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SEISMIC PERFORMANCE OF BELL TOWERS

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Abstract

The dynamic and earthquake response of bell towers is examined here. The dynamic characteristics of two bell tower were measured in-situ through a series of free vibration tests that were performed following a strong earthquake sequence. Subsequently the dynamic and earthquake response of one of these bell towers was numerically simulated employing a 3-D elastic numerical simulation shell of the bell tower and its foundation. The earthquake response is examined for a variety of load combination conditions employing design spectra as well as the ground motion recorded at a small distance from this bell tower during a prototype earthquake motion. This study is further included to examine the influence of the soil-foundation interface. Towards this end, the non-linear uplift mechanism of the foundation block from the underlying soil is numerically simulated. The earthquake response is examined comparatively in terms of global response parameters, such as base shear, overturning moment, and top displacements as well as peak stress values at particular parts of the bell tower that stress concentration occurs. These response values are studied in a comparative way in order to validate these numerical simulations with the observed performance.

Keywords: Bell towers, In-situ Dynamic Measurements, Numerical Simulation, Earthquake Response, Soil-structure Interaction.

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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 ([3] to [10]). 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 ([1] and [2]). 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 ([3] to [10]). 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 ([12] to [20]).

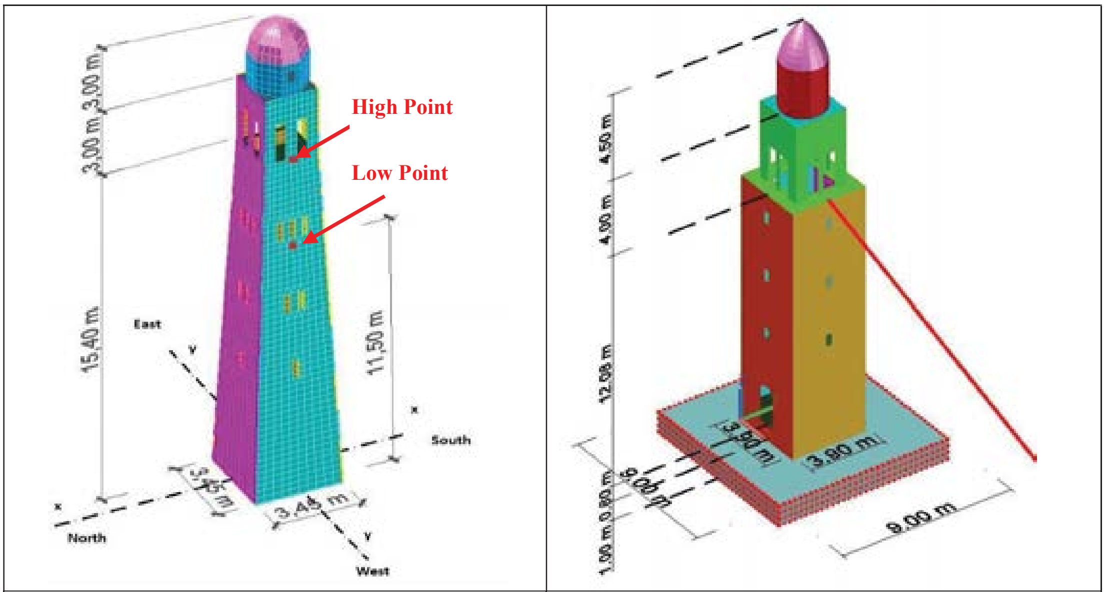
The purpose of this investigation is to study the seismic performance of relatively new bell towers compared to the old ones. Two of these relatively new bell towers, namely the bell tower of Agios Gerasimos at Lixouri [21] and the bell tower of Panayia Argiliotissa at Havriata, are selected for further in-depth study. Both structures are made of reinforced concrete and are distinct for the following reasons. A review of the seismic performance of bell towers and especially those located in Kefalonia, Greece is presented in [21] and [22].

