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SEISMIC PERFORMANCE OF BELL TOWERS IN KEFALONIA, GREECE DURING THE 2014 EARTHQUAKE SEQUENCE

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Abstract. *Bell towers are structures that are of particular interest regarding their dynamic and earthquake response, which has been the subject of research in the past. A large number of bell towers with dimensions much larger than the ones examined here are located in numerous cities in Italy and around the world. This paper deals with in-situ measurements as means of system identification of two bell-towers located in the island of Kefalonia, Greece which was subjected during the winter of 2014 in an intensive earthquake sequence. The height of both these bell towers is approximately 21.5m; they are made of reinforced concrete with a hollow cross-section. Their dynamic characteristics of these bell towers were measured in-situ through a series of free vibration tests that were performed following the earthquake sequence. Subsequently their dynamic response was numerically simulated employing a 3-D elastic numerical simulation shell elements. The foundation was included in the numerical model assumed to be a mass-less concrete block that is formed by mass-less horizontal slabs and vertical slabs. A number of linear horizontal and vertical links are also employed aimed at representing the resistance of the soil surrounding the foundation block.. Following a successful numerical simulation of the measured dynamic response for both bell towers, predictions of the forcing levels for these two bell towers were obtained. Towards this objective all the available information obtained by the in-situ measurements was utilized together with the measurements of the ground acceleration during the most damaging 3rd February 2014 earthquake event, as mentioned before. Next, use was made of these predictions in order to discuss the observed performance. In this discussion, the influence of the structure-foundation-soil interaction is also considered.*

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1 INTRODUCTION

Bell towers are structures that are of particular interest regarding their dynamic and earthquake response, which has been the subject of research in the past. A large number of bell towers with dimensions much larger than the ones examined here are located in numerous cities in Italy and elsewhere. The largest percentage of these bell towers is built by stone or brick masonry. In many cases earthquake activity constitutes the major cause of serious damage for bell towers that many times leads to partial or total collapse (see [21], Manos and Kozikopoulos 2015). Consequently, there is a major international concern for the stability of numerous bell towers. This resulted to significant international research effort that includes in-situ monitoring of the response of bell towers on a temporary basis, like the one attempted here, or more sophisticated and on a permanent basis ([4, 6, 7, 8, 9, 12, 13, 14, 15, 16, 17, 18, 20]). Foundation problems for bell towers are evident in many case the most celebrated being Pisa's grand bell tower in Italy, that is quoted as a major medieval engineering error. Therefore, the soil flexibility is also an area of research interest for these structures especially when their dynamic and earthquake response is under investigation [5, 16, 17, 18, 20, 21, 22].

The purpose of this paper is to study the seismic performance of relatively new bell towers. Towards this the following procedure will be employed. First, a limited discussion will be presented in order to demonstrate the seismic performance of these structures in the island of Kefalonia, which is located in the most seismically active region of Greece. Next, two of these bell towers, namely the bell tower of Agios Gerasimos at Lixouri [21] and the bell tower of Theotokos at Chavriata, are selected for further in-depth study. Both structures are made of reinforced concrete and are distinct for the following reasons.

1) They represent the highest of the various bell towers that were considered.
2) They have been built after the destructive earthquake sequence of 1953 that heavily damaged a number of the old bell towers in the island. Moreover, they have been built according to relatively new design and construction practices after another damaging sequence of 1983, replacing in this way older bell towers that sustained seismic damage.
3) Both structures are quite close to locations where the ground acceleration during the most damaging 3rd February 2014 earthquake event was recorded. This enables response predictions on the basis of these recorded seismic ground motions.
4) Both structures are near to locations where other types of structural systems developed medium to heavy damage.

For these two structures the in depth study consist of the following stages:
1a) Obtain measurement of their geometry in-situ together with observations of their actual performance during the damaging event together with observations of nearby damaged structures.
2a) Obtain the fundamental dynamic properties for both structures from specific in-situ investigations that were conducted for this purpose.
3a) Obtain predictions of the forcing levels for these two bell towers. Towards this objective all the available information from stages 1a and 2a was utilized together with the measurements of the ground acceleration during the most damaging 3rd February 2014 earthquake event, as mentioned before.
4a) Use these predictions in order to discuss the observed performance. In this discussion, the influence of the structure-foundation-soil interaction is also considered.
5a) Finally, draw certain general conclusions from this investigation

2 IN-SITU OBSERVATIONS OF THE EARTHQUAKE RESPONSE OF BELL TOWERS IN KEFALONIA

The island of Kefalonia, together with the nearby islands of Zakinthos, Leykatha and Ithaka (the birth place of the Homeric hero Ulysses), belongs to a seismic zone of Greece with the highest design ground acceleration equal to 0.36g, where g is the acceleration of gravity. This is shown in figure 1 which depicts the seismic zoning map of Greece, together with the urban areas that were subjected to damaging earthquake sequences during the last 70 years that are in chronological order Argostoli (1953), Volos (1955), Thessaloniki (1978), Almiros (1980), Kalamata (1986), Pyrgos (1993), Kozani (1995), Aigio (1995), Athens (1999), Lixouri (2014). A description of the consequences of this earthquake activity is given by Manos [19] as well as by a number of publications [1, 2].

Figure 1. Seismic zoning map of Greece together with the urban areas that were damaged by earthquake activity during the last seventy (70) years.

A brief description of the consequences of the latest earthquake activity in the island of Kefalonia during 2014 together with considerable seismological and engineering information is included in the relevant report prepared under the auspices of the Earthquake Engineering Research Institute [1]. Furthermore, a numerical study of the seismic performance of Christian Churches that were damaged considerably by the 3rd of February 2014 Kefalonia earthquake is included in the work by Manos and coworkers together with a study of a R/C structure as well as the Bell tower of Agios Gerasimos. The reader is referred to these publications and only essential information is repeated here.

Figure 2. Total destruction of buildings during the Kefalonia 1953 earthquake.

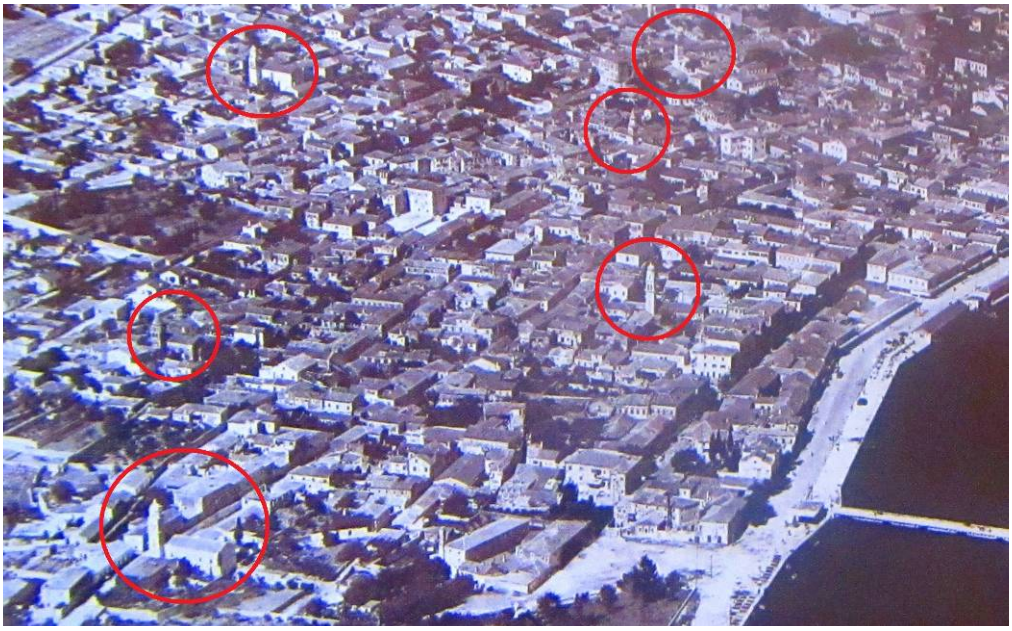


Figure 3. Areal view of Argostoli Kefalonia prior to the 1953 earthquake.

