Earthquake Performance of Bell Towers in the Island of Kefalonia, Greece

George C. Manos
Laboratory of Strength of Materials and Structures, Aristotle University, Thessaloniki 54006, Greece

Abstract: The dynamic and earthquake response of bell towers, located at the Island of Kefalonia, Greece, is examined here. These structures were subjected during the winter of 2014 to an intensive earthquake sequence. The dynamic characteristics of two bell towers were measured in situ. Subsequently the dynamic and earthquake response of each bell tower was numerically simulated employing 3-D dynamic elastic numerical simulations taking into account the soil-foundation deformability. It is demonstrated that the soil-foundation-structure interaction influences the dynamic and earthquake response predictions for this structure quite significantly. It also demonstrates the usefulness of such in-situ testing towards formulating realistic numerical models in order to yield realistic predictions of the dynamic and earthquake response of the examined structures. The obtained numerical analyses utilize the earthquake ground motion which was recorded at close distance from both bell towers. The numerical predictions of the earthquake response of both bell towers are utilized to draw conclusions of their actual earthquake performance. It is concluded that the soil-foundation interaction was a critical response mechanism. The newly built RC (reinforced concrete) bell towers performed satisfactorily.

Key words: Earthquake response, bell towers, in-situ system identification, soil-foundation interaction.

1. Introduction

Bell towers are structures that are of particular interest regarding their dynamic and earthquake response. A large number of bell towers with relatively large dimensions are located in numerous cities in Italy and elsewhere. Most of these bell towers are built by stone or brick masonry. In many cases earthquake activity constitutes the major cause of serious damage for bell towers and leads to partial or total collapse. Consequently, there is a major concern for the stability of numerous bell towers. This resulted in 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 [1-8]. Foundation problems for bell towers are evident, in many cases 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 [9-12]. The purpose of this investigation is to study the seismic performance of relatively new bell towers compared to the old ones. 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 [13, 14]. From the historical seismicity it is estimated that at least 20 earthquakes over Magnitude 6.2 of the Richter scale, and at least 8 amongst them over Magnitude 7.0 of the Richter scale have occurred at this island over the last 600 years [15]. In 1953 a very destructive earthquake damaged numerous bell towers as indicated by Figs. 1 and 2. In Fig. 3a, a special tax bond issued by the Greek government to assist the devastated island depicts one of the collapsed bell towers. In Fig. 3b, a bell tower damaged by the recent 2014 earthquake sequence is shown. However, this represents an
exception; in contrast, most of the newly built bell towers since 1953 performed satisfactorily despite the severity of the ground shaking, as this is documented by the recorded earthquake ground motion.

Fig. 1  The capital city of the island (Argostoli) prior to the 1953 earthquake. Six bell towers are indicated.

Fig. 2  The capital city of the island (Argostoli) immediately after the 1953 earthquake.

Fig. 3  (a) The tax bond of 1953; (b) The Kourouklata bell tower after the 2014 earthquake event.

The objective of this work is to study the dynamic and seismic response of two relatively newly built quite tall bell towers in order to analyze their satisfactory performance. These bell towers are: The Agios Gerasimos bell tower at Lixouri [16] and the bell tower of Panayia Agriliotissa at Havriata [17]. They are selected because for both of them the ground acceleration during the most intense seismic 2014 event was recorded at a close distance. Detailed information can be found in previous publications [16, 17]. Therefore, only summary information is given here.

2. Methodology

Initially, the basic geometry of each structure and its foundation block was specified, depicted in Figs. 4 and 5. Next, through a system identification study the fundamental vibration characteristics of each bell tower were defined. Based on this information 3-D linear-elastic numerical simulations were formed including the ability to represent the soil-foundation deformability. Dynamic spectral analyses were performed employing these numerical models together with the 2014 earthquake ground motion recordings. The obtained results are presented and discussed in the form of horizontal base shear force or horizontal displacement amplitudes.