Figure 1a. Stamp issued in 1953 by the Greek Post Organization

Figure 1b. Partial collapse of the bell tower at the village of Kourouklata during the recent 2014 earthquake sequence

2 IN-SITU MEASUREMENTS

In the present work a summary of in-situ measurements is presented again for two bell towers located in the island of Kefalonia. These bell towers are : a) The bell tower of Agios Gerasimos at Lixouri. b) The bell tower at the village of Havriata. Information on these two bell towers are also included in [21] and [22]. The basic geometrical features of these two bell towers are depicted in figure 2. Both bell towers were constructed after a catastrophic earthquake sequence in 1953 which destroyed numerous bell towers in this island. Both bell towers are made of reinforced concrete and did not sustain any structural damage during the recent 2014 earthquake sequence despite the severity of the earthquake ground motion.



a) Agios Gerasimos Bell Tower. Used instrumentation	b) Havriata Bell Tower. Used pull-out arrangement

Figure 2. The Agios Gerasimos bell tower at Lixouri (left) and the Panagia Agriliotissa Bell tower at Havriata (right)

2.1. In-situ Measurements of Agios Gerasimos Bell Tower

The dynamic characteristics of this bell tower were measured in-situ through a series of free vibration tests that were performed following the damaging earthquake sequence 3rd of February 2014 seismic. The dynamic characteristics of this bell tower were measured in-situ through a series of free vibration tests that were performed following the damaging earthquake sequence 3rd of February 2014 seismic. A set of two tri-axial accelerometers were utilized together with a digital data acquisition system. These accelerometers were fixed at two locations along the height of the bell tower. The first accelerometer was fixed on a horizontal reinforced concrete (R/C) slabs which was built internally on this bell tower and was connected in a monolithic way with all four peripheral R/C walls. This R/C slab was immediately below the bells and could be reached through a R/C staircase which was also built internally and was also connected in a monolithic way with the peripheral R/C walls. This R/C slab was located at 15.4m from the ground level, which is named here “High Measuring Point (up)” and the accelerometer fixed at this level is depicted in figure 2a. The second tri-axial accelerometer was also fixed internally on the West R/C wall at a location 11.5m from ground level, which is named here “Low Measuring Point (down)”. It can be seen that both instruments were located at an axis of symmetry of this structure named y-y in figure 2a (East-West).

The bell tower was excited by a pull-out free vibration test sequence ([20], [21] and [22]). More details are included in [21]. Figure 3 depicts the measured bell tower response at the low and high measuring point during a pull-out in-situ test. The presence of a quite dominant frequency is quite clear in these free vibartion dynamic acceleration responses. This dominant response frequency, which is equal to 2.343Hz (0.427 sec. period), was extracted from transforming the obtained measurements in the frequency domain through a Fast Fourier Transform (FFT) algorithm.

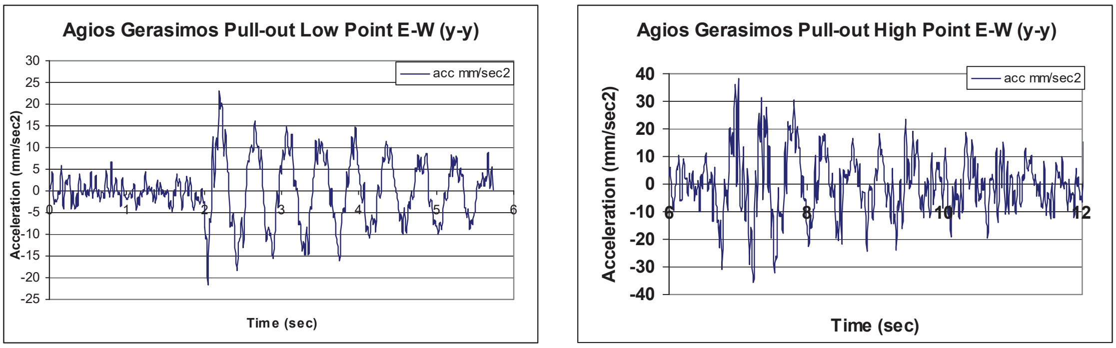


Figure 3. Free vibration horizontal acceleration response at the "Low and High measuring points" of the Agios Gerasimos bell tower at Lixouri

2.3. In-situ Measurements of the Bell Tower belonging to the Panayia Agriliotissa.

The church of Panayia Agriliotissa church at Havriata village, built in 1693, is partially supported by a backfill and a system of retaining walls depicted in figure 4. The dynamic characteristics of this bell tower were measured in-situ through a series of pull-out excitations tests (see Manos et al, [21]), which were performed on 10th October, 2015, approximately twenty (20) months after the damaging earthquake sequence of 3rd 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. 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 4 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).