The earthquake of 1953 was of 7.3 Magnitude on the Richter scale and subjected to intense shaking the islands of Kefalonia, Ithaka and Zakinthos causing widespread destructions and human loss of 467 people. Following this earthquake a large number of people fled these islands, despite the recovery efforts. Argostoli, the capital city of Kefalonia, had the appearance of a heavily bombed city during World War II, as can be seen in figure 2. In comparison, figure 3 was taken before the 1953 earthquake. In this figure the location of a number of bell towers is also visible, indicated with red circles in order to facilitate the reader. It seems that the majority of these structures sustained considerable damage. It is indicative that the Greek Post Organization issued that year two stamps one of which depicts the collapse of a bell tower, as is shown in figure 4. In line with this stamp is the partial collapse of the bell tower at the village of Kourouklata during the recent 2014 earthquake sequence, shown in figure 5. The other stamp, shown in figure 6, depicts the moved earth together with the rubbles resulting from this movement together with the map of Kefalonia in the background. Indeed, due to the geological formations of this island the 1953 earthquake as well as the most recent one (2014) was accompanied with the loss of stability of slopes as well as soft soil formations, the most spectacular being the one at the Lixouri harbour (figure 5).

Figure 4. Stamp issued in 1953 by the Greek Post Organization

Figure 5. Partial collapse of the bell tower at the village of Kourouklata during the recent 2014 earthquake sequence

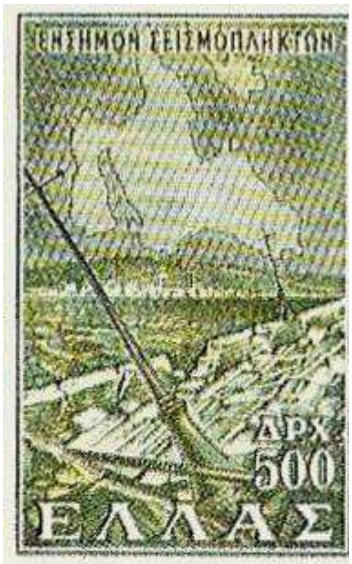


Figure 6. Stamp issued in 1953 by the Greek Post Organization

Figure 7. The embankment at the Lixouri harbour after the 2014 earthquake sequence.

Table 1. Historical seismicity as is compiled by Papazachos and Papazachou 2003.

Date of occurrence	*Magnitude in Richter*	*Location that suffered*	*Maximum Mercali Intensity*
Spring, 1469	7.2	Kefalonia	IX
30-9-*1636*	7.2	Kefalonia	IX
16-July-1638	6.4	Kefalonia	VII
24-August-1658	7.0	Kefalonia	IX
28-August-1714	6.4	Kefalonia	VIII
23-June-1741	6.4	Lixouri	VIII
13-June-1759	6.3	Argostoli	VIII
24-July-1766	7.0	Kefalonia	IX
22-July-1767	7.2	Kefalonia	X
14-March-1862	6.5	Argostoli	IX
4-February-1867	7.4	Kefalonia	X
24-January-1912	6.8	Kefalonia	X
27-January-1915	6.6	Ithaka	IX
7-August-1915	6.7	Ithaka	IX
12-August-1953	7.2	Kefalonia	X
17-September-1972	6.3	Kefalonia	VII
17-January-1983	7.0	Kefalonia	VI

Table 2. Description of Seismic Intensity by the Modified Mercali Scale

The following is an abbreviated description of the levels of Modified Mercalli intensity.

Intensity	Shaking	Description/Damage
I	Not felt	Not felt except by a very few under especially favorable conditions.
II	Weak	Felt only by a few persons at rest,especially on upper floors of buildings.
III	Weak	Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated.
IV	Light	Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V	Moderate	Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.
VI	Strong	Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.
VII	Very strong	Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.
VIII	Severe	Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.
IX	Violent	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.
X	Extreme	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.

2.1 Bell towers in Kefalonia today.

From a survey of the bell towers in the cities of Argostoli, Lixouri and some of the villages that have been affected by the 2014 earthquake sequence the following picture emerges concerning the bell towers that exist today.

In the first category are 3-D structures with a symmetric cross-section in plan and rather slender in height (figures 8a to 8j). Most of these bell towers are relatively new. That means that they have been built after the destructive earthquake of 1953. They are mainly R/C structures of relatively similar morphological feature. The main variable seems to be their height, as can be seen in figures 8a to 8j (from 12m to 21.5m). A number of them are tapered in height (figures 8a, 8c, 8e, 8f, 8g) whereas for a number of them their cross section remains constant along the height (figures 8b, 8d, 8h, 8i). A number of these bell towers are detached, that means that they are not connected to any adjacent structure all along their height (figures 8a, 8b, 8c, 8d, 8h) whereas a number of them are connected to the neighbouring church thus making their earthquake response more complex. The only one of these bell towers that was damaged is the one depicted in figure 8j. The damage of this bell tower is not part of the present study.


a) Ag. Gerasimos Lixouri 21.5m measured Detached,


b) Panagia, Havriata 21.5m measured Detached,


c) Ag. Spiridon Argostoli 14m estimated Detached,


d) Archagelos Argostoli 12m estimated Detached


e) Evagelistria Argostoli 19m estimated Connected to the church


f) Ag. Eleftherios Argostoli Estimated 19m Partially connected to the church


g) Ag. Nikolaos Argostoli Estimated 19m Partially connected to the church


h) Mazarakata 12m estimated Detached


i) Pantokratoros Lixouri, Estimated 15m Connected to the church


j) Ag. Ioanis Kourouklata Estimated 19m. Partially connected to the church

Figure 8. Bell towers with a symmetric cross section in plan and rather slender in height.

In the second category are bell towers that have a rather planar geometry; that is they are much stiffer in one horizontal direction than in the other one, as depicted in figures 9 and 10. The one shown in figure 9c has been severely affected by the 1953 earthquake sequence, whereas the rest have been damaged by the recent 2014 earthquake sequence with a variable level of severity (figures 9a, 9b, 9d, 9e). The bottom story of the 9d bell tower was repaired with R/C jacketing in 2015.

-

a) Agia Thekla 2014, Bottom story connected to the church. The very top collapsed and the columns of the 2nd story heavily damaged

b) Panagia, Argostoli 2014. Bottom story partially connected to the church, Slight damage at the top right corner

c) Panagia Evaggelistria, Kastro. Top part collapsed 1953. Bottom story heavily damaged. Detached

d) Kecgrionos Monastery 2014, Lixouri. Bottom story damaged. Partially connected in-plane to neigbouring wall.

e) Ag. Marina Soulari, A stiff stone-masonry bell tower was constructed after 1953 partially connected in-plane with the west peripheral wall of the church. This was totally destroyed after the 2014 earthquake sequence.

