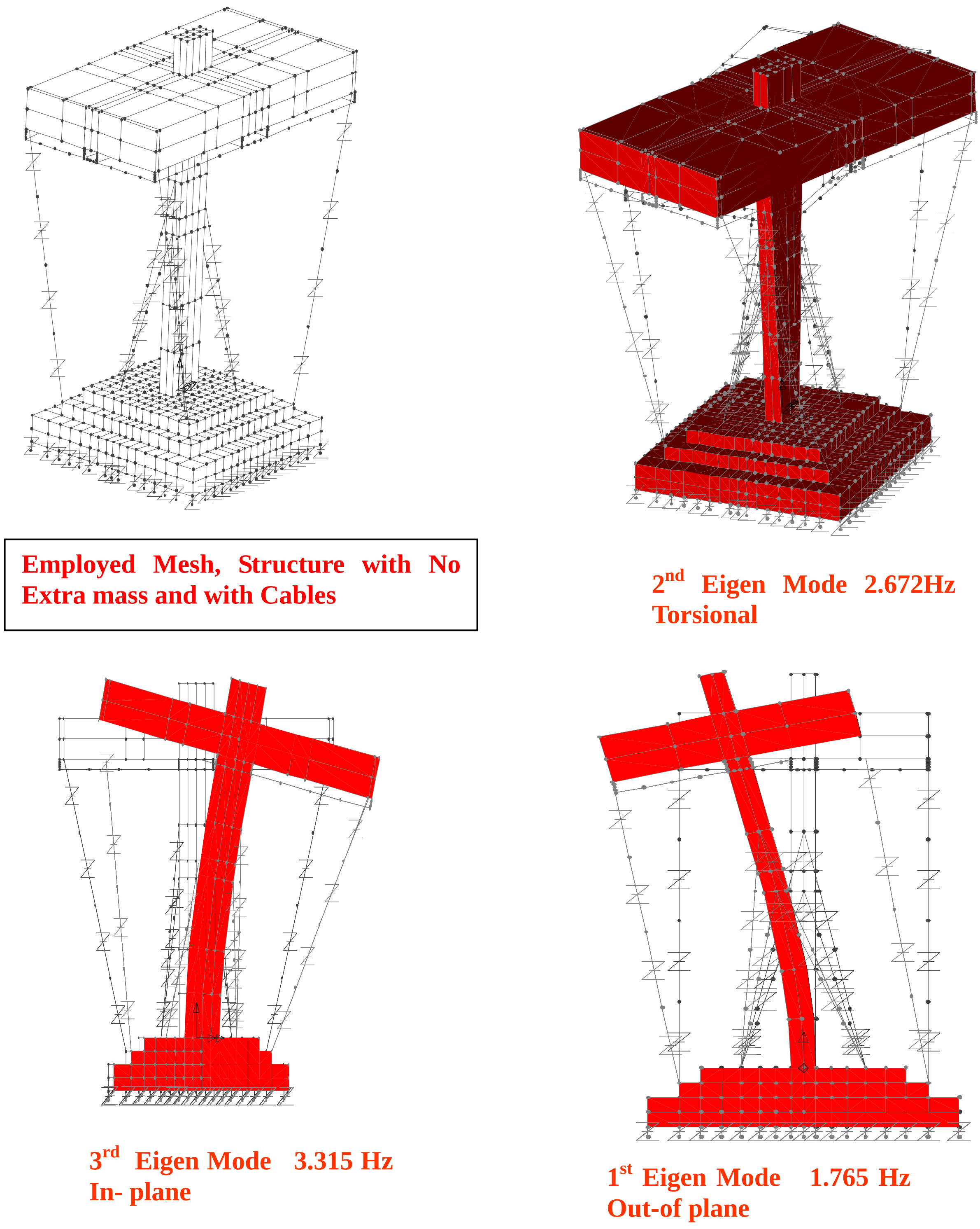


Figure 16. Eigen-modes and eigen- frequencies for model structure without extra mass and with diagonal cables. Flexible foundation conditions.