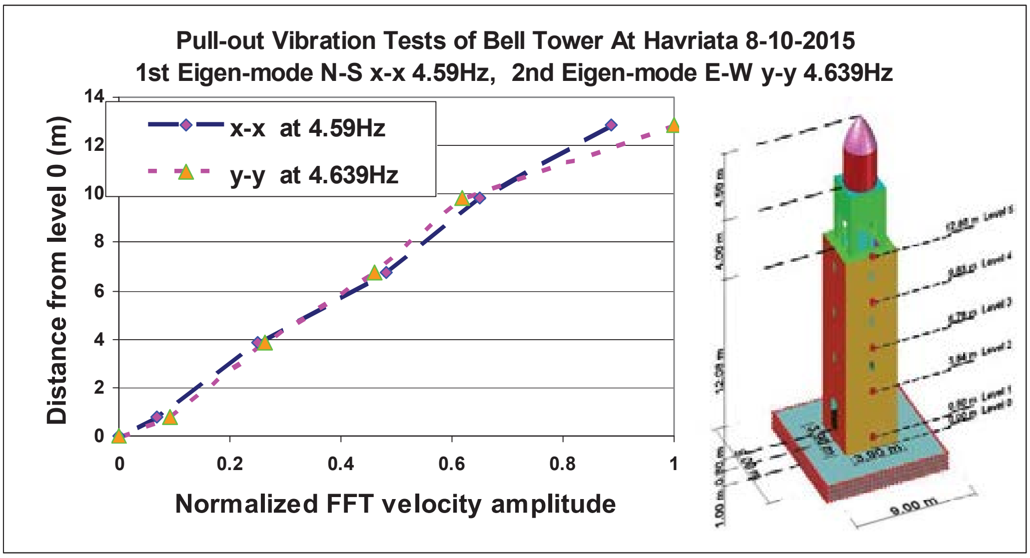


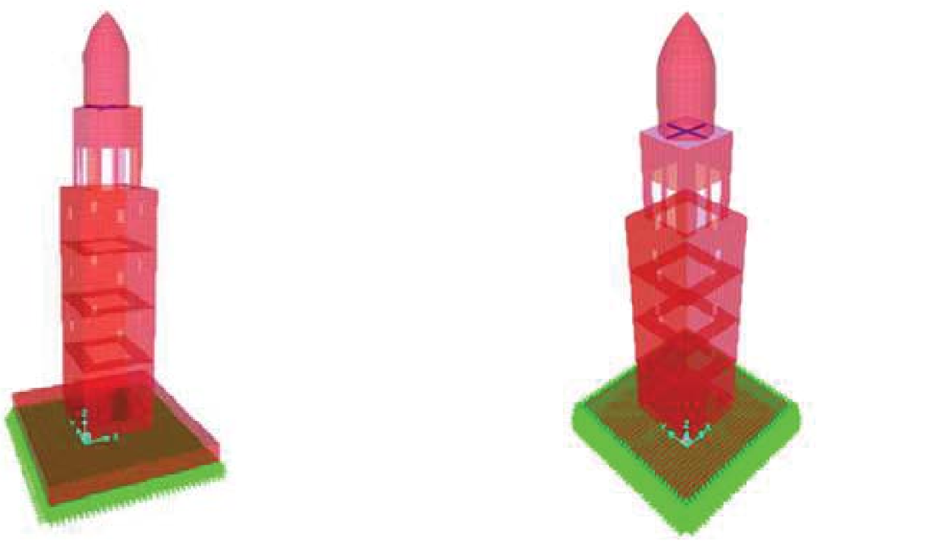
Figure 4. FFT velocity response amplitudes along the height of the Havriata bell tower for the main eigen-frequencies.

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 4 (left) 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 4 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.

3 NUMERICAL SIMULATION OF THE DYNAMIC AND EARTHQUAKE RESPONSE OF THE PANAYIA AGRILIOTISSA BELL TOWER AT HAVRIATA

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 500kgf. 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 5. 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 (figure 5):

- 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]).