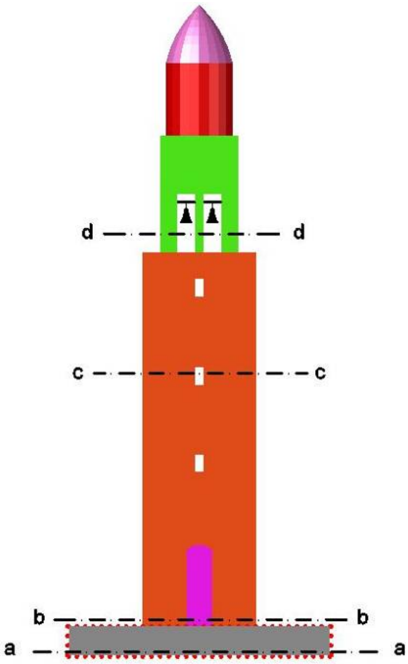
Figure 9. Damaged planar bell towers.

a) Valianou, A relatively flexible low height bell tower was built after 1953 partially connected in-plane with the west peripheral wall of the church.

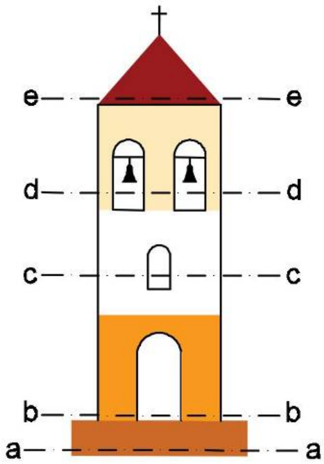
b) Ag. Sofia Mantzavinata. Detached with no damage. Nearby stone masonry Basilica with extensive damage.

c) Ag. Spiridon Mantzavinata. Detached with no damage. Nearby stone masonry Basilica with extensive damage.

Figure 10. Relatively low-height planar bell towers with no damage.



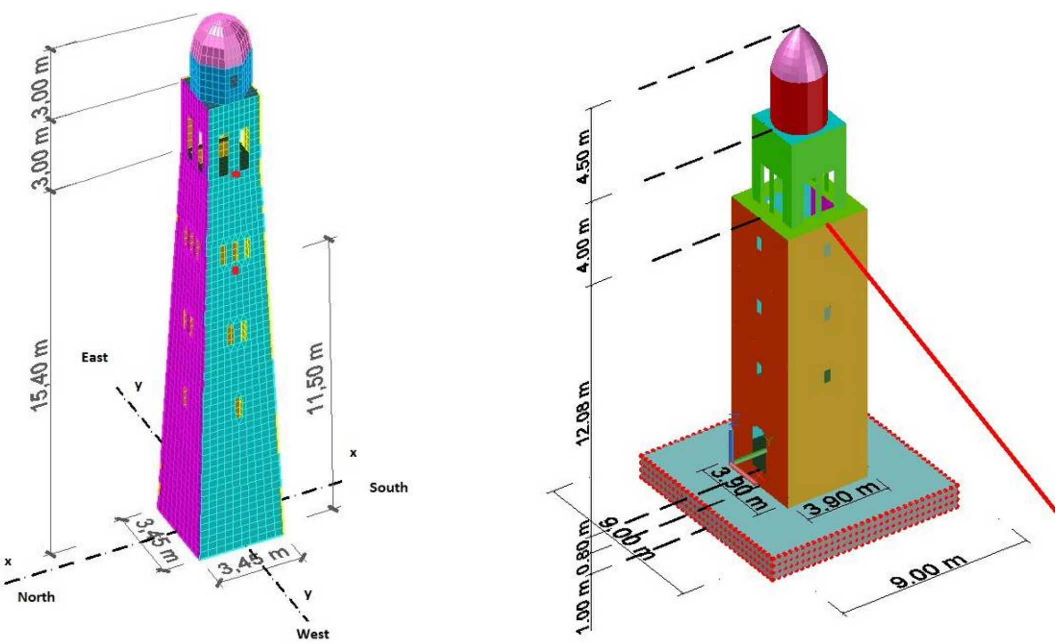
a) Bell towers with a symmetric cross section in plan and rather slender in height.



b) Relatively low-height planar bell towers

Figure 11. Cross sections where typical damage patterns are observed

In figure 11 certain horizontal cross-sections are indicated for both types of bell towers where typical damage patterns are observed. The main underlying cause for the development of such typical forms of damage can be due either to in-plane or out-of-plane states of stress. Because seismic actions have a random orientation that continuously changes during a strong earthquake event the in-plane and out-of-plane states of stress are usually combined. However, their distinction is useful as it serves in understanding the observed consequences in a less complex way. As is shown in figure 11a and 11b in both structures the chosen cross section represent locations that large amplitude states of stress (in-plane or out-of-plane) are expected to occur combined at the same locations with relatively small resistance due to the geometric characteristics of these cross-sections (presence of openings) or material characteristics (soil-foundation contact surface). Due to the way the mass is distributed in both types of bell towers, the presence of the bells as well as the basic structural formation that is of a cantilever type one expects that the horizontal components of a strong earthquake motion will cause a distribution of shear and bending moment demands that will increase as one considers horizontal cross-sections that are located from the top to the bottom of a bell tower. Based on this the cross-section of bell towers decrease in dimensions as one moves from bottom to the top, as is the case for the bell tower of Agios Gerasimos at Lixouri (figures 11a and 8a) as well as the bell towers shown in figures 8e, 8f, 8g. This of course holds when modes of vibration of a displacement form that increase from bottom to the top dominate the dynamic response of these structures during a given seismic event. On the contrary, this is not the case for higher order modes of vibration that in certain cases may have a considerable effect on the upper structural parts of these bell towers. For both types of bell towers with the horizontal cross sections designated as a-a the possibility of damage developing at the soil-foundation contact surface is indicated where both the shear and the overturning moment amplitudes are expected to attain their maximum values from the horizontal excitation of a seismic event. Damage to the upper relatively weak in the out-of-plane direction parts is indicated by cross-section e-e of the planar type (figure 11b and figures 9a, 9b and 9c). The possible damage indicated by cross-sections d-d for both types (figures 11a and 11b) can result from either in-plane and /or out-of-plane states of stress demands (figures 8j, 9a, 9c). Another possible source of damage springs from the interaction between a bell tower and the adjacent church when these two structures are not detached with properly designed seismic gaps. This is the case of the low-height stiff bell tower that is shown in figure 9e.



a) The bell tower of Agios Gerasimos at Lixouri. b) The bell tower of Theotokos at Havriata
Figure 12. Two bell towers that were subjected to pull-out excitations

3 IN-SITU MEASUREMENT OF THE DYNAMIC RESPONSE OF THE BELL TOWER AT HAVRIATA

The dynamic characteristics of this bell tower were measured in-situ through a series of pull-out excitations tests (see Manos et al, [20]), which were performed on 10^{th} October, 2015, approximately twenty (20) months after the damaging earthquake sequence of 3^{rd} of February 2014. This type of excitation that was also employed for the bell tower of Agios Gerasimos at Lixouri [21], namely pull-out excitation (see figures 12a and 12b), was produced from a sudden sadden rupture of a high strength wire, which was part of a relatively flexible steel cable.

This steel cable was attached externally at the North concrete wall of the bell tower, at a height of approximately 14m from the ground. The other side of this cable was attached at a rigid weight lying at the ground surface, which was located at a distance of approximately 30m from the North side of this bell tower, as indicated in figures 13 and 14. As can be seen in these figures the direction North-South represents the main axis of the employed pull-out excitation.

Figure 13. Pull-out excitation for the Havriata bell tower. The red line indicates the flexible steel cable.

Figure 14. Pull-out excitation for the Havriata bell tower. The flexible steel cable is also indicated.

