ABSTRACT

The Aegosthena fortress was built in western Attica at around 340 B.C. The northeastern tower, already severely damaged in the course of time, underwent partial collapse in 1981 following the earthquake which took place at the nearby Corinthian Gulf. The study for the monument’s restoration was carried out by the Directorate for Restoration of Ancient Monuments. First, constructional analysis of the tower and investigation of the mechanism of collapse will be summarily presented, followed by discussion on the computational analysis strategy adopted for the study of the tower’s structural behavior and the definition of the necessary strengthening measures, taking into account the need to preserve the authentic construction system of dry-stack masonry. The strategy comprised two phases. The first aimed at qualitatively understanding the weak points of the simply rebuilt (without additional interventions) structure, in order to investigate the need for further measures. To this purpose, spectral dynamic analysis was applied on two continuous models with shell elements, simulating the simply rebuilt tower and the rebuilt one with additional interventions. This approach could address the issue of detecting structural weaknesses and of demonstrating the effect of the proposed interventions. The second phase consisted in the dimensioning of those interventions and the control of the repaired structure with the use of the discrete element method. The sequence of a wider applicability approach, followed by a more compatible to the structural system, was found effective in addressing the issue of the anastylosis with the corresponding (for each phase of the study) accuracy.

Keywords: Anastylosis; Ancient Towers; Numerical Modeling

1. INTRODUCTION

The Aegosthena fortress was built at the eastern side of the Corinthian Gulf. It is dated at around 340 B.C. and was part of the Athenian defensive system, which extended all over the Attica region. It consists in an acropolis (citadel) built on top of a steep hill, to protect a vast fortified area extending from the foothill to the sea westwards. The northeastern tower is one of the four towers that reinforce the acropolis’ eastern wall. It is a small, quadrangular dry-stack construction, with external dimensions approximately 7.5m at its lowest level and total remaining height around 11m (Figure 1). The tower had two accessible floors but – being built on the slope of the hill - it is composed of three distinct levels. The eastern and northern elevations are founded outside the fortified area directly on rock substratum, as shown in Figure 1 (left). Below this foundation, the slope extends another 10 m above the ground which surrounds the hill. On the contrary, the western and southern elevations are founded on top of the hill, inside the fortified area, approximately 3.5 m above the foundations of the previously mentioned elevations and 13.5-14 m above the ground. So, a ground level is formed between the hill slope and these two (eastern, northern) elevations. This level is approximately 5m high (corresponding to 10 rows of stone blocks east and north) and was filled with carefully built layers of stones, aggregate and earth. The entrance to the tower was located, as shown in Figure 1 (left) on the first level; it was therefore accessible only through the interior of the citadel. This level is 3.5m high (corresponding to 7 rows) and roughly square in plan. The western and southern elevations are founded on the ground floor’s filling material, with the exception of the S.W. corner, which is founded on a more rigid (though not directly the rock) substratum.
The second level, only partially preserved, hosted the catapults used for the defense of the tower. Its existence is proven by the beam cases carved on the 18th course of stones, 9 m above the ground level (Figure 1 right). The protruding stones at the top of the S.E. corner (Figure 1 left) suggest that, close to the tower, the fortification wall was rising gradually until the 19th row, possibly to protect the external staircase leading to the entrance of the second floor. The tower was standing as shown in Figure 1 (left) until February 24th, 1981. This day, the eastern Corinthian Gulf earthquake has struck the building, resulting mainly in the collapse of the free standing southeastern corner and a big part of the eastern elevation (Figure 1 right). The pathology of the existing structure (Figure 1 right) was extremely severe and further collapse in case of a, even low intensity, earthquake was certain. In addition, the foundations of the western and southern elevations had gradually settled due to the deformability of the filling material, resulting in block subsidence up to 40 cm in the middle of the western elevation. Sliding of the blocks of the still standing southern elevation was evident by the opening of the joints (Figure 1 right). Moreover, the northeastern corner of the tower (on the upper left of Figure 1 right) was in an extremely unstable situation due to the severe opening of joints, the rotation of the remaining blocks outwards and the subsequent loosening of the corner connection.