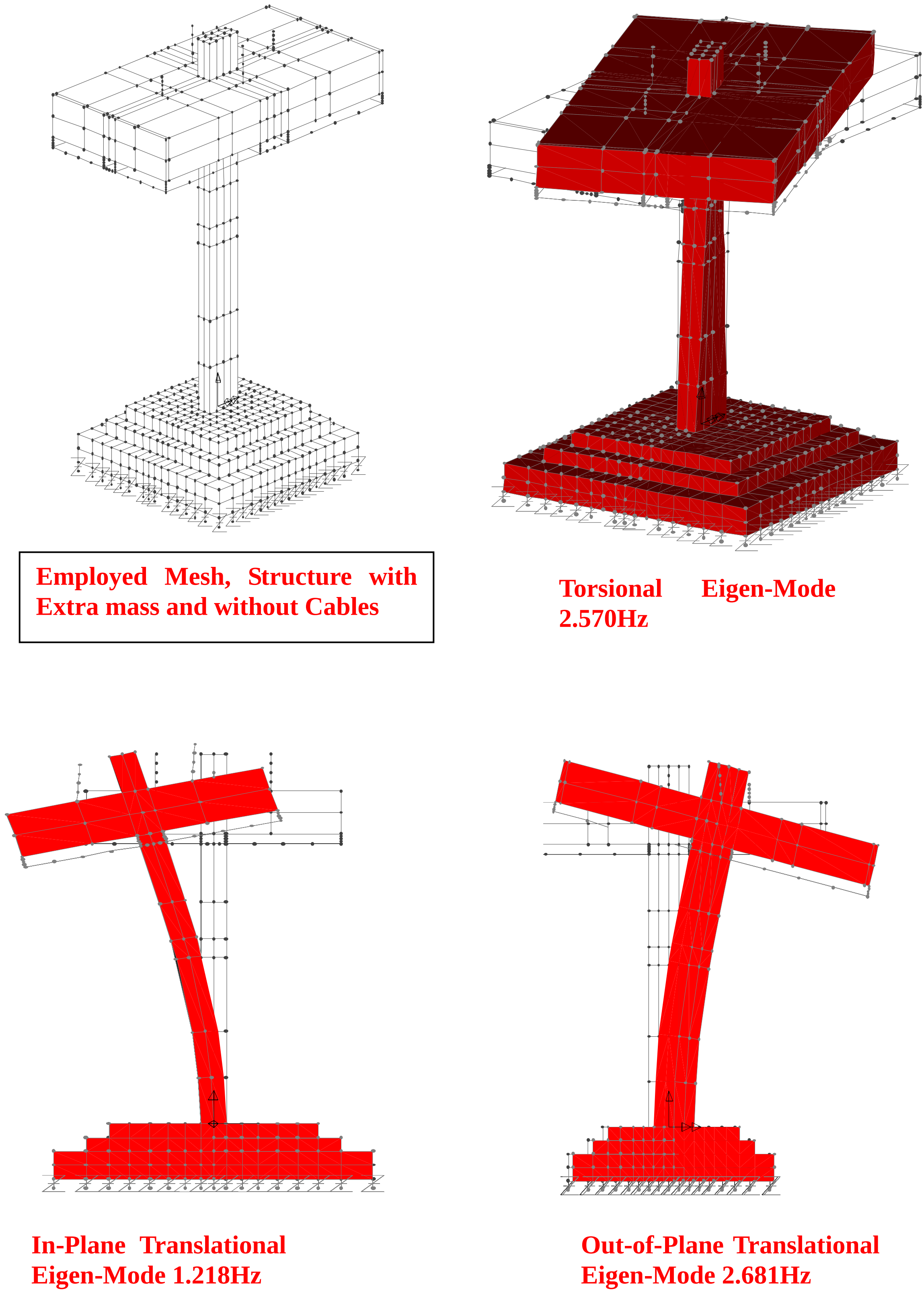


Figure 17. Eigen-modes and eigen- frequencies for model structure with extra mass and without diagonal cables. Flexible foundation conditions