The level of this pull-out excitation was capable of producing horizontal vibrations by exciting mainly the horizontal translational eigen-modes of this bell tower. This dynamic response could be capture by the employed tri-axial velocity sensors with a sensitivity of 0.001mm/sec and a data acquisition system with a sampling frequency of 400Hz. The velocity response measurements were made by securing the velocity sensors at 5 levels along the height of the bell tower. This is depicted in figure 15 whereby for each level the vertical distance from level 0 is also indicated e.g. at 0.80m (level 1), 3.84m (level 2), 6.78m (level 3), 9.83m (level 4), 12.88m (level 5). Due to major difficulties in accessing levels higher than level 5, no attempt was made to place velocity sensors higher than level 5. Level 0 represents the top surface of the reinforced concrete block with a thickness of 0.5m end plan dimensions 9m x 9m. There is a layer of well compacted gravel with a thickness of at least 0.5m below this reinforced concrete block.

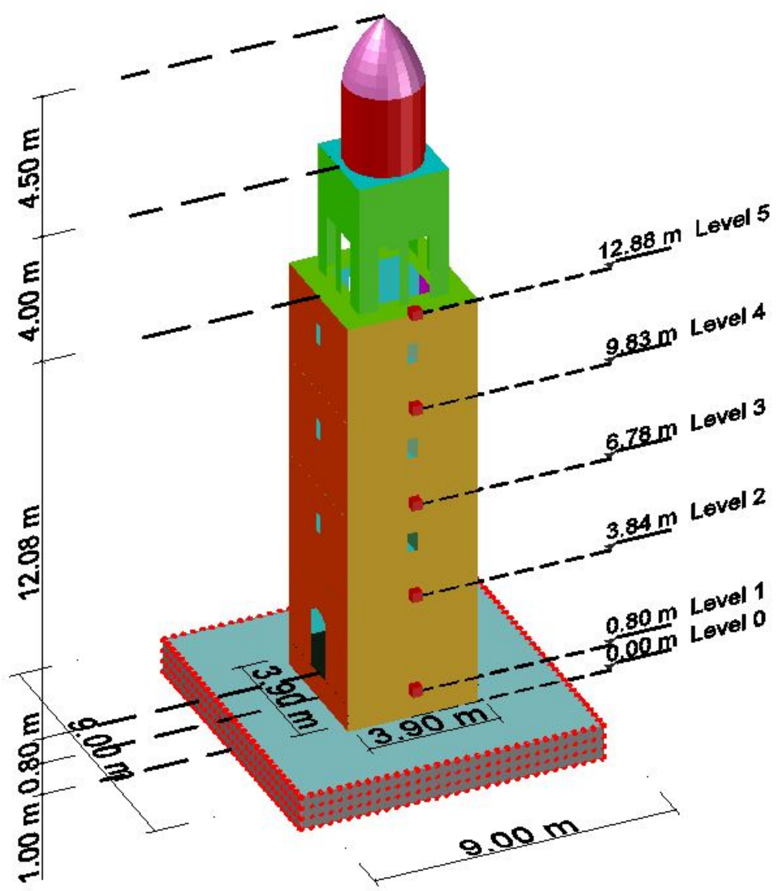


Figure 15. The basic dimensions of the Havriata bell tower together with the location of the five levels where the velocity sensors were attached.

All the obtained data were subsequently studied both in the time and frequency domain through available FFT software. Figure 16 depicts the variation of the measured North-South (x-x) and East-West (y-y) peak velocity response along the height of the Havriata tower, as obtained from the pull-out tests. Two complete test sequences were performed and the plotted values represent the average of the measured response during these two sequences at the same location. These plotted values are the peak values of the measured velocity response. As can be seen in this figure, the measured peak velocity response in the North-South direction (excitation axis) is larger, as expected, from the peak East-West velocity response and it occurs almost at the initiation (0.1625sec from starting time) of the used pull-out excitation. The East-west measured peak velocity response is considerably smaller than the peak North-South velocity response and it occurs at a later stage (1.4425sec). The studied bell tower is almost symmetric in the two horizontal directions (North-South x-x and East-West y-y). Moreover, as already stated, the excitation axis is along the North-South direction. The development of the East-West translational response must be attributed to the fact that, as will be shown in this section as well as section 4, which deals with the numerical simulation, both the 1st North-South and 2nd East-West translational eigen-modes of this bell tower have very similar eigen-frequency values.

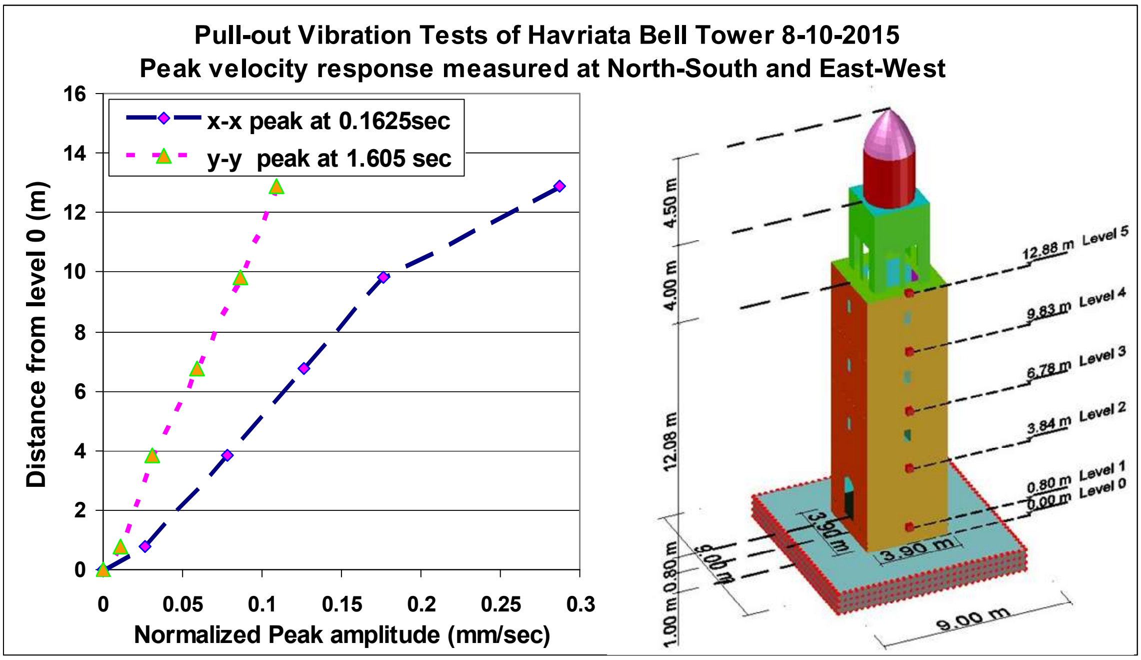


Figure 16. Variation of the peak velocity response along the height of the Havriata bell tower.