The study for the monument’s restoration was undertaken by the Directorate for Restoration of Ancient Monuments (DRAM) and comprised two phases; the first one consisted in (1.a) the documentation of the collapsed blocks and study of their original position; and (1.b) the constructional and structural analysis and assessment of the behaviour of the monument, including earthquake behaviour, in view of the selection and pre-dimensioning of the interventions (Toumbakari 2012). The second phase took place after the approval of the aforementioned studies by the Central Archaeological Council and comprised (2.a) the detailed study of the tower’s structural behaviour including the dimensioning of the intervention scheme approved at the first phase; (2.b) geological & geotechnical study, to define the foundation conditions and derive the mechanical constants to be used for the design of the new foundation; (2.c) structural design of the new foundation; (2.d) application study for the anastylosis of the collapsed blocks; and (2.e) worksite design and management (Toumbakari 2015). The paper focuses on the structural studies of the first (1.b) and second (2.a) phases.

2. DESCRIPTION, ASSESSMENT & INTERPRETATION OF THE DAMAGES

2.1 Constructional analysis & structural assessment

The constructional analysis of the tower comprised a detailed survey of the dimensions and deformations of the structure, the state of the foundation (for which a small archaeological excavation was also carried out) and the documentation of the individual blocks. Based on these data, it focused on understanding the peculiar configuration of the monument before 1981, as it was felt that it was the key to the interpretation of the collapse mechanism of the monument. Indeed, if the building blocks of the southern and eastern elevations
were properly interlocked, then the structure could not have been standing in this way but would have rather suffered from collapse of the upper courses due to out-of-plane bending. On the basis of archival photos, the configuration of the missing eastern part of the southern elevation could be gradually reconstructed. Figure 2 shows the result of this study, which consisted in documenting the position and direction of each building block of the broader S.E. corner of the tower on the basis of photographs. To the left of the drawing, the standing stone blocks are shown (in light grey colour). To the right, the identified stones are drawn as full blocks. The middle part, whose configuration remains unknown, has been schematically drawn with the assumption that the length of the individual blocks remains similar to the surveyed ones.

Focusing on the S.E. corner, it can be observed that, instead of interlocking building blocks, as in the case of all other corners, in this particular area a construction joint is formed. The blocks were laid in such a way, so as to connect the eastern elevation not to the southern one, but instead to the fortress’ eastern defensive wall (which runs roughly perpendicular to the southern elevation). This characteristic is typical to all the towers of the acropolis’ eastern fortification wall and can be explained by the choice of the ancient builders to reinforce the external leaf of the curtain walls through this connection. This choice, which is understandable from the points of view of the construction speed as well as the limitation of the damages by catapults, leads, however, to the creation of joints; thus, the interlocking between elevations weakens in favor of the connection to the curtain wall. If, now, the uneven (in terms of height as well as deformability) foundations are considered, the vulnerability of the N.E. tower becomes easily discernable.

2.2 The collapse mechanism

The theoretical reconstruction of the missing southern elevation and the understanding of the connection between the fortification wall and the eastern elevation of the tower permitted the study of its mechanism of collapse. As shown in Figure 3, four stages can be proposed:
Stage I: it consists in the gradual collapse of the wooden beams of the roof and floor due to weathering.
Stage II: it consists in two different sub-stages, which could run in parallel without necessarily mutual connection. At stage IIa, gradual settlement of the southern elevation (which was founded on the filling material) could occur. The lack of connection to the S.E. corner facilitates settlement of the wall at this area. Stage IIb is associated to the out-of-plane collapse of the external leaf of the fortification wall, facilitated eventually by increase of pressure by the stone-earth filling and certainly by seismic events.
Stage III: at this stage, pronounced settlement renders the right (eastern) part of the southern elevation (between the entrance and the vertical joint) completely free-standing.
Stage IV: failure of the S.E. elevation, resulting in the collapse of the still standing rows of the second floor.
3. COMPUTATIONAL STRATEGIES FOR STRUCTURAL RE-DESIGN