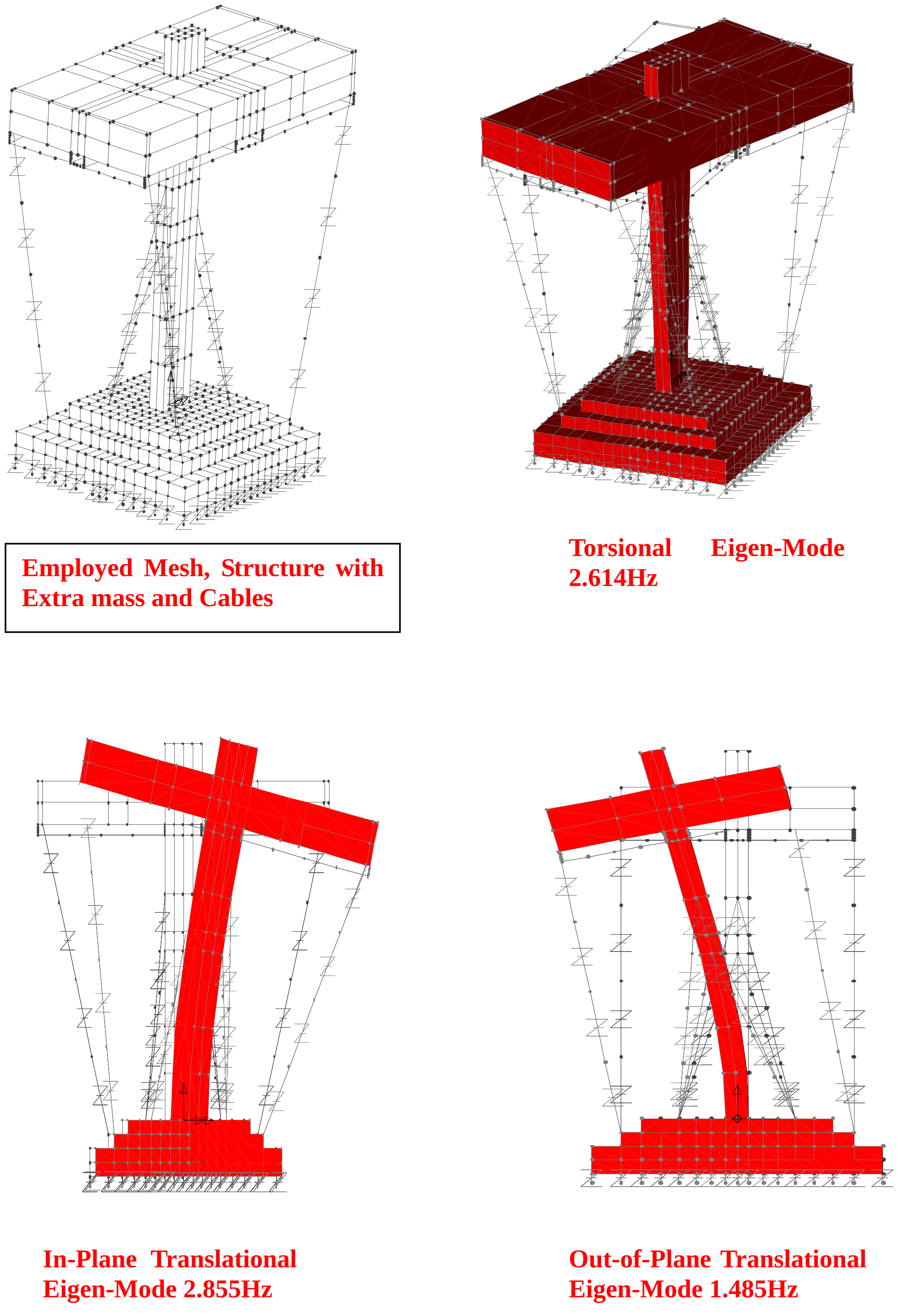


Figure 18. Eigen-modes and eigen- frequencies for model structure with extra mass and with diagonal cables. Flexible foundation conditions.

7. Conclusive remarks

- The non-linear behavior of the studied bridge pier moel has already been observed on replica models, which were built for this purpose and tested at the laboratory. It was found that the predominantly flexural mode of response and the corresponding flexural type of damage, which was concentrated at the bottom of the model pier, was in agreement with the predicted behavior aimed at, according to the design of this model structure.
- The measured eigen-modes and eigen-frequencies in the x-x in-plane and y-y out-of-plane directions is in the range of 1.2Hz to 3.2Hz, which includes influences arising from the flexible foundation conditions.
- The measured damping values, extracted from the decay of the free vibration response, is in the range of 0.9% to 1.6% for such low-intensity type of excitation.
- The numerical simulation of the eigen-modes and eigen-frequencies, including the flexible foundation conditions, are in good agreement with the corresponding observed values, as they resulted from the the low-intensity tests with this bridge pier model structure, that includes the soil-foundation interaction. This agreement is better in the in-plane than in the out-of-plane direction.
- The European test site at Volvi was extended to include a network of numerous strong motion accelerographs as well as a crane, a store house and a power generated hydraulic system. This facility is now being utilized for tests with a single bridge pier model, focusing on the influence that the soil-foundation interaction will have on the response of this pier.

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Acknowledgements.

This work has been partially supported by the European Union, Projects ENV.5V-CT93-0281, ENV.4-CT.96-0255 and currently by EVG1-CT-2001-00040. The partial financial assistance of the Directorate XII of the European Commission, is here gratefully acknowledged.