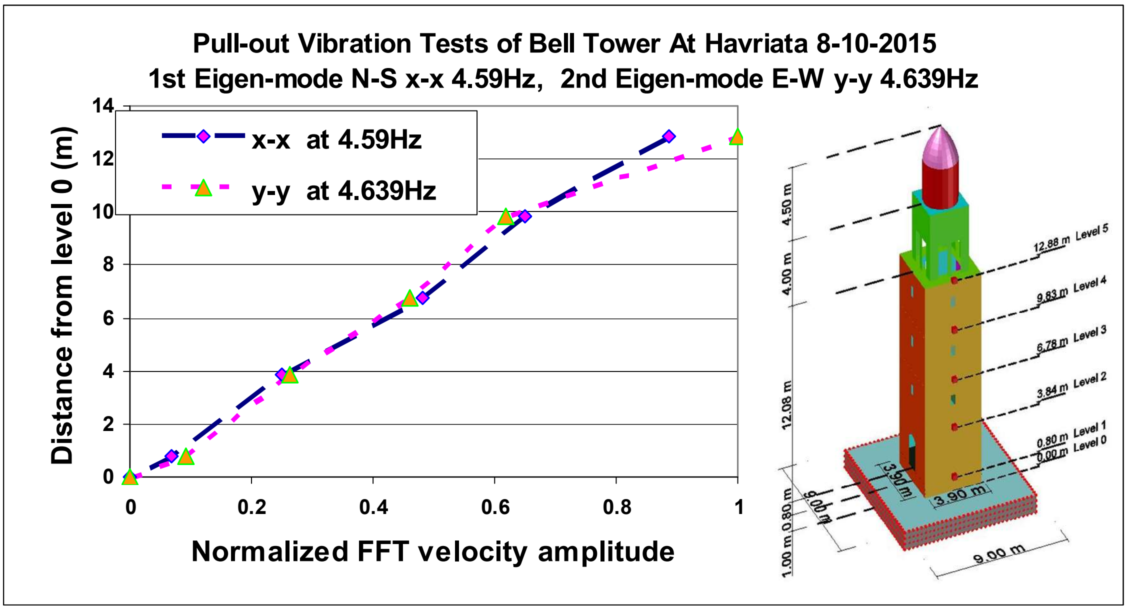
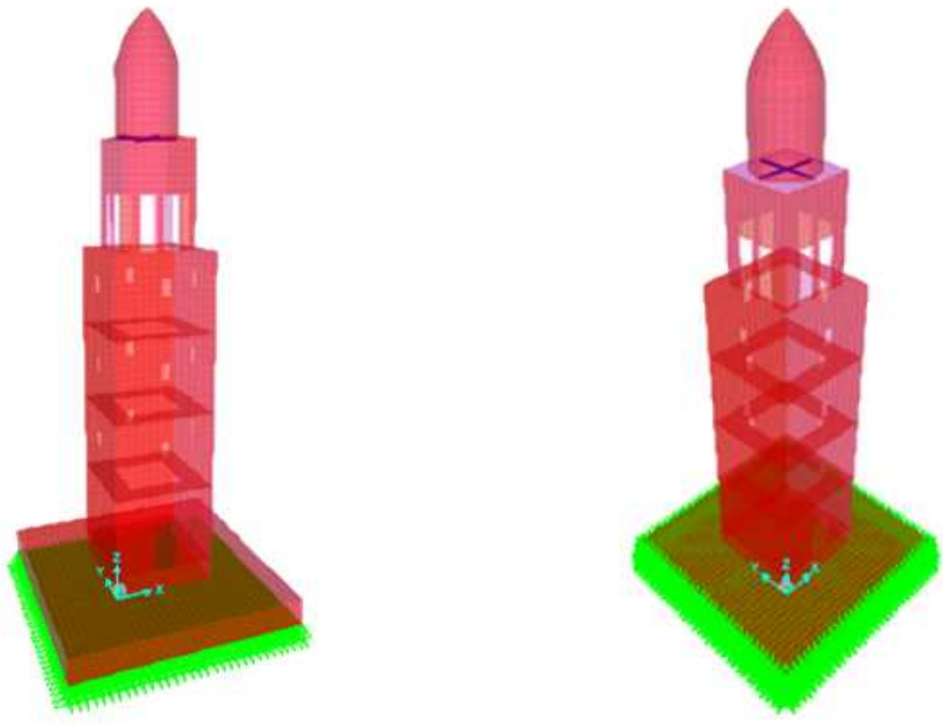


Figure 17. FFT velocity response amplitudes along the height of the Havriata bell tower for the main eigen-frequencies.

From the analysis of the measured velocity response in the frequency domain the main eigen-frequency values were obtained equal to 4.59Hz for the N-S (x-x) and 4.639Hz for the E-W (y-y) directions, respectively. These frequency values, as will be explained, correspond to the 1st and 2nd horizontal translational modes. This is concluded from the same frequency analysis by selecting and plotting along the height in figure 17 the FFT amplitudes that correspond to these frequency values, as obtained from the measured vibration response along the height of the bell tower at each one of the five (5) selected levels. The plotted in figure 17 FFT amplitude variation along the bell tower height for these specific eigen-frequency values are normalized, thus representing the mode shapes of the two N-S and E-W eigen-modes. As can be seen in this figure, the resulting mode shapes have an almost linear variation along the height from level 0 to level 5 thus representing the translational horizontal mode-shapes of the bell tower in the N-S (x-x) and E-W (y-y) directions.

4 NUMERICAL SIMULATION OF THE DYNAMIC RESPONSE OF THE HAVRIATA BELL TOWER

The dynamic response of this bell tower was numerically simulated employing a linear elastic dynamic analysis utilizing shell elements assumed to be of reinforced concrete as an isotropic material with a Young's Modulus equal to E=10000MPa and typical to reinforced concrete. The bells were assumed to weight 500kg. They were simulated with a steel beam that was placed at the right location and height where the actual steel beam supporting the bells is located. A mass and weight multiplier was used for this beam to account for the extra mass and weight of the bells. The geometry and thickness of the external walls, the slabs and the top vault of this bell tower were included in this numerical simulation.



a) Foundation block simply-supported b) Foundation block supported with 3-D links

Figure 18. 3-D numerical simulation of the Havriata bell tower

The foundation was assumed to be a mass-less concrete block of total thickness equal to 1m that was numerically formed by the following parts:

- Two mass-less and stiff horizontal slabs with a thickness 0.45m were located one at zero (0) level and the other at a depth of 1.0m from zero (0) level. These slabs represent the upper and lower horizontal planes of the foundation concrete block. In addition, four mass-less and stiff vertical slabs having a thickness 1.0m were also added. Two of these slabs were placed at the x-z plane and the other two at the y-z plane of the numerical model. These vertical slabs represent the peripheral vertical planes of the foundation block facing the East-West (x-z plane) and the North-South (y-z plane) directions. In this way the foundation block was formed that represents in itself a very rigid part of the whole structure. In order to approximate the possibility of a flexible interface between this foundation block and the surrounding soil volume two types of boundary conditions were introduced in this numerical model. In the first case all the points of the bottom slab of this foundation block were simply supported whereas in the second case all these points were connected to the supporting media with 3 directional linear springs with a stiffness equal to 5000000KN/m, a value that corresponds to a very stiff soil (see also Manos and Kozikopoulos [21]).