3.1 The strategy in relation to the various phases of the study

The study of interventions on monuments is related, among other, to the institutional provisions for decision-making. The instances for decision-making are usually interdisciplinary. This provision ensures the presence of all disciplines involved in the understanding and protection of monuments (from their own scientific standpoint), but, on the other hand, sometimes renders communication and discussion difficult. It is therefore advisable to adopt a step-by-step procedure, which consists in the submission of a first study for approval, in which the pathology of the monument is analyzed, interpreted and measures are proposed with justification of their effectiveness. After approval with eventual amendments, a second study can then be undertaken, in which detailed analyses are carried out, as well as the dimensioning of the interventions, the design of materials etc. A possible exception to this approach could be allowed in the case of small monuments with well-studied typologies and no significant damages. In this case, the submission of a study in one step could be envisaged. The submission of a study in phases, common in other areas of engineering works, is very effective because it permits to reach consensus more easily, reducing therefore the risk of major modifications from the Engineer’s side. It was this strategy that was applied to the study of the N.E. tower.

3.2 Phase 1: numerical reproduction of the pathology & assessment of the interventions’ effectiveness

3.2.1 Considerations regarding the anastylosis concept

Prerequisites to a numerical study are the understanding of the building behaviour as well as a clear conception of the anastylosis project. The latter is very important for all monuments, but it endows a particular significance for ancient ones, because their ruinous state permits a variety of possible interventions, ranging from simple conservation of the remains to reconstruction of the supposed “initial”, “authentic” monument (or parts of it). Two are the key parameters which (should) frame the anastylosis options: the quantity of the available authentic material (building blocks) and the safety of the “repaired” structure. There is, of course, interaction between them: the more ancient material is available, the higher are the chances that the “repaired” monument fulfills basic engineering rules of safety, such as uniform distribution of masses and/or stiffnesses, closed plan (at least at the lowest rows) etc. and therefore less new material and/or other interventions are necessary to ensure structural safety. Indeed, an anastylosis project is the result of the, more or less successful and harmonious, balance of the aforementioned.

In the case of the N.E. tower, it was evident that the monument would not be conserved in the state to which it was reduced after the February 1981 earthquake (Figure 1 right) because the collapsed ancient blocks were
available and could be put back to their initial position. Consequently, the ruin could not provide the reference situation needed for the study and reproduction of the pathology, the assessment of the structure’s weaknesses and the documentation of the effectiveness of the interventions. This reference was developed after the anastylosis concept was defined; it was based, as already mentioned, on the simultaneous consideration of the available material and safety requirements. To start with, simple engineering judgement is opposed to the restoration of the situation before the earthquake (similar to 1933, Figure 1 left), because the monument would continue to be extremely vulnerable, unless drastic (and therefore incompatible to monumental values) measures were taken. If simple measures to reduce vulnerability, such as the reconstruction of the (lost) middle part of the southern elevation, were adopted, the safety of the structure would increase with limited new material. An additional requirement was also adopted: enhance safety by creating, as much as possible, structural continuity at the area of the joint between the southern and eastern elevations without alteration of the construction characteristics revealed by the survey (Figure 2). This could be achieved by various means, the simplest being the connection of the blocks on both sides of the joint with clamps. This anastylosis concept provided the reference situation. The corresponding model is shown in Figure 5 (left) and consists in a continuous masonry “envelope” with varying foundation conditions.

3.2.2 The rationale for the selection of the analysis approach

After having defined the reference situation, the analysis approach was discussed. Criterion for selection has been the expected response of the reference structure previously defined (Figure 5 left). The limited dimensions of the tower (cf. introduction) and the thickness of the building blocks (which varied from 60-65 cm at the base to 45-50 cm at the top) led to the hypothesis that the structure’s self-weight and friction forces would sufficiently resist seismic action, and therefore stone displacement, if any, would be localized at the top rows. This hypothesis was based not only on engineering considerations, but, and above all, on the understanding of the building and its behaviour with time. This understanding can only be acquired through detailed survey of the pathology, which has to be carried out by the Structural Engineer of the project. In the case of the N.E. tower, the survey consisted in the detailed measurement of the joint openings, the in- and out-of-plane displacements and other geometrical changes, such as rotation, soil and block settlement, as well as thorough inspection of the integrity of each stone block separately. Indeed, stone superficial cracks, far from being of secondary importance (because they do not threaten overall stability), are, on the contrary, precious indicators of stress concentrations, and therefore building deformation which cannot always be measured by other means. Thanks to this survey, it was proven that the joint openings and displacements of the lower parts of the structure (up to the 15th-16th row out of 20-21 standing) were very limited. Most of the important displacements were in-plane (due to the foundation settlement), whereas most out-of-plane ones were located, as expected, at the upper rows. Taking into account that the upper part of the tower was going to be dismantled in order to remove excessive displacements, survey data were reported on a 3D Autocad model (Figure 4) in order to preserve the configuration of the tower’s deformation and permit the detailed study of the building, not only within the framework of this study but in the future as well.

