

Strengthening Proposals for the Trigonio Tower at the City Walls of Thessaloniki

DELIZISI Vasiliki^{1, a}, AXIOTIDOU Matina^{2, b}, DOUDOUMIS Ioannis^{3, c} and
IGNATAKIS Christos^{3, d}

¹Papathanasiou M. 16, Drymos Thessaloniki 57200, Greece

²Nemesseos 17, Athens 11152, Greece

³Dept. of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, Greece

^avdelizisi@gmail.com, ^bm.axiotidou@yahoo.com, ^cdoud@civil.auth.gr, ^dignatak@civil.auth.gr

Abstract The Trigonio tower, constructed in the 15th century for fortification reasons, is part of the city walls of the old town of Thessaloniki. The tower is a cylindrical masonry building and has been considered as part of the cultural heritage of humanity by the UNESCO World Heritage Committee. Today it presents some major damage to its load bearing system, including intense cracking, moisture damage and disintegration of mortar, bricks and stones. An extensive research study for the evaluation of the tower's bearing capacity, as well as for the suggestion of efficient strengthening interventions was held, in order to effectively prolong its expected lifetime. To this purpose, refined analytical models of 3D finite elements have been formed and used for the estimation of the elastic and inelastic tower's response to gravity and seismic loads. Results' evaluation of several static and dynamic analyses showed that the tower's stability is not at risk under the action of gravity loads in general, but it's bearing capacity under strong seismic excitations is questionable, thus becoming obvious the necessity of strengthening interventions. An invertible intervention of strengthening the tower with proper sets of external pre-stressed steel rings was also examined. Linear (elastic) analyses showed that they cannot sufficiently evaluate the effectiveness of this strengthening process. Inelastic analyses, using material laws of Drucker-Prager's type, showed that the use of prestressed steel rings can significantly reduce the plastic strains' appearance and therefore the cracking in the masonry walls of the monument.

Keywords: Trigonio Tower, repair, strengthening, 3D finite element, elastic analysis, inelastic analysis, prestressed steel ring, peripheral stress, plastic strain

Introduction

The present paper is based on two studies (Axiotidou et al. 2007, Delizisi 2008) that were performed as part of a Master in Science program of Aristotle University of Thessaloniki titled: "Protection, Conservation and Restoration of Architectural monuments".

History of the Tower The Trigonio Tower is located in the North-eastern corner of the city walls of Thessaloniki (Fig. 1). It is a tower used by the artillery during the ottoman period. During the 20th century it was used by the Greek army. It was characterized as a monument of Greek Cultural Heritage in 1962. It is also considered as part of the cultural heritage of humanity by the UNESCO World Heritage Committee.

Its current name came from the chronicle of Ioannis Anagnostou concerning the fall of Thessaloniki in 1430. As it is mentioned the Turks invaded the city from Trigonion, more precisely from an unguarded tower (Tsaras 1958). However it is not clear whether Trigonion was the name of the area or the name of the tower that existed there.

The time of the construction of the Tower isn't accurately known. However, the morphology, some common elements with other ottoman fortification constructions and the historical data appoint the second half of the 15th century as the most probable construction period. There are signs of

modifications in the interior of the monument, which aimed at making the tower compatible with the new cannons that date at the beginning of the 16th century (Stefanou 1988).

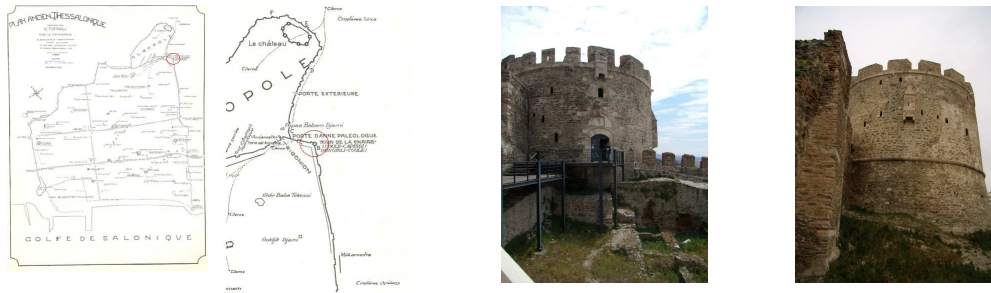


Figure 1: The location of the city tower in the city walls (Tafrali 1913). Views of the Trigonio Tower

Description of the Tower The tower is formed as a cylinder. Its diameter ranges from 24.85m at the base to 23.45m at the highest level of the tower. The pitch of the surface is greater till the entrance level and it mildens at higher levels. Due to the ground pitch, the monument's height varies from 20.00-22.00m at the south-eastern side to 11.15m at the north-western side. It is a construction of great volume, serving a defensive role. It must be mentioned that at the core of the ottoman tubular structure there is a smaller rectangular tower of the early Byzantine era, while the space between them was filled up with soil material.