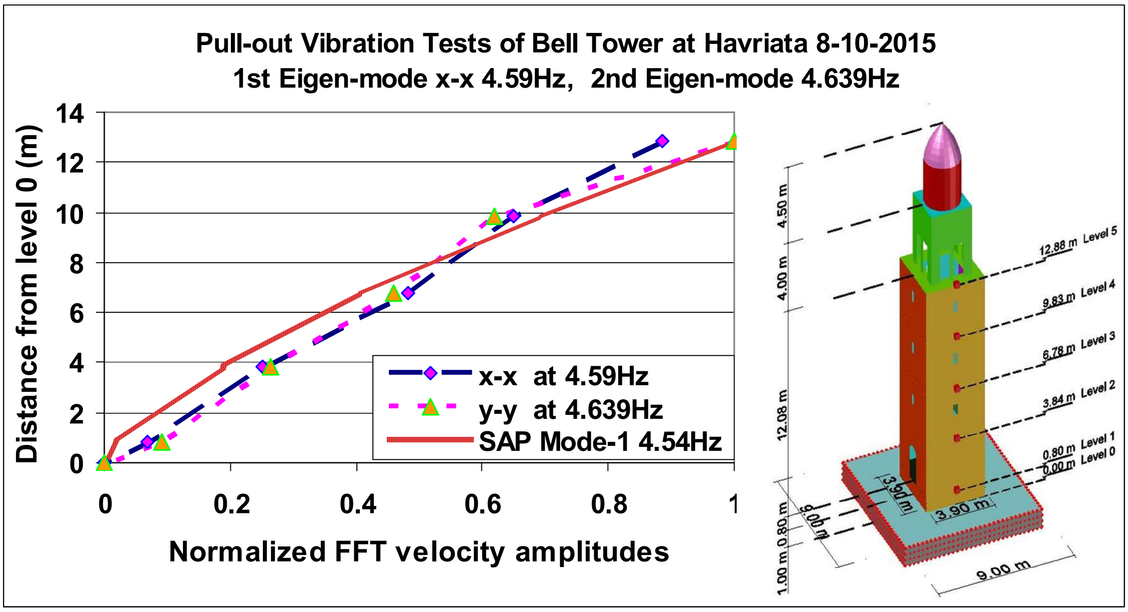


Figure 19. Comparison of measured and predicted dynamic characteristics of the Havriata bell tower.

In figure 19 the numerically predicted N-S / E-W eigen-frequencies values and eigen-mode shapes (SAP Mode-1) are compared with the corresponding measurements (N-S x-x and E-W y-y). Only the numerical results for the x-x 1st mode are plotted in this figure because the corresponding results for the second y-y mode are almost identical, due to the geometric symmetry of this bell tower. The plotted eigen-mode shape is limited to that part of the bell tower where measurements of its dynamic response were obtained from the in-situ campaign, as already described, from level 0 to level 5. As can be seen in this figure reasonably good agreement can be seen between both the measured and numerically predicted eigen-mode shape as well as the eigen frequency values. The numerical results obtained from the simply supported foundation block are almost identical to the corresponding results obtained from the numerical model whereby the foundation block is supported by 3-D stiff links (5000000KN/m), as already described. Thus, in the case of the Havriata bell tower it was not necessary to employ less stiff 3-D links at the soil foundation interface in order to reach a reasonably good agreement between measured and numerically predicted dynamic response characteristics for this structure. On this point the it is important to underline that in the case of the bell tower of Agios Gerasimos at Lixouri [21], whereby the authors followed and identical methodology for in-situ measurements and numerical simulation, it was necessary in-order to reach such a reasonable degree of agreement between measured and predicted dynamic response to use 3-D links with a stiffness value equal to

21000KN/m. The obvious conclusion is that in the case of the Havriata bell tower the flexibility of the surrounding soil volume exercised much less influence on its dynamic response than in the case of the Agios Gerasimos at Lixouri bell tower. This observation is in good agreement with the overall picture of the two locations as the Havriata bell tower is located at the top of a hill whereas the Agios Gerasimos bell tower is located at a close distance from the harbor almost at sea level. However, it must also be underlined that this conclusion is based on in-situ dynamic response measurements generated from relatively very low level excitations and it should not be extended to the very intense excitation that both structures were subjected to during the earthquake sequence of 3rd February, 2014. On the contrary, as will be described in the following, both structures were subjected to very intense earthquake excitations during the 3rd of February 2014 seismic event [1].

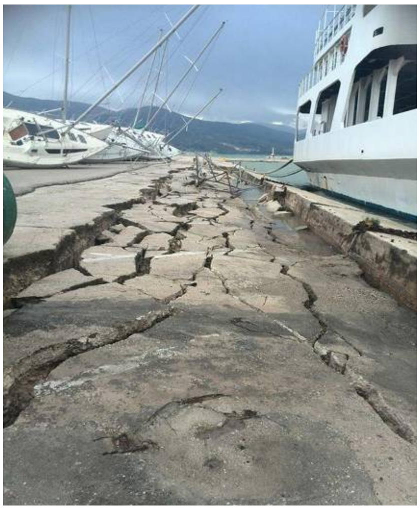


Figure 20a. The earth fill embankment at the harbor immediately after the seismic event of 3rd February 2014

Figure 20b. The failure of the retaining wall and the backfill at the South side of The Theotokos church of Havriata (see also figure 13).

Figure 20a depicts the failure of the earth fill embankment at the harbor of Lixouri due to the seismic event of 3rd February, 2014. The bell tower of Agios Gerasimos is located to a close distance (400m) from this harbor site [21, 22]. Similarly, figure 20b shows the failure of the retaining wall and the backfill at the South side of the Theotokos church of Havriata, at a distance of less than 50mm from the bell tower (see figure 13). Moreover, the church of Theotikos of Havriata developed also significant structural damage. These observations represent visual evidence of the severity of the ground motion of this seismic event. In addition, both in Lixouri as well as in Havriata the seismic excitation was recorded [1, 21, 22], by strong motion accelerographs located at close distance from either bell tower (approximately 300m). From the North-South (N-S) and East-West (E-W) horizontal components of the Havriata record of the 3rd of February seismic event the response spectra curves of figure 21 were derived. These are inelastic constant ductility response spectra curves and were derived assuming a damping ratio value ξ=5% and two ductility ratio values either μ=1.5 or μ=3.0. In the same figure, the design spectra curves derived using the EuroCode 8 [23] design provisions (Type-1 and Type-2) assuming soil category D, ground design acceleration a_g=0.36g (g being the acceleration of gravity), importance factor equal to γ=1 and response modification factor equal to q=3. Finally, the acceleration/force levels as were provided by the old Greek Seismic Code (prior to 1983) are also indicated [11]. The new Greek seismic code acceleration/force levels are similar to the ones specified by EuroCode 8 and are not included in this plot [23].

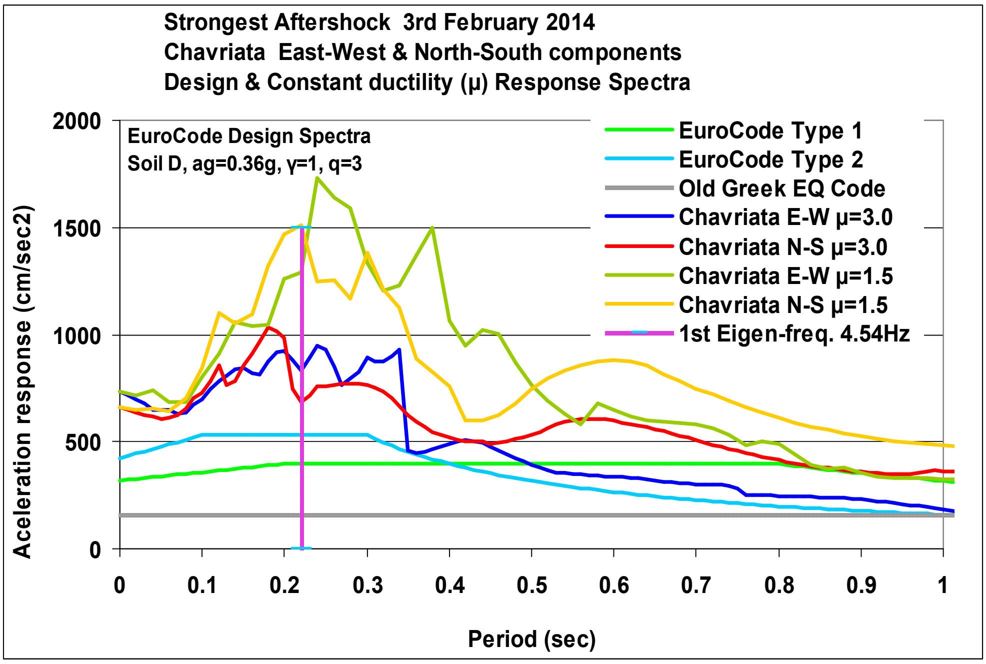


Figure 21. Design and Response spectra curves for the 3rd of February earthquake sequence as recorded at a close distance from the Havriata bell tower.