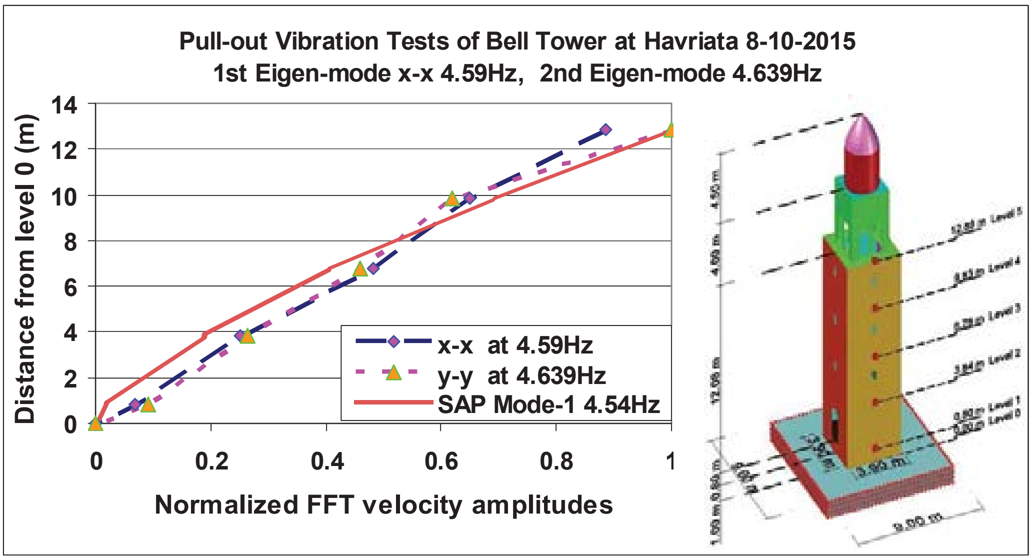


Figure 6. Comparison of measured and predicted dynamic characteristics of the Havriata bell tower.

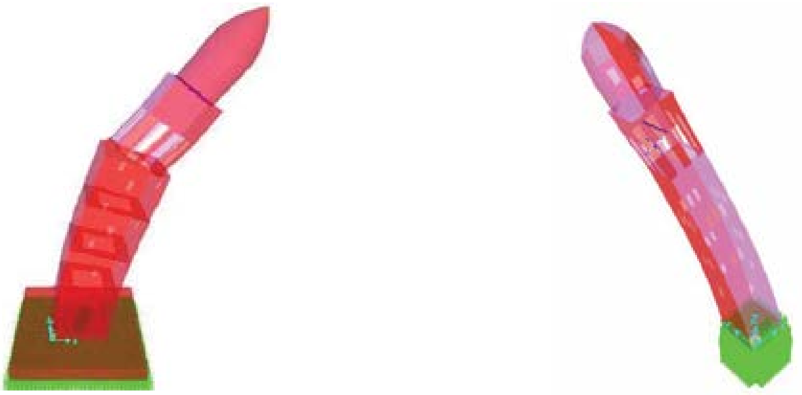
In figure 6 the numerically predicted N-S / E-W eigen-frequencies values and eigen-mode shapes (SAP Mode-1) are compared with the corresponding measurements (N-S x-x and E-W y-y). Only the numerical results for the x-x 1st mode are plotted in this figure because the corresponding results for the second 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 7a. The damaged longidutinal peripheral masonry South wall of the church after the seismic event of 3rd February 2014

Figure 7b. The failure of the retaining wall and the backfill at the South side of The Theotokos church of Havriata

Figure 7b show 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. Moreover, the church of Panayia Agriliotissa of Havriata developed also significant structural damage (figure 7a). 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).

It must be underlined that the Havriata bell tower was designed and constructed well after 1996, when the provisions of the new Greek seismic code became mandatory ([11], [23], [24]). 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.



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 8. 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 8, 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 the same figure the measured 1st eigen-frequency and eigen-mode in either the North-West or the North-South direction is also indicated. A good agreement can be seen between the measured and the numerically predicted values. It must be underlined that in the case of the Havriata bell tower the foundation block was supported at its base to the soil with a sereies of link elements representing stiff-soil conditions. In figure 8a and 8b, 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.