Figure 4. View of the tower’s deformation before dismantling (2015) from NW (left) & NE (right) up to the 18th row. Note the curved-shaped deformation of the still standing northern and western elevations
Figure 5. The model used for spectral analysis before (left) & after (right) the interventions (view from SW)

This 3D Autocad model provides the necessary evidence, regarding the hypothesis on the effect of the structure’s dimensions on the limitation, or even blockage, of stone displacement at least at the lowest two thirds of the tower’s height. Consequently, the adoption of elastic behavior, of a continuous (and not discrete) elements’ model and of spectral dynamic analysis were considered sufficiently accurate (Figure 5). Simulation was made with plane F.E. with the use of the code Sofistik. The stone block properties were defined as follows: compressive strength 55 MPa, tensile (Brazilian) strength 3 MPa, modulus of Elasticity 85 GPa (Spanos & Lamaris 2009). The choice of the modulus of Elasticity was based on bibliographical references (Oliveira et al 2006, Senthivel et al 2006, Vasconcelos & Lourenço 2009). Two values were selected, namely 8 GPa & 16 GPa, and sensitivity analysis of the structural response was carried out. It was found that this parameter did not significantly affect the elastic behaviour of the tower and the value of 8 GPa has been conservatively selected. The Hellenic Aseismic Code (EAK) classifies western Attica in zone ΙΙ, with design ground acceleration \( A = 0.24g \). The earthquake action was applied in two horizontal directions, whereas the vertical component was omitted. The superposition of the design spectrum for soil class B in one direction and 30% of it in the perpendicular one, resulted in 8 earthquake combinations. The importance of the monument was taken into account via the corresponding coefficient \( \gamma = 1.3 \). The coefficient of seismic behaviour \( q \) was chosen equal to 1.5 and foundation coefficient \( \theta \) equal to 1.0.

3.2.2 The reference model (phase 1): description, main results and need for further interventions

In the reference (“initial”) model (Figure 5 left) the existing boundary conditions were maintained: in the case of the northern and eastern elevations, which lay on rock substratum, restriction of displacement in the three directions was applied, justified by the fact that the rock surface has not only been flattened but in some places also carved, in order to hinder sliding of the blocks of the first row. In the case of the southern and western elevations, which lay on deformable soil, movement was horizontally restricted and springs were used for the vertical direction. Finally, the SW corner was also modelled as lying locally on rock substratum.