The entrance to the monument is located in the western side. In the interior eight chambers can be found (Fig. 2): the entrance (X1), three cannon chambers (X2, X3, X4), three store houses (X5, X6, X8) and one room with special features of fortification architecture (X7). All the rooms are organized within the monument in a circular manner. These rooms were created in the outer periphery of the structure, in different levels. The room X8 is an exception as it was constructed near the core of the tower.

Every room can be accessed through a system of corridors and stairs. The monument lacks an indiscrete circulation. The chambers are being covered with semi-cylindrical masonry barrel vaults. The main hall of the entrance is the exception as it is covered by a semi-spherical dome. The floor of every chamber is covered with flat stones.

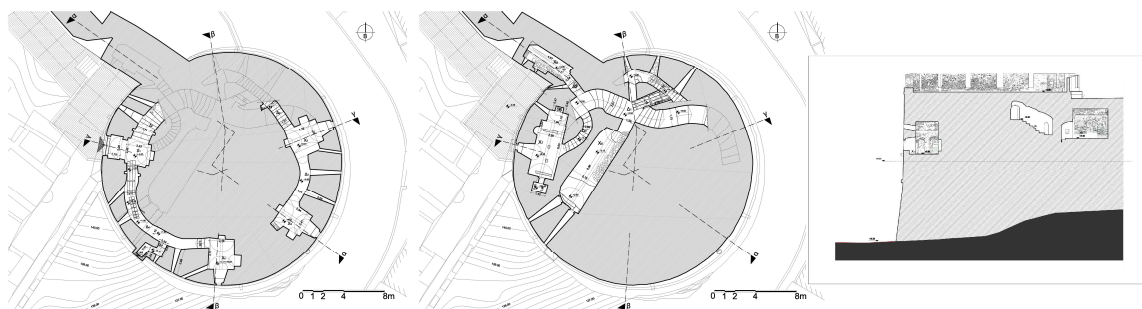


Figure 2: Horizontal and vertical sections of the tower (Axiotidou et al. 2007)

The fact that makes unique the monument compared to other similar constructions is the lack of a central chamber.

The tower is made of masonry. The basic materials are rubble or flat stones of serpentine, bricks and conjunctive mortar. An erratic placement of stones and bricks can be observed. Seldom are met ashlars of porous stone in the outline of the openings. There can also be found scattered marble architectural members in second use.

The conjunctive mortars of the initial construction are lime mortars of two types:

- White, coarse-grained
- Red, medium-grained, with splinters of ceramic material (Axiotidou et al. 2007).

Pathology

The tower presents intense damage and deteriorations of various types, the most important of which are the cracks and the attack of humidity. There is also significant disintegration of mortars, bricks and certain stones. In certain places also there is loss of the masonry material.

In the three cannon chambers there are large throughout cracks along the key line of the ceiling vaults extending as well lower to the surrounding walls. Large cracks can also be found in the two main corridors of tower. In the chambers X6 and X8, there are cracks of detachment that extend vertically to all the chambers' height. Those cracks were formed where the walls of the old-Byzantine Tower are in contact to the newer construction of the cylindrical Ottoman Tower.

Humidity can be traced externally in vertical areas under the gutters, as well as at the lower zone near the ground level (amounting humidity). The humidity is more intense in the northern part of the tower, because of the limited sun effect. In the areas where the humidity is more intense plant growth has appeared. On the ceiling of the chambers at the upper levels, the attack of humidity is intense and can be attributed to the failure of the waterproof material used in the terrace. Regarding chamber X8, humidity can be attributed to the two apertures of water collection that lead to an inner cistern (Axiotidou et al. 2007).

Elastic Analysis

As part of a restoration project concerning the Trigonio Tower a preliminary evaluation of the tower's bearing capacity was performed by using a refined analytical structural model with 3D solid finite elements and linearly elastic material laws (Fig. 3). The various static and dynamic response-spectrum analyses were performed using the SAP2000 software (Axiotidou et al. 2007).