The same procedure was followed for the Lixouri record of the 3rd of February 2014 seismic event. The relevant response and design spectra curves are shown in figure 22. The measured in-situ eigen-period of the 1st translational mode for the Havriata and the Agios Gerasimos-Lixouri bell towers is also plotted in figures 21 and 22, respectively.

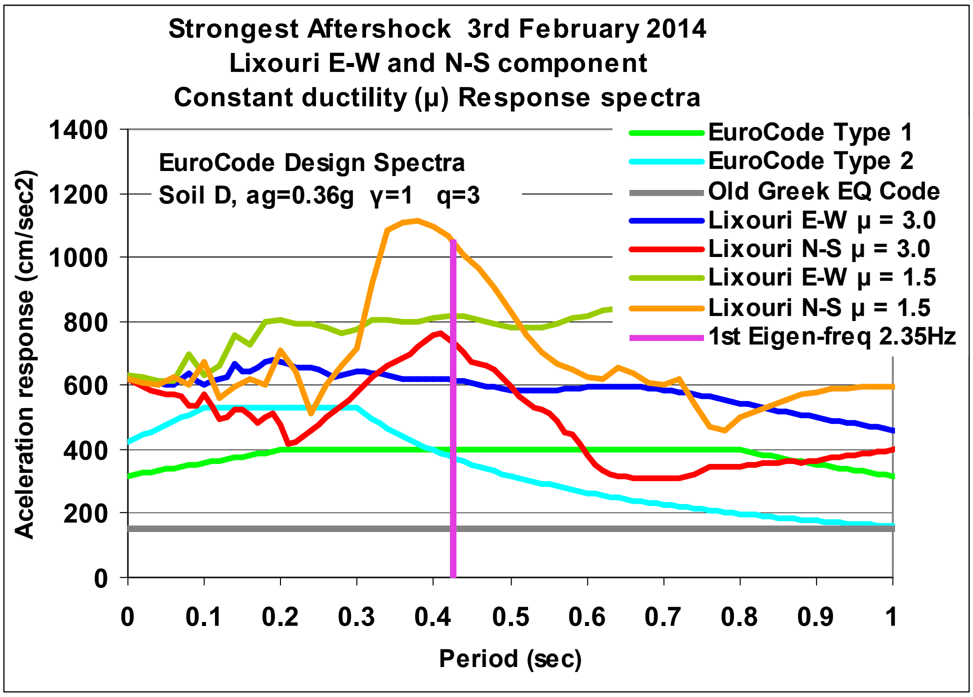


Figure 22. Design and Response spectra curves for the 3rd of February earthquake sequence as recorded at a close distance from the Agios Gerasimos bell tower at Lixouri.

It must be underlined that the Havriata bell tower was design and constructed well after 1996, when the provisions of the new Greek seismic code became mandatory. The adopted in the case of Agios Gerasimos bell tower design acceleration/force levels for its seismic design could not be ascertained up to now. However, in this case from eye witness reports the foundation block in the case of Agios Gerasimos bell tower was much thicker and as such was included in the numerical simulation.

Studying the response and design spectra curves of figures 21 and 22 in connection to the measured in-situ eigen-period of the 1st translational mode for the Havriata and the Agios Gerasimos-Lixouri bell towers, that is expected to generate the largest part of the earthquake response for both structures the following observations can be made.

a) The 1st eigen-period value for both structures corresponds to the period range with the relatively large spectral values.

b) The presence of the relatively flexible soil in the case of the Agios Gerasimos results in a larger eigen-period value (0.425sec) than the corresponding eigen-period value for the Havriata bell tower (0.220sec).

c) The presence of the flexible soil in the case of the Lixouri also influenced the seismic ground motion as can be seen when the response spectral curves of figure 22 (Lixouri ground motion record) are compared with the corresponding response spectral curves of figure 21 (Havriata ground motion record). Thus, it can be seen that in the case of the Havriata ground motion the period range with the largest spectral values is from 0.15sec to 0.4sec whereas in the case of the Lixouri ground motion record this period range was sifted to 0.3sec to 0.6sec. This influence of the soil flexibility is to be expected for both the structural dynamic response as well as for the frequency content of the ground motion.

d) The combination of the above observations b and c together with observation a lead to the conclusion that both structures were subjected to significant seismic forces. A preliminary estimate of these forces can be derived, based on the design and spectral acceleration values that correspond to the value of the dominant 1st translational eigen period for either tower is as follows.

e) On the basis of the EuroCode design spectra curves a preliminary estimate of the seismic forces is that of 40% to 50% of the tower weight. This estimate is based on the hypothesis that the assumed response modification factor value ($q=3$) is a realistic assumption. On the basis of the inelastic constant ductility response spectra curves (assuming $\mu=3$), a preliminary estimate of the seismic forces is that of 60% to 80% of the tower weight. These force levels are considerably larger than the corresponding forces levels based on the old Greek seismic code provisions. On the contrary, these seismic forces levels are considerably smaller than the preliminary estimates based on the response spectra curves derived from the actual recordings of the seismic ground motion at either Havriata or Lixouri.

f) These preliminary seismic force estimates are based on assumed values for the response modification factor (q) or the ductility (μ) equal to 3. However, this assumption is based on the development of plastic structural response that is also accompanied with permanent structural damage that could not be observed in either structure. Therefore, either the preliminary estimate of the seismic forces should be based on smaller q and μ values (e.g. $\mu=1.5$) or a different plastic energy mechanism should be adopted as the one leading to the reduction of the seismic forces that developed in either structure.

g) Assuming a ductility value $\mu=1.5$ for the Havriata bell tower leads to a preliminary estimate of the seismic forces equal to 140% of the tower weight, a relatively quite large value. Similarly, assuming a ductility value $\mu=1.5$ for the Agios Gerasimos at Lixouri bell tower leads to a preliminary estimate of the seismic forces equal to 80% to 100% of the tower weight, again a relatively large value. In the later case, the large foundation bock and the flexible soil may have contributed to the reduction of the actual seismic forced to somehow moderate levels, thus contributing to the stable performance of this bell tower. The same mechanism is also expected to influence the earthquake response of the Havriata bell tower. In this case the final stable performance was also the result of a relatively resent design and construction based on the provisions of the new Greek seismic code, as already mentioned. A more detailed evaluation of the earthquake response of these bell towers is necessary in order to quantify the validity of these observations.