2.1 In-Situ System Identification

The system identification utilized free vibration excitations introduced by the sudden release of a steel cable which was rigidly attached at certain height of each bell tower, as shown by the red arrows in Figs. 4 and 5. The dynamic free vibration response was recorded by a number of sensors placed along the height of each bell tower. Through signal analysis process the fundamental horizontal translational eigen-frequencies and eigen-modes were identified. For the Agios Gerasimos bell tower the East-West eigen-frequency was equal to 2.343 Hz [16].
For the bell tower of Panayia Agriliotissa at Havriata [17] the East-West and the North-South eigen-frequencies were equal to 4.639 Hz and 4.590 Hz, respectively.

2.2 3-D Numerical Simulation of Agios Geasimos Bell Tower

Shell elements were used to represent the RC (reinforced concrete) walls of this bell tower assuming a Young’s Modulus equal to \( E = 10,000 \) MPa. The bells were assumed to weight 500 kg. Information on the soil-foundation interaction can be found in Ref. [16] employing a combination of rigid slabs and flexible links in a parametric study. The eigen-mode and eigen-frequency numerical predictions are depicted in Fig. 6. The best agreement with the corresponding eigen-frequency value (2.34 Hz) obtained from the in-situ measured dynamic response was derived for flexible links with axial stiffness value corresponding to medium stiffness soil conditions.

2.3 3-D Numerical Simulation of the Bell Tower of Panayia Agriliotissa at Havriata

The same procedure as before was followed for this bell tower. The eigen-mode and eigen-frequency numerical predictions are depicted in Fig. 7. The best agreement with the corresponding eigen-frequency values (4.639 Hz and 4.590 Hz) obtained from the in-situ measured dynamic response was derived for
links at the soil-foundation interface with axial stiffness value corresponding to relatively stiff soil conditions. For both bell towers, the obtained agreement between the numerically predicted and the measured in-situ fundamental eigen-frequency values ensures the validity of the corresponding numerical models. Therefore, these numerical models are used in the subsequent dynamic analysis for predicting their earthquake response.

3. Numerical Simulation of the Earthquake Response of the Agios Gerasimos Bell Tower

The numerical model described in Section 2.2 is employed together with the spectral curves of the ground acceleration recorded at a distance approximately 350 m from this bell tower [18]. Because of the proximity of the bell tower to the location of recording the ground acceleration during this seismic event it is concluded that the seismic input is represented with very good accuracy in the subsequent numerical analysis. The characteristics of this acceleration ground motion are depicted in Fig. 8. In this figure, the two constant ductility ($\mu = 3, \zeta = 5\%$) horizontal acceleration spectral curves (Lixouri E-W and Lixouri N-S) are compared with the Euro-Code 8 design spectral curves [19] (Type-1 and Type-2) for ground design acceleration 0.36 g (Seismic Zone III for Greece) and soil category D (soft soil). The value of the response modification coefficient ($q$) was assumed equal to 3 which is in line with the ductility coefficient value ($\mu = 3, \zeta = 5\%$). In the same plot the “Old Greek Code” (prior to 1992) design acceleration is also plotted. Assuming that the earthquake response of the bell tower is dominated by the two fundamental translational modes depicted in Fig. 6, the relevant eigen-period value ($T = 0.426$ s) is also indicated in this plot.

The obtained peak earthquake response, in terms of horizontal base shear and horizontal displacement at the top of the Agios Gerasimos bell tower, is listed in Table 1. This is done for load combinations of the dead load ($D = 2,081$ kN including the weight of the foundation block) and either the East-West response spectral curve (Comb3) or the North-South response spectral curve (Comb4), which were obtained from the recorded acceleration of the ground motion at Lixouri

![Fig. 7](image)

*Fig. 7* (a) Translational mode E-W ($y-y$). $T = 0.219$ s, $f = 4.5703$ Hz (stiff soil); (b) Translational mode N-S ($x-x$). $T = 0.218$ s, $f = 4.59$ Hz (stiff soil).