Creation of the Model During the formation of the model a number of problems had to be solved, concerning the lack of symmetry, the complexity of the various chambers and corridors in the interior of the tower and the lack of knowledge concerning the thickness of the cylindrical body, the foundation of the monument and the material properties. The basic concession made was considering the core of the cylinder as an empty space and applying the earth pressure of the non saturated soil that fills the core on the internal surface of the cylinder. It must be noted that the soil infill is considered non-saturated, based on the fact that the first repair measure must be the waterproofing of the terrace.

Static and Dynamic Analysis (Existing Structure) The loads used for the elastic linear analyses were the dead load (g), the earth pressures of the non saturated material (p) and the seismic action in all three axes (E_x , E_y , $0.70E_z$) according to the Greek Seismic Code, properly modified for a monumental structure. The live loads were considered negligible. During the modal analysis, Ritz vectors were used.

The evaluation of the results led to the conclusion that, for the vertical compression stress, the worse load combination is the one of the static loads ($1.35g+1.50p$). The masonry, however, presents no special pathology problems due to its satisfactory compressive strength ($f_{ck}=1.60\text{MPa}$). Regarding the peripheral tensile stress, the worse combination was the seismic one ($g+p\pm E$). The tensile stresses in many parts of the structure surpass 1.0MPa and are much higher than the assumed tensile capacity of the masonry ($f_{tk}=0.16\text{MPa}$). In certain corridors and openings the values exceed 2.0MPa .

Static and Dynamic Analysis (Strengthened Structure) The results of the dynamic analyses led to the proposal of strengthening the bearing capacity of the tower using 4 sets of external prestressed steel rings (each set consists of three $\varnothing 32$ bars) and to further analyses in order to check the effectiveness of the steel rings.

The conclusion of the analyses of the strengthened model was that -despite the high prestress of the rings- there is only a slight drop of the value of the peripheral tensile stress. This must be attributed to the great thickness of the masonry (about 5m). More precisely, prestress under normal temperature conditions causes a drop of about 0.15MPa in the value of tensile stress, while for higher temperature

(that causes the rings to become loose) the drop in the value of the stress is around 0.12MPa. Those values are met in the exterior cheek of the cylinder, while the drop is greater in the interior (about 0.23MPa) (Fig. 4).

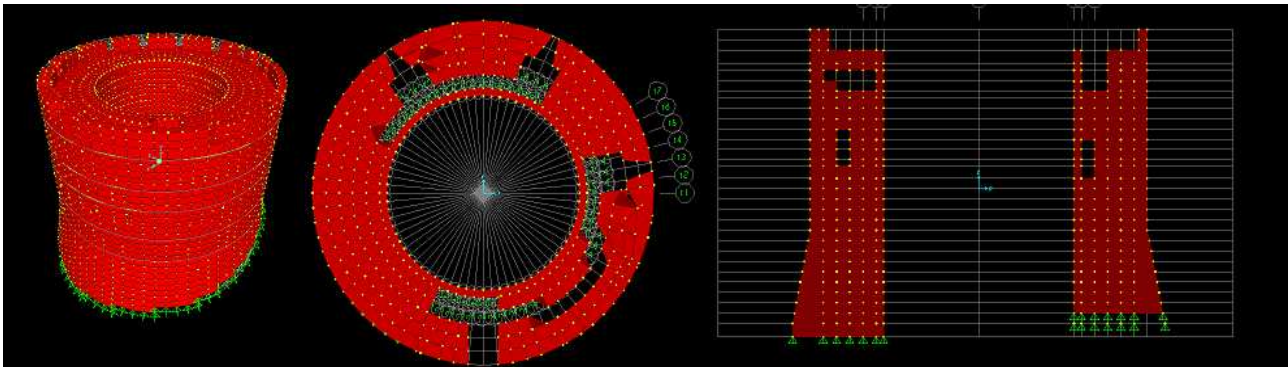


Figure 3: The model in the environment of SAP2000

Since the extreme values of the peripheral stresses strongly exceed the tensile strength of the masonry, it is obvious that inelastic analyses must be further performed for making possible the estimation of Tower's bearing capacity and checking the efficiency of the proposed strengthening intervention with steel rings.