4.1. Results from the design spectral curves

In this section the performance of the Havriata bel tower will be studied adopting earthquake force levels according to the type-2 Euro-Code 8 design spectra for soil category D. Moreover, the design ground acceleration for the island of Kefalonia, as specified by the seismic zoning map of the new seismic code of Greece, equal to 0.36g (g the acceleration of gravity) is also adopted. The importance factor for the bell tower is set equal to 1. A critical point is the value of the behaviour factor (q).

$q = q_o k_w \geq 1.5$

where: q_o is the basic value of the behaviour factor, dependent on the type of the structural system and on its regularity in elevation;

k_w is the factor reflecting the prevailing failure mode in structural systems with walls

For reinforced concrete structures that are formed by uncoupled walls, as one can classify the Havriata bell tower, the minimum value for the behaviour factor is q_o =3 [23]. This value can become even higher when the detailing ensures behaviour of high ductility. At the same time, because of the dimensions of the bell tower and the presence of the bells at is upper part, such a structure could be considered as partly resembling an inverted pendulum in which case the behaviour factor value ranges from q_o =1.5 to 2.0. The value of q_o given for inverted pendulum systems may be increased, if it can be shown that a correspondingly higher energy dissipation is ensured in the critical region of the structure. The value of k_w for the bell tower can range from 1 to 1.43. Part of the concept of high values for the behaviour factor is based on the fact that the strong seismic ground motion effect will be absorbed from ductile inelastic hysteretic response of its structural members and thus reduce the overall structural response in a way similar to that of an overdamped elastic response associated to reduced levels of seismic forces. For the purposes of this study a value of q=3 could be adopted for the behaviour factor considering this bell tower as an RC wall system. In a complementary study the inelastic response spectral curves were derived for the two horizontal components of the ground motion recorded at Havriata at a small distance of the bell tower during the strongest event on the 3rd of February, 2014. These inelastic spectral curves were derived for an assumed ductility factor value m=3, to be in accordance with the adopted value of the behaviour factor adopted for the design spectra derived from the provisions of Euro-Code 8, as described above. These are depicted in the following figure 9 for the East-West and the North-South horizontal components of the named recording of the ground motion at Havriata.

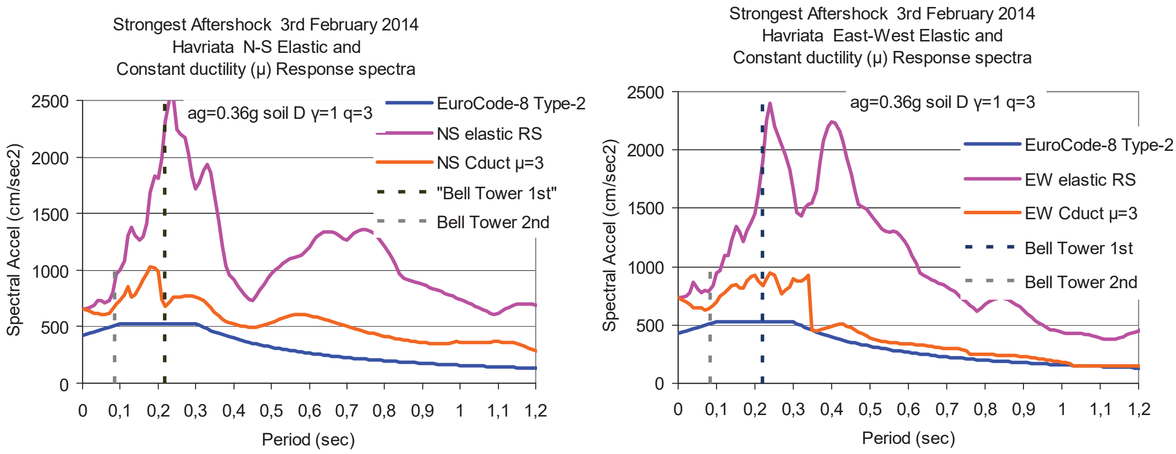


Figure 9. The response and the design spectra for the Havriata Bell tower

In figure 9, the eigen-period values of the 1st and 2nd eigen-modes in either the North-South or the East-West directions are also indicated. Apart from the Euro-Code-8 design spectral curves and the inelastic (m=3) constant ductility spectral curves of the Havriata strong motion record in the two horizontal directions, the corresponding elastic spectra curves are also plotted. As can be seen from these plots, the inelastic constant ductility curve values are larger than the corresponding Euro-Code 8 design spectral values for both eigen-periods and for both directions. Similarly, the elastic response spectral curve values are much higher than the corresponding inelastic constant ductility (m=3) spectral curves. Moreover, the following observations can be made.