![Fig. 8](image)

*Fig. 8* Constant ductility spectra; curves based on the recorded ground acceleration together with design spectral acceleration curves.

| Load combinations | Base shear ($kN$) | Top displ. ($mm$) |
|-------------------|------------------|------------------|
| Comb3 D + R. Spectrum (E-W) | $Q_y = 472$ | $U_y = 33.8$ |
| Comb4 D + R. Spectrum (N-S) | $Q_x = 411$ | $U_x = 35.3$ |
| Comb5 D + Time history (N-S) + Time history (E-W) | $Q_y = 1,246$ | $U_y = 102.5$ |
| | $Q_x = 1,249$ | $U_x = 88.3$ |
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The peak horizontal base shear of load combinations 3 and 4 represents approximately 23% of the total weight (super structure + foundation block). As can be seen in Fig. 8, for the dominant translational eigen-period values the inelastic ($\mu = 3$) spectral curve values (either E-W or N-S for the recorded ground motion) are twice as large as the Euro-Code 8 design spectral values ($q = 3$) and four times larger than the “Old Greek Seismic Code” design acceleration value. This is indicative of the intensity of this specific earthquake excitation for this structure. Despite these high force levels, the peak horizontal displacement at the top of the bell tower represents 0.165% of the total bell tower height. This is partly due to the fact that inelastic response was assumed by employing constant ductility inelastic response spectral curves ($\mu = 3$) although they were no visible signs of such ductile response mechanisms. This will be further investigated for the bell tower of Panayia Agriliotissa at Havriata. Combination 5 in Table 1 employs directly the horizontal N-S and E-W ground acceleration components and an elastic dynamic time history analysis. From the relevant values listed in Table 1 the peak horizontal base shear value is this time 60% of the total bell tower weight whereas the peak top horizontal displacement is 0.495% of the bell tower height.

4. Numerical Simulation of the Earthquake Response of the Bell Tower of Panayia Agriliotissa at Havriata

The methodology employed for the Agios Gerasimos bell tower is next applied for the bell tower of Panayia Agriliotissa at Havriata. Again use is made of the spectral curves of the ground acceleration recorded at a close distance approximately 300 m from this bell tower [18]. Figs. 9 and 10 depict the elastic ($\xi = 5\%$) and the constant ductility ($\mu = 3$, $\xi = 5\%$) acceleration response spectral curves for the East-West and North-South horizontal components of the recorded acceleration for this earthquake ground motion, respectively. The significant reduction introduced by the inelastic response in the spectral amplitudes is evident in both plots. The Euro-Code 8 design spectral curves [19] (Type-2) for ground design acceleration 0.36 g (Seismic Zone III for Greece) and soil category D (soft soil) is also plotted for comparison, adopting a behaviour factor value ($q$) equal to 3.

Again the Euro-Code 8 inelastic design spectral curves ($q = 3$) are of almost 50% smaller amplitude than the constant ductility ($\mu = 3$) spectral curves obtained from this recorded ground motion. The 1st fundamental translational eigen-period (0.218 s) is indicated in both these plots demonstrating that it lies within the period range of relatively high spectral acceleration amplitudes. It can be concluded that this bell tower would have been subjected to very intense shaking, almost twice the acceleration of gravity, unless it could develop certain non-linear inelastic response mechanisms that could reduce the level of horizontal seismic forces, as it is indicated by the constant ductility response spectral curves.

As already mentioned, a critical point is the value of the behaviour factor ($q$).

$$q = q_o \cdot k_w \geq 1.5 \quad (1)$$

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 RC structures that are formed by uncoupled walls, as one can classify the Havriata bell tower, the minimum value for the behaviour factor is $q_o = 3$ [19]. 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 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 reduces the overall structural response in a way similar to that of an overdamped elastic response associated to reduced levels of seismic forces.

Table 2 Significant modal mass ratios.

| Mode No. | Period (s) | Modal mass ratio (% of mass of the superstructure) |
|----------|------------|--------------------------------------------------|
|          |            | $U_x$ | $U_y$ | Sum $U_x$ | Sum $U_y$ |
| 1        | 0.22       | 52.4% | 52.4% | 0          |
| 2        | 0.219      | 51.1% | 52.4% | 51.1%      |
| 3        | 0.086      | 21.6% | 73.9% | 51.1%      |
| 4        | 0.084      | 20.9% | 73.9% | 72.0%      |

Inspection of all the visible components of the superstructure did not reveal the developments of such inelastic response of the various RC structural components of the superstructure. Alternatively, such response mechanism could develop between the foundation block and the surrounding soil and is investigated next.