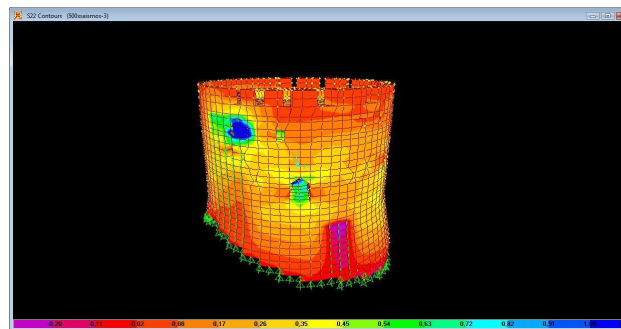


Figure 4: The peripheral stress diagrams (s_{22}) of the strengthened model in the SAP2000 software

Inelastic Analysis

The elastic analysis offers an approximate view of the structure's performance as it is unable to represent cracking. It was therefore decided a more rigorous approach using non-linear analysis with inelastic material laws of Drucker-Prager's type. This effort was realised as a part of a diploma thesis project. To this purpose a more refined analytical model was used.

Creation of the Model The model was created according to the ADINA software requests (Adina R&D Inc. 2006, Doudoumis et al 2006), using 3D solid finite elements for the masonry material, as well as for the soil material that fills the core of the Tower (Fig. 5). The constitutive laws of both these materials consist of a conventional Drucker-Prager's yield surface together with a Tension Cut-off Limit, the value of which was chosen according with the tensile strength of each material. In this model the rest of the modelling assumptions that were taken into consideration have already been mentioned in the *elastic analysis*.

Static and Dynamic Analysis (Existing Structure) For the static analysis of the Tower's model, only the dead load of the masonry and the soil material was taken into consideration. The live loads are considered negligible and were not used in the solution process, due to the huge volume of the monument and the soil material of its core. The loading was applied in 100 successive steps.

Checking the diagram of plastic zones as well as the plastic strain diagram, we are led to the conclusion that the developed stress exceeds the strength of the materials at the contact surface of

masonry and soil material as well as at some openings. The phenomenon is mainly connected to the excess of the tensile strength. The maximum value of the developed peripheral stress is 162.6 kN/m^2 . The general conclusion is however that the stability of the tower is not at risk under the action of static loads due to its own weight.

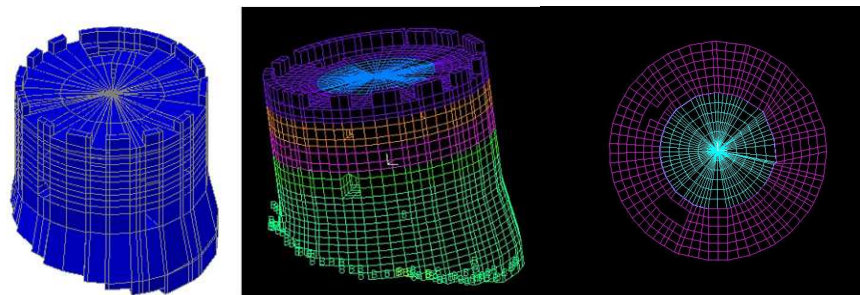


Figure 5: The analytical model of the Trigonion Tower as it was created using the programs AutoCAD and ADINA

During the solution process of the dynamic analyses, the dead weight of the masonry and the soil material was used, as well as the seismic action. Concerning the seismic action, the accelerograms recorded during the earthquake of Thessaloniki in 20 June 1978 ($\text{PGA}=0,146\text{g}$) were used. As mentioned earlier the live loads were considered negligible.

Checking the diagrams of plastic zones leads to the conclusion that -during the dynamic loading- the plastic zones fluctuate depending on the values of the ground acceleration. The phenomenon is mainly connected to the excess of the tensile strength and is met at the contact surface of masonry and soil material as well as at some openings (Fig. 6). However, there are areas where the values of compression strength exceed the compression strength of the material. An interesting observation is that the values of plastic strains do not fluctuate from the time peak of the earthquake till the last time step examined.