As far as the analysis is concerned, emphasis was put in the study of the displacements as well as the developing normal and shear forces on each elevation. The study of the displacements highlights the sensitivity of the structure. As an example, in Figure 6 (left) the northern elevation’s out-of-plane movement is shown. The main earthquake component is applied in an E-W direction and the secondary one in a N-S direction. The different colours of the displacement tensors correspond to out-of-plane displacements in opposite directions within the same (northern, in this case) elevation. This result is qualitatively compatible to the deformation configuration, recorded by the survey (Figure 4). In Figure 7 (left) is shown that the central area of the southern elevation moves opposite to the rest of the elevation. These results are typical for all load cases and elevations and reveal the effect of the uneven (both in height and deformability) foundations as well as the lack of mechanisms to ensure diaphragmatic function of the structure. Finally, the typical calculated horizontal normal and shear forces ranged between ±50 kN/m and only locally reached or overpassed 100 kN/m. The latter were mainly to be found at the areas where abrupt changes of the boundaries were modeled and have therefore been neglected.
The developing stresses were all below 1 MPa, and are therefore insignificant. Simple calculations have shown that a horizontal force equal to 50kN/m can be resisted by friction up to the 14th-15th (out of 21) rows, if a friction coefficient equal to 0.7 is used. This result is compatible to the recorded joint openings, which, as already mentioned, are concentrated mainly at the upper third of the structure. The analysis of the reference structure revealed, however, that the reconstruction of the southern elevation is not a sufficient measure to ensure a good earthquake behaviour and additional measures had to be envisaged with the aim to improve the displacement pattern, such as provision of a rigid foundation at the southern and western elevations in replacement of the deformable one and introduction of diaphragmatic function without the construction of floors or other invasive external structures. The first requirement could be met by the construction of an underfoundation with micropiles. This solution would not only ensure rigid foundation but is, in addition, ideal in preserving the ancient filling material. The second requirement could be met by the introduction of clamps and dowels at pertinent locations. All those interventions were subsequently introduced to the “repaired” model and a second analysis was carried out, in order to assess their effect.

3.2.3 The model after the interventions (phase 1): description, main results and conclusions

The model used for the simulation of the “repaired” situation (Figure 5 left) differed from the “initial” one, as follows: (a) the underfoundation of the southern and western elevations was simulated by restriction of displacement in the three directions; (b) in order to produce, as much as possible, a homogeneous response of
the elevations, linear elements -simulating steel clamps- have been introduced at three levels, namely row 10 (just below the entrance), row 15 (above the entrance) and row 17 (which, at that time was considered to be the topmost complete row. The main properties of these linear elements were: $E=200$ GPa, $f_{yt} = f_{yc} = 500$ MPa, $f_t = f_c = 550$ MPa. Finally, the contour of the upper rows was slightly changed and some stones were removed, to be in line with the available ancient blocks at that time. As it will be seen in subsequent paragraphs, this contour changed again at the second phase of the study, to accommodate more ancient blocks that were in-between identified. These changes, which affect the 1-2 upper rows, are not likely to influence in a significant way the response of the model. The effect of the two additional interventions is clearly illustrated through the study of the displacements of the structure’s elevations. As shown in Figures 6 (right) and 7 (right), the out-of-plane displacement becomes homogeneous. This means that the displacement tensors on each elevation now point at the same direction. These observations are valid in all load cases, which means that, thanks to those two interventions, a quasi-diaphragmatic function has indeed been generated. The effect of the interventions was less pronounced on the typical calculated horizontal normal and shear forces, which slightly decreased but still ranged roughly between ±50 kN/m with local pics equal or above 100kN/m. The latter, as previously reported, concentrated at local geometrical changes. The developing stresses remained below 1 MPa. However, as previously, possibility for joint opening and stone displacement at the top third of the structure persisted, as horizontal forces around 50kN/m cannot be in theory resisted by friction above this level.

The main conclusion of the first phase is, that the analysis of the “initial” structure highlighted some problems regarding the tower’s earthquake response, which could not have been predicted by an empirical restoration approach without numerical modeling. Furthermore, the study of the “repaired” structure proved the necessity of the proposed measures, and permitted their acceptance, not without some reluctance, by the Central Archaeological Council. However, the understanding of the building behaviour, and namely the certainty of joint opening at the topmost rows (which sooner or later would lead to collapse of individual blocks) raised the dilemma of hindering it or not, without simultaneously undermining the capacity of energy dissipation of the building. It has therefore been decided to proceed to further investigations, by reinforcing the corners of the structure with dowels and securing with clamps and dowels the topmost blocks, which belonged to rows which would remain incomplete due to lack of authentic blocks. What would be the effect of this reinforcement on the earthquake behaviour of the monument?