a) The 1st eigen-period value for the Havriata bell tower corresponds to the period range with the relatively large spectral values. This is also the case for the Agios Gerasimos bell tower.

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). These numerically predicted eigen-period values were validate by the in-situ measurements.

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 Lixouri ground motion record [1] are compared with the corresponding response spectral curves of 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.

The modal analysis results of the numerical model of the Havriata bell tower resting on links representing stiff soil conditions are listed in the following table. In this numerical model the part that simulates the foundation block which is below the ground surface is represented by a rigid box formed by relatively rigid finite elements in a way that the weight of the actual foundation block is realistically approximated. At the same time these elements are assumed to be practically mass-less as not to interfere in an unrealistic way with the dynamic system of this structure. Moreover, in order to account for the influence of soil deformability the link elements supporting the foundation block to the soil were provided with a damping ratio equal to 10% of critical.

Table . Modal Participating Mass Ratios

Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ
Number	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
1	**0,22**	**0,52381**			0,52381	0	0
2	**0,219**		**0,5114**		0,52381	0,5114	0
3	**0,086**	**0,21565**			0,73946	0,5114	0
4	**0,084**		**0,2085**		0,73946	0,7199	0
5	0,062				0,73952	0,7199	0
6	**0,05**			**0,51**	0,73952	0,7199	0,51
7	0,041	0,06824			0,80776	0,7199	0,51
8	0,04		0,0646		0,80776	0,7845	0,51
9	0,039	0,00473			0,81249	0,7845	0,51
10	0,032	0,05812			**0,87061**	**0,7845**	**0,51**

Table . Base Reactions

Load case	CaseType		GlobalFX (tnf)	GlobalFY (tnf)	GlobalFZ (tnf)
DEAD (with Foundation)	LinStatic	Max			**534,6 / 225,6***
1. EuroCode 8 q=3	LinRespSpec	Max	**70,1 / 13%**	**70,1 / 13%**	
2. EW-res spectra Con. Duc. m=3	LinRespSpec	Max		**104,4 / 20%**	
3. NS-res spectra Con. Duc. m=3	LinRespSpec	Max	**90,9 / 17%**		
4. Dead + EC8x-x + 0.3EC8y-y	LinRespSpec	Max	**70,1 / 13%**	**20.4**	**534,6 / 225,6***
5. Dead+0.3 EC8x-x + EC8y-y	LinRespSpec	Max	**21.0**	**67.8**	**534,6 / 225,6***
6. Dead+(RSEW + 0.3RSNS) m=3	LinRespSpec	Max	**27.3**	**104.4**	**534,6 / 225,6***
7. Dead+(RSEW + 0.3RSNS) m=3	LinRespSpec	Max	**90.9**	**31.3**	**534,6 / 225,6***

- with and without the weight of the foundation block. The listed percentage values in this table were derived from comparing the base shear values in each case with the total weight of the bell tower including the foundation

As can be seen from the modal participating mass ratio values listed in the above table the 1st and second translational modes in the x-x (North-South) and the y-y (East-West) direction mobilize a considerable part (more than 72%) of the total mass of the superstructure. The rest of the higher order modes used in the subsequent dynamic spectral analyses (up to 10 modes) mobilize up to 87% and 78.5% of the total mass of the superstructure in the x-x and y-y horizontal directions, respectively. Therefore, no further amplification of the dynamic response was employed. The resulting base shear values are listed in Table . The listed percentage values in this table were derived from comparing the base shear values in each case with the total weight of the bell tower including the foundation. As can be seen, the seismic force levels when the constant ductility response spectra curves are employed are higher (20% of the total weight) than those employing the Euro-Code-8 design spectral curves (13% of the total weight). The same is obviously valid when the load combinations, described in load cases No. 6 and 7 (constant ductility response spectra) are compared with load cases No. 4 and 5 (Euro-Code-8 design spectra).