5. Further Numerical Study of the Earthquake Response of the Bell Tower of Panayia Agriliotissa at Havriata

In this section the performance of the Havriata bell tower is studied in specific distinct steps. All the numerical models include the superstructure rigidly connected to a non-deformable foundation having horizontal dimensions $9 \text{ m} \times 9 \text{ m}$ and a thickness of $1 \text{ m}$ approximating in this way the actual foundation of this bell tower. This foundation block is supported with a number of linear elastic 3-D links which are provided with an increased damping capacity ($\xi = 10\%$). The obtained modal mass ratio values are listed in Table 2 whereby the first four translational eigen-modes correspond to cumulative modal mass ratio above 70% for either the $x$-$x$ or the $y$-$y$ directions.

Table 3 lists the horizontal base shear values which were predicted from a series of dynamic analyses, either along the East-West ($y$-$y$) or the North-South ($y$-$y$) direction. EC8 denotes dynamic spectral analyses using the Euro-Code 8 design spectrum ($q = 3$, Figs. 9 and 10). EW-rs or NS-rs denote dynamic spectral analyses using the constant ductility response spectrum ($\mu = 3$, East-West or North-South) whereas EW-rs elastic or NS-rs elastic denote dynamic spectral
Table 3  Predicted base shear values.

| Load case                  | Base shear \(x-x\) (kN) | Base shear \(y-y\) (kN) |
|----------------------------|-------------------------|-------------------------|
| 1. EC8x + 0.3EC8y          | 688/12.9%               | 200/3.8%                |
| 2. 0.3EC8x + EC8y          | 206/3.9%                | 665/12.5%               |
| 3. EW-rs + 0.3NS-rs        | 268/5.0%                | 1,024/19.2%             |
| 4. NS-rs + 0.3EW-rs        | 892/16.7%               | 307/9.3%                |
| 5. EW-rs elastic           |                         | 2,241/58%               |
| 6. NS-rs elastic           | 2,794/53%               |                         |
| 7. EW time history          |                         | 2,714/52%               |
| 8. EW time history          |                         | -3,114/60%              |
| 9. NS time history          | 3,377/66%               |                         |
| 10. NS time history         | -3,280/65%              |                         |

Table 4  Predicted peak response values.

| Load case                  | Horiz. displ. At top (mm) | Link force Vertical (kN) |
|----------------------------|----------------------------|-------------------------|
| 1. EC8x + 0.3EC8y          | \(U_x = 14.75\)/\(U_y = 4.48\) | 1.4/-20.5              |
| 2. 0.3EC8x + EC8y          | \(U_x = 4.43/\ U_y = 14.69\) | 0.6/-20.2              |
| 3. EW-rs + 0.3NS-rs        | \(U_x = 5.72/\ U_y = 23.11\) | 5.9/-20.2              |
| 4. NS-rs + 0.3EW-rs        | \(U_x = 19.06/\ U_y = 7.01\) | 5.4/-24.5              |
| 5. EW-rs elastic           | \(U_y = 52.2\)           | 18.0/-38.1             |
| 6. NS-rs elastic           | \(U_x = 64.3\)           | 28.9/-55.9             |
| 7/8. EW Time His.          | \(U_y = 55.2/-52.5\)     | 13.5/-11.9             |
| 9/10. NS Time His.         | \(U_x = 68.2/-63.2\)     | 31.1/-32.9             |

analyses using instead the elastic response spectrum (Figs. 9 and 10). Finally, horizontal base shear values were obtained using the corresponding East-West or North-South ground acceleration records through a time history dynamic elastic analysis. The value in % what percentage of the total bell tower weigh (including the weight of the foundation block) each of the listed base shear values represents. In all these dynamic analyses the dead weight of the superstructure and the foundation block is superimposed. These results confirm that all base shear values, obtained from either the elastic response spectral curves or the horizontal acceleration components of the recorded ground motion, represent horizontal forces larger than 50% of the total bell tower weight. On the contrary, the base shear values obtained when using either the Euro-Code 8 design spectrum (\(q = 3\)) or the constant ductility response spectrum (\(\mu = 3\)) represent horizontal forces below 20% of the total bell tower weight.