3.3 Phase 2: earthquake behavior of the tower & design of the interventions

3.3.1 Modeling strategy

The second phase focused on the understanding, in as much as possible quantitative terms, of the behaviour of the “repaired” structure, as defined by the analysis of the first phase, with emphasis on the effect, as well as the calculation, of the developing forces on the clamps and dowels to be inserted at various parts of the walls. Now, the modeling strategy needed to change and other tools were necessary to address this issue. The seismic response of the tower was analyzed with the discrete element method (code 3DEC) following the methodology developed at the Lab. of Earthquake Engineering, NTUAthens (Psycharis et al. 2003, Papantonopoulos et al 2002). The model is shown in Figure 8 (left) and corresponds to the model shown in Figure 5 (right), which represents the anastylosis project at the first phase (2014-2015). The elevations were modelled as vertical walls because the slight inwards inclination of their external surface (11 cm at the height of 10 m) was considered insignificant in structural terms. The stone blocks have been modeled as rigid blocks with density equal to 2700 kg/m$^3$. The elastic constants simulating the joints were considered equal to $K_n=5.0\times10^9$ Pa/m (normal direction), $K_s=1.0\times10^9$ Pa/m (shear direction), whereas no tensile capacity was assumed (Dasiou et al. 2008). The angle of friction was chosen 36.87° (corresponding to friction coefficient $\tan\phi=0.65$) and cohesion equal to zero. The clamps and dowels were modelled as non-linear springs with properties reported in Table 1 (Dasiou et al. 2008, Psychiatris et al 2009, Psychiatris & Toubakarti 2010).

As already mentioned, the monument is located at the eastern coast of the Corinthian Gulf, in an area surrounded by active seismic faults. On the basis of data on the area’s seismic history (Ambraseys 2009) and the records of the Inst. of Geodynamics, Univ. of Athens, earthquakes with intensity Ms= 6 are expected.
Moreover, in a radius of 100km earthquakes of smaller intensity, but still above \( Ms \approx 5\) are expected. Those data are compatible to the seismic records which are expected to occur in Attica (Psycharis 2007, Ambraseys & Psycharis 2012). Consequently, five seismic records have been selected (Table 2) with a variety of spectral contents. Time-histories were applied in the three directions.

Two sets of analyses have been carried out. The first consisted in the application of the time-histories as obtained by the corresponding records. In this case, the model (Figure 8 left) showed an excellent response, the calculated displacements as well as developing forces being very limited. The second set of analyses was applied on the improved model (Figure 8 right), to which additional blocks were added and the 18\(^{th}\) row completed to ensure a closed plan. Analysis has now been carried out with the application of the records multiplied by a coefficient, in order to raise the recorded maximum acceleration at the level of the design ground acceleration assigned by the Hellenic Aseismic Code for this area.

### 3.3.2 Main results

Despite the application of time-histories adapted to code provisions, and therefore corresponding to very strong solicitations, neither structural failure, nor collapse of isolated stone blocks were observed, with only one exception. As shown in Figure 8 (right), local failure of stones of the upper N.E. corner occurs when the Cascia earthquake is applied. Such failures, however, are considered acceptable, given the magnitude of the applied seismic actions, and no measures to counteract them have been considered necessary. Since practically no failure was observed, criterion for the assessment of the model’s response and the effectiveness of the interventions has been the permanent (remaining) displacements of the stone blocks. To this purpose, the displacement of characteristic stone blocks has been monitored and reported in diagrams.

| Table 1. Mechanical properties of the non-linear springs used for the simulation of clamps and dowels |
|---------------------------------|------------------|------------------|
| **Clamps** | **Dowels** |
| Active length \( r_{len} \) | 0.06 m | 0.01 m |
| Axial stiffness \( r_{ax} = E \cdot A / L \) \([N/m]\) | 0.75x10\(^8\), \( E= 105 \) GPa | 10** |
| Shear stiffness \( r_{s, stiff} = K_s = G \times A/L \) \([N/m]\) | 0.28x10\(^8\), \( G= 40 \) GPa | 0.6x10\(^9\), \( G= 40 \) GPa |
| Shear strength \( r_{u,shear} = (\sigma_y / \sqrt{3})\times A \) or \( \tau_y \times A \) \([N]\) | \( \approx 0.15\times10^5 \), \( \sigma_y = 300 \) MPa | \( \approx 0.5\times10^5 \), \( \tau_y = 180 \) MPa |
| Tensile strength \( r_{f,u} = f_u \times A \) \([N]\) | \( \approx 0.36\times10^5 \), \( f_u = 420 \) MPa | 10*** |
| Tensile yield strain \( r_{strain} \) | 0.20 | |
| Tensile and shear failure strain \( **** \) | 10\(^{20}\) | 10\(^{20}\) |