Load Cases	Top of Tower Hor. Displ. (mm)	Base Shear (tnf)	Over Moment (tnf-m)	Vertical link force* (tnf)
4. Dead + EC8x-x + 0.3EC8y-y	Ux=14.75 / Uy=4.48	Qx=70 Qy=20	Mx=1301 My=-1911	0.14 / -2.09
5. Dead + 0.3 EC8x-x + EC8y-y	Ux=4.43 / Uy=14.69	Qx=21 Qy=67	Mx=1900 My=-1305	0.06 / -2.06
6. Dead + (RSEW + 0.3RSNS) m=3	Ux= 5.72 / Uy=23.11	Qx=27 Qy=104	Mx=2389 My=-1381	0.60 / -2.06
7. Dead + (RSNS + 0.3RSEW) m=3	Ux=19.06 / Uy=7.01	Qx=91 Qy=31	Mx=1449 My=-2164	0.55 / -2.50

* The vertical force that develops at the link connecting the foundation block with the soil located at the North-West corner of this foundation block

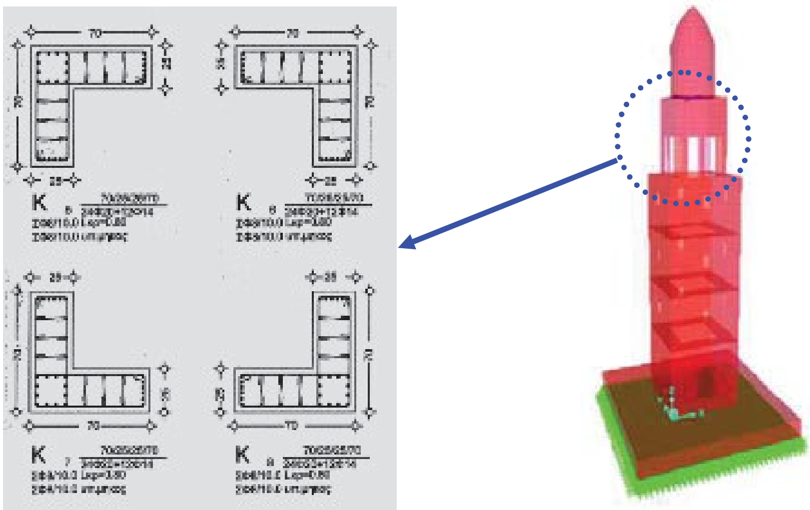


Figure 10. Structural detailing of the columns at the level supporting the bells

Figure 10 depicts the structural detailing of the part of the bell tower with the columns above the level 12.00m, which as depicted in figures 11, is highly stressed both in tension and shear from the load combinations that include either the EuroCode-8 design spectral curves or the constant ductility response spectral curves of the recorded ground motion during the strongest earthquake event of the 2014 sequence. This bell tower was designed according to the provisions of the Euro-code 8 by a designer civil engineer (see acknowledgements) assuming design ground acceleration 0.36g, behaviour factor q=3, importance factor γ=1, and soil category B according to the provisions of the new Greek seismic code. The quality of the concrete was C20/25 and the reinforcing steel S400s (assumed yield stress equal to 400MPa). As was shown in figure 10, the ineleastic spectral acceleration curves derived from the recorded ground motion assuming ductility factor equal to m=3 result in base shear values approximately 50% higher than the corresponding base shear values derived from the Euro-Code-8 design spectral values (see Table). It was underlined that no structural damage could be seen at this bell tower after the event despite the fact that it was subjected to earthquake forces exceeding the level of the assumed earthquake forces in the design. The same can be seen when one compares the maximum displacement response values predicted to develop at the top of the bell tower employing the Euro-Code-8 design spectra with the corresponding constant ductility response spectra curves. The same can be seen when on compares the stress levels that develop at the columns of the part of the bell tower below the level where the bells are located. This part of the structure is selected because the columns located at this part of the whole structural system develop relatively high level of stresses. These stress levels can be met successfully by the reinforcing details of these columns.