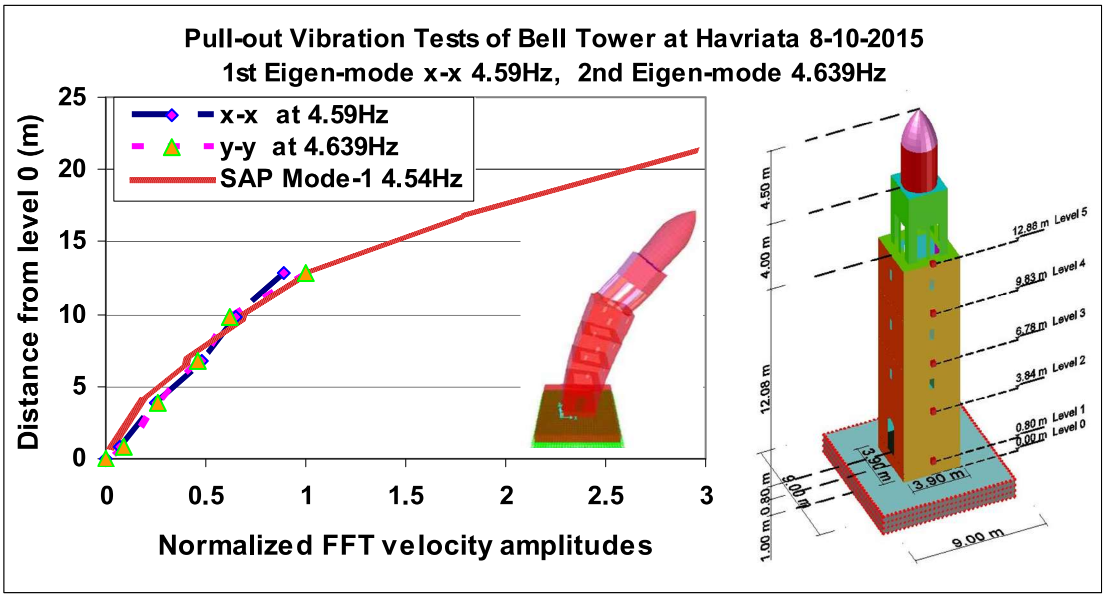
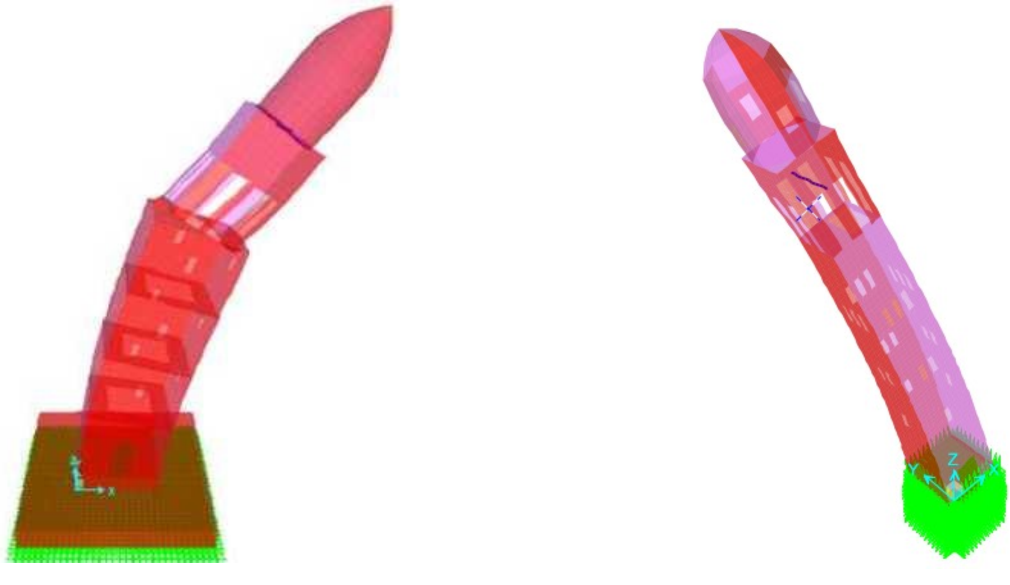


Figure 23. Comparison of measured and predicted dynamic characteristics of the Havriata bell tower.



a) Theotokos bell tower at Havriata.
Translational mode N-S (x-x)
T=0.220sec, f=4.54Hz, (stiff soil)

b) Agios Gerasimos bell tower at Lixouri.
Translational mode N-S (x-x)
T=0.425sec, f=2.35Hz, (medium soil)

Figure 24. Comparison of translational eigen-modes and eigen-frequensies of the Thetokos bell tower at Havriata and the Agios Gerasimos bell tower at Lixouri.

In figure 23, the mode-shape of the 1st translational eigen-mode of the Havriata bell tower is shown, as predicted numerically for the full height of this bell tower (from level 0 to the top). In figure 24a and 24b, the mode shapes of the 1st translational eigen-mode for the Havriata and the Agios Gerasimos bell towers are depicted, respectively. As can be seen in these figures, the flexible soil in the case of the Agios Gerasimos bell tower leads to a increase in the flexibility of the whole structure. On the contrary, the stiff soil in the case of the Havriata bell tower results to a relatively stiff structure and leads to a more pronounced response of its upper part which is relatively more flexible than its lower part.

5 CONCLUSIONS

- Reasonably good agreement was obtained between both the measured and numerically predicted eigen-mode shape as well as the eigen frequency values for the Havriata bell tower.
- It was demonstrated by the in-situ measurements of the dynamic response of the Havriata as well of the Agios Gerasimos bell tower that the foundation deformability is a significant parameter that must be taken into account in order for the numerical response predictions to have any realism.
- On the basis of the EuroCode design spectra curves a preliminary estimate of the seismic forces is that of 40% to 50% of the tower weight. This estimate is based on the hypothesis that the assumed response modification factor value (q=3) is a realistic assumption. On the basis of the inelastic constant ductility response spectra curves (assuming μ=3), a preliminary estimate of the seismic forces is that of 60% to 80% of the tower weight. These force levels are considerably larger than the corresponding forces levels based on the old Greek seismic code provisions. On the contrary, these seismic forces levels are considerably smaller than the preliminary estimates based on the response spectra curves derived from the actual recordings of the seismic ground motion at either Havriata or Lixouri.
- These preliminary seismic force estimates are based on assumed values for the response modification factor (q) or the ductility (μ) equal to 3. However, this assumption is based on the development of plastic structural response that is also accompanied with permanent structural damage that could not be observed in either structure. Therefore, either the preliminary estimate of the seismic forces should be based on smaller q and μ values (e.g. μ=1.5) or a different plastic energy mechanism should be adopted as the one leading to the reduction of the seismic forces that developed in either structure.
- Assuming a ductility value μ=1.5 for the Havriata bell tower leads to a preliminary estimate of the seismic forces equal to 140% of the tower weight, a relatively quite large value. Similarly, assuming a ductility value μ=1.5 for the Agios Gerasimos at Lixouri bell tower leads to a preliminary estimate of the seismic forces equal to 80% to 100% of the tower weight, again a relatively large value. In the later case, the large foundation bock and the flexible soil may have contributed to the reduction of the actual seismic forced to somehow moderate levels, thus contributing to the stable performance of this bell tower. The same mechanism is also expected to influence the earthquake response of the Havriata bell tower. In this case the final stable performance was also the result of a relatively resent design and construction based on the provisions of the new Greek seismic code, as already mentioned. A more detailed evaluation of the earthquake response of these bell towers is necessary in order to quantify the validity of these observations.

- In situ measurements like the ones presented here are very important to be able to identify sources that may significantly influence the dynamic and earthquake response of civil engineering structures. They may also serve the purpose to help formulate a realistic numerical model by properly incorporating such influences in order to yield realistic predictions of the dynamic and earthquake response of the examined structure.

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- To the memory of Ray W. Clough, Professor Emeritus of the University of California, at Berkeley, U.S.A.

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