\( * \) describes the length of connectors’ local deformation on either sides of the joint \( ** \) in other words, the dowel does not work in tension \( *** \) practically zero tensile strength \( **** \) in order to avoid numerical failure of the clamps and dowels.
Table 2. Seismic records used (Ambraseys & Psycharis 2012)

<table>
<thead>
<tr>
<th>Record</th>
<th>Mw</th>
<th>R (km)</th>
<th>Longitudinal comp.</th>
<th>Transverse comp.</th>
<th>Coef.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Syntagma (1999)</td>
<td>5.97</td>
<td>10</td>
<td>1.07</td>
<td>0.89</td>
<td>0.52</td>
</tr>
<tr>
<td>Assisi-Stallone (1997)</td>
<td>6.04</td>
<td>14</td>
<td>1.42</td>
<td>0.34</td>
<td>0.36</td>
</tr>
<tr>
<td>Cascia (1979)</td>
<td>5.89</td>
<td>1</td>
<td>1.84</td>
<td>0.64</td>
<td>0.90</td>
</tr>
<tr>
<td>Kozani (1995)</td>
<td>6.61</td>
<td>14</td>
<td>2.13</td>
<td>1.16</td>
<td>1.10</td>
</tr>
<tr>
<td>Bisaccia (1980)</td>
<td>6.93</td>
<td>21</td>
<td>0.90</td>
<td>2.02</td>
<td>2.10</td>
</tr>
</tbody>
</table>

The selected blocks were located at rows 1, 10, 14, 17, 21 at the NE corner and rows 10, 14, 17, 21 at the SW corner as well as in the middle of row 19 of the southern elevation. Main results are shown in Figure 9. In those diagrams, axis X corresponds to the E-W direction and axis Z to N-S direction. In general, the maximum displacements during rocking were 2-3 cm (Assisi-Stallone, Cascia, Kozani) to 6 cm (Syntagma, Figure 9 left) with corresponding permanent displacements around or below 1 cm. The effect of the Bisaccia earthquake was, as expected, most pronounced due to the existence of long-period pulses, which, however, did not produce collapses. In this case, maximum and permanent displacements reached 10-15 cm at the most vulnerable row 21 of the tower’s N.E. corner (Figure 9 right). The combination of the recorded x- and z- displacements suggests the occurrence of rotations at the observed block. However, even this result was considered acceptable, as the records were multiplied by 2.67, which could appear exaggerated.

Figure 9. Displacement time histories in the middle of the top row of the southern elevation (Syntagma 1999); at the upper row of the NE corner (Bisaccia 1980)

Figure 10. Left: calculated tensile forces developing at the clamps located in the middle of row 19 at the southern-(time-hist. 28), eastern- (time-hist. 30) and western elevation (time-hist. 32) (Bisaccia 1980); right: calculated shear forces at the dowels of southern elevation: at the S.E. corner between rows 8 & 9 (time-hist. 33) and in the middle blocks between rows 18 & 19 (time-hist. 34) (Assisi-Stallone 1997)
The overall satisfactory behavior of the model is attributable, not only to the change of the foundation conditions and the rebuilt of the southern elevation, but to the presence of the connectors (clamps and dowels) as well. Crucial for the understanding of their behavior and their possible effect on the stones (to which they are embedded) is the calculation of the corresponding forces. To this purpose, clamps and dowels located at characteristic points were selected, monitored and the evolution of the developing normal and shear forces during the earthquake reported on diagrams. The points of observation were located mainly at the upper rows of the tower, where, as expected, the developing forces are the highest. The tensile forces at the clamps were monitored in the middle of rows 17 and 19, whereas the shear forces at the dowels at the corners of rows 19-21 and in the middle of row 19. The connectors’s strength was chosen to be lower than the stone tensile and shear strengths (Zambas 1994). Concerning the clamps, a reduction of the aforementioned tensile strength was admitted, resulting finally in clamp design tensile strength equal to 36kN and dowel design shear strength 40 kN. The maximum calculated tensile forces were recorded at the Assisi-Stallone and Bisaccia earthquakes, reaching 25kN. Only in the case of Cascia earthquake, where, as previously mentioned, local collapse has occurred, did the tensile forces reach 30-35kN at the areas around failure. Clamp tensile forces were also monitored at lower rows (9, 14) but, as expected, the calculated values in these places were lower than 15kN. As an example, the calculated tensile forces at three elevations of row 19 under the Bisaccia earthquake are shown (Figure 10 left). It is recalled, that this earthquake is characterized by long period pulses; in this case, the tower is most vulnerable. Nevertheless, the tensile forces remain below 25kN, suggesting that neither failure of the connectors, nor of the stones is likely to occur. The dowel shear forces remained extremely low, namely below 5kN (Figure 10 right). This result is compatible to the fact that the dowels, more than the clamps, are likely to “react” only after the friction forces are overpassed. Thus, they could have eventually been omitted. The main reason, why they were kept, is that they are expected in the long run to act as “stoppers” of the gradual opening of the joints due to the earthquakes to come in the future.