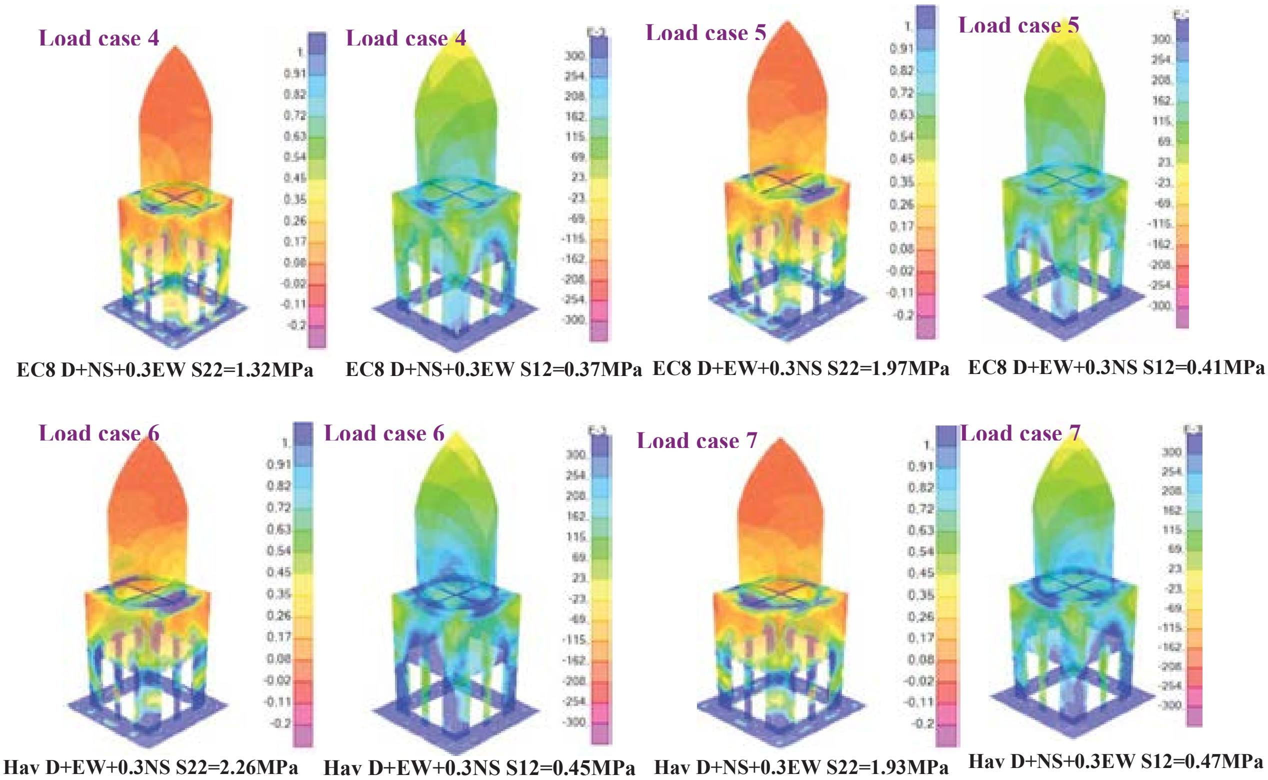


Figure 11. Stress levels that develop at the part of the bell tower supporting the bells for the various loading conditions

4 CONCLUSIONS

- The measured 1st eigen-period values compare well with the numerically predicted values for both bell towers. The 1st eigen-period value for the Havriata bell tower corresponds to the period range with the relatively large spectral values. This is also the case for the Agios Gerasimos bell tower.
- 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). The employed numerical simulations recognized this fact by employing the appropriate support conditions of the foundation block.

- The seismic force levels when the constant ductility response spectra curves are employed are higher (20% of the total weight) than those employing the Euro-Code-8 design spectral curves (13% of the total weight).
- The bell tower columns above the level 12.00m, represent the part of the bell tower which is highly stressed both in tension and shear.
- No structural damage could be seen at this bell tower after the event despite the fact that it was subjected to earthquake forces exceeding the level of the assumed earthquake forces in the design.

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- To the memory of Nick Simos, Senior Researcher, Brookhaven National Labotatory, U.S.A.

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