Table 4 lists the predicted peak horizontal displacement values at the top of the bell tower together with the extreme axial forces that arise at the 1,024 vertical links that support the foundation block to the underlying soil. A positive link force sign indicates that foundation block at the particular location of this link has the tendency to uplift from the underlying soil. This is the result of the overturning moment at the foundation-soil interface that arises from the horizontal seismic forces and leads to extreme positive or negative link force values at the edges of the foundation block. As can be seen from the amplitude of the values of these link forces listed at the last column of Table 4, this tendency is obviously more pronounced for the linear elastic response analyses than for the dynamic analyses employing either the Euro-Code design spectrum or the constant ductility response spectra of Figs. 9 and 10. This observation confirms that the soil-foundation interaction is the non-linear response mechanism that partly explains the performance of these newly built bell towers.

6. A Non-linear Push-Over Numerical Study of the Earthquake Response of the Bell Tower of Panayia Agriliotissa at Havriata

A number of non-linear analyses were performed whereby the uplifting mechanism of the foundation block from the underlying soil was numerically simulated by providing the vertical support links of the foundation block with a non-linear constitutive behaviour shown in Fig. 11. As can be seen each link could develop a non-linear compression representative of a relatively moderately stiff soil whereas it could sustain comparatively very little tension, thus allowing in this way such a possible non-linear uplift mechanism of the foundation to be numerically simulated. The compressive soil behaviour has not yet been validated with data from in-situ soil samples.

Due to space limitations the results presented here belong to a static one direction “push-over” type-I of
analysis whereby the dead load is initially applied followed by applying a system of horizontal forces at the upper part of the bell tower, as depicted in Fig. 12. This system of forces is applied in a continuously increasing manner from the West to the East. This system of horizontal forces results in a horizontal displacement distribution along the bell-tower height similar to the dominant 1st translational eigen-mode in this direction (see Fig. 7). Moreover, the non-linear constitutive behaviour of each vertical support link is multi-linear elastic.

The obtained response is plotted in Fig. 13 in terms of the variation of the East-West horizontal base shear against the variation of the corresponding horizontal displacement predicted numerically at three specific levels of the bell tower indicated in Fig. 12: (a) at the top of the bell tower; (b) at the base of the bells; (c) at the base of the columns.

This is also shown in Fig. 14 in terms of the variation of the overturning moment at foundation-soil interface versus the horizontal East-West displacements predicted at the three levels along the height of the bell tower denoted in Fig. 12.

From both Figs. 13 and 14 the variation of either the base shear or the overturning moment versus the
horizontal displacement exhibits strong non-linear trends for base shear value approximately equal to 1,000 kN (corresponding overturning moment value approximately equal to 16,000 kNm). The corresponding horizontal displacement value at the top is approximately equal to 50 mm. From this point an increase of the base shear value results in a noticeable amplification in the corresponding increase of the horizontal displacement amplitude. It is interesting to note that the peak base shear predicted values listed in Table 4 for either employing the Euro-Code 8 design spectrum or the constant ductility acceleration response spectrum are equal or below the 1,000 kN level. On the contrary, the peak predicted base shear values based on elastic response (obtained from either dynamic spectral or time history analyses) are three times larger than the 1,000 kN level. Consequently, the qualitative observations stated at the end of Section 4 are confirmed from the results of the current “push-over” non-linear numerical simulation.

This uplifting of the foundation block non-linear response mechanism is shown in Fig. 15 whereby the state of axial forces at the links supporting the foundation block is plotted. At this instant of the push-over analysis a large part of the foundation block is uplifted (light colour) with a relatively narrow part of the east side of this block under compression (red colour). This is also depicted by the variation of the link axial forces in Fig. 16 for vertical links located at specific corners of the foundation block.

7. Conclusions

(1) In contrast to severe damage and collapse of bell towers in the past the newly built RC bell towers in the island of Kefalonia-Greece performed in a satisfactory way despite the severity of the seismic ground motion.

(2) Critical parameter was the design of the superstructure and the foundation block of the newly built bell towers that took into account the current high levels of design ground acceleration specified by the new Greek seismic code.

(3) A contributing factor in the performance of these newly built bell towers is the one that allowed for non-linear response mechanism at the soil-foundation interface without developing undesirable forms of damage either to the foundation or the soil volume.

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