5. GENERAL CONCLUSIONS

The N.E. tower of the ancient fortress at Aegosthena, Attica has been very severely damaged at the 1981 earthquake of the eastern Corinthian Gulf. The computational strategy adopted for the design of the tower’s anastylosis, consisted in two phases. The first phase aimed at qualitatively understanding the weaknesses of the structure, the effect of the different foundation conditions and the absence of diaphragm (at the level of the ancient floor and roof, which were not going to be rebuilt) on its seismic response. Spectral dynamic analysis was applied because the assumption of elastic behaviour was considered sufficient and compatible to the pathology. On the basis of the results, the repair and strengthening measures were proposed, modeled and the structure was again analyzed. After approval of the interventions by the Central Archaeological Council, a second phase took place, in which emphasis was put in understanding, in as much as possible quantitative terms, the behaviour of the repaired structure, including the developing forces on the clamps and dowels that were going to be inserted at various parts of the walls. For this purpose, the seismic response of the tower was studied with a time-history discrete elements’ analysis. The sequence of the two analysis methods was found effective in addressing the issue of the anastylosis with the corresponding (for each phase of the study) accuracy. A comparison of the two approaches results in the following conclusions:

1. The first method is more widely known, and the tools for it are easily available. On the other hand, the continuous model used for the analysis does not correspond to the tower’s construction system. However, thanks to the monument’s limited dimensions - which were expected to lead to a quasi-elastic response of the system (at least at the two thirds of the height) - the deformation pattern of the tower could be satisfactorily reproduced by the “initial” model and the expected opening of joints at the upper rows could be predicted by simple comparison of the calculated forces to the locally available frictional resistance. Moreover, the effectiveness of the proposed interventions could be clearly demonstrated.

2. The second method is ideal for the modeling of the dry-stack construction system. It is, however, more demanding, in terms of computational tools as well as selection of the pertinent time-histories to be used. Moreover, taking into account that the possible interventions on the ancient blocks are quite unilaterally defined (as they consist on the introduction of connectors at pertinent locations), their dimensioning can be easily carried out following simple, limit-state principles.
It can therefore be concluded that the elastic dynamic analysis is a pertinent method for the study of the behaviour of ancient towers, provided their dimensions are limited and the wall is sufficiently thick to reduce slenderness and all secondary effects associated to the latter.

6. ACKNOWLEDGMENTS

Special thanks and gratitude are addressed to Prof. I. Psycharis (School of Civil Eng., NTUA) for his permanent advice, including the selection of the time-histories used, and for permission to use the code 3DEC, as well as to Mr S. Bitzarakis, Struct. Eng., M.Sc. (at Sofistik Ltd) for his advice and technical support regarding spectral analysis. The German Archaeological Institute of Athens is also gratefully acknowledged for permission granted for the publication of the photo of the tower taken in 1933 by W. Wrede.

7. REFERENCES


Itasca Consulting Group, Inc. 3DEC: 3-Dimensional Distinct Element Code, Theory and Background.