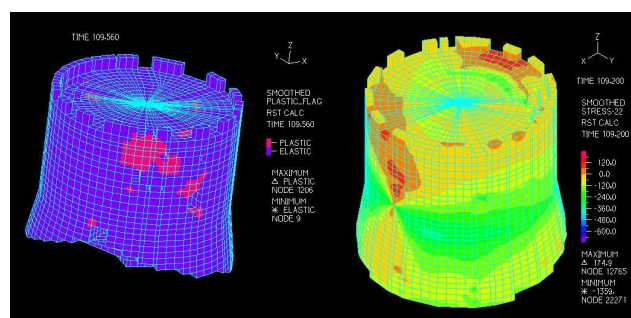


Figure 6: The plastic diagram and the diagram of peripheral stress of the existing body.

Static and Dynamic Analysis (Strengthened Structure) As already mentioned, the results of the elastic analysis concerning the strengthened Tower (using four sets of prestressed rings) were not very encouraging, since it seemed that the use of steel rings was not sufficient. It was therefore decided to further study the effectiveness of those rings using a non-linear analysis. To this purpose the four sets of prestressed steel rings were introduced into the previously mentioned analytical model created in the environment of the ADINA software using elasto-plastic hardening material law.

At the various analyses of the strengthened model, the prestress of the rings was applied as an equivalent temperature load, after the dead load was fully applied and before the application of seismic loads.

The results of the dynamic analyses showed that the lower sets of prestressed rings develop higher stress values, with the maximum value reaching 591.2 kN/m^2 .

The examination of the stress diagrams of the non-linear analyses led to similar conclusion with the elastic analysis: there seem to be only a slight drop of the stress values (Fig. 7). However, further

comparison between the plastic strain diagram of the original model and the strengthened one leads to the conclusion that the use of prestressed rings significantly reduces the amount of plastic strains in the masonry. More precisely, there is a drop of 25% in the maximum value of plastic strain (0.048% compared to 0.065%), while the reduction of the areas presenting plastic strains is about 50%. That is being translated to significantly less cracking in the masonry of the Trigonio Tower in case of an earthquake similar to the one of Thessaloniki in 1978. It is also noted that the presence of the rings causes only a very slight drop in the plastic strains of the soil material.

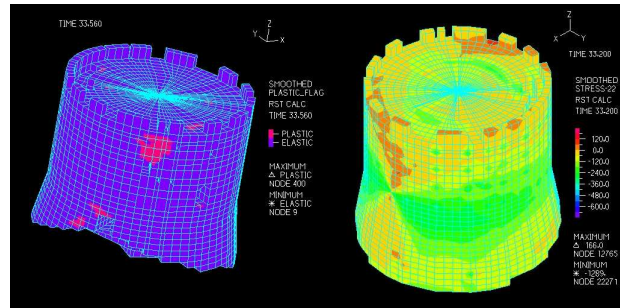


Figure 7: The plastic diagram and the diagram of peripheral stress of the strengthened body.

Conclusions

The stability of the tower is not at risk under the action of static loads due to its own weight. For seismic motions similar to Thessaloniki's 1978 earthquake, the Tower could present masonry cracking that looks alike the existing one. It could also present some local collapse in several sensitive areas, like the openings within the walls. The proposed invertible strengthening intervention, with 4 sets of external prestressed steel rings, can significantly increase the towers bearing capacity against seismic actions.

References

- [1] Anagnostou, I "Narration of the last fall of Thessaloniki. Poem on the fall of Thessaloniki." Tsaras, G (edit.) (1958). Library of Byzantine Thessaloniki, Thessalonikis 1, Thessaloniki.
- [2] Axiotidou, M, Gouliopoulou, V, Delizisi, V, Lialios, K, Mpitzikopoulos, D, Nasi, M, Papakonstantinou, E, Syngelaki, E, and Chalkidi, A (2007). "Trigonion Tower. Analysis of the existent situation. Proposal of restoration and re-usal." Workshop of interdepartmental co-operation, Aristotle Univ. of Thessaloniki, Thessaloniki.
- [3] Delizisi, V (2008). "Estimation of the bearing capacity of the Trigonio Tower using an inelastic behaviour finite element model." Aristotle Univ. of Thessaloniki, Thessaloniki.
- [4] Doudoumis, I N, and Kallioudakis, E D (2006). "Seismic response of Rotunda monument due to the Thessaloniki 1978 earthquake," in *Proc. 13th European Conference on Earthquake Engineering*, paper 418, CD-version.
- [5] Stefanou, S (1988). "Trigonion Tower. Contribution to the study of the Turkish fortifications of Thessaloniki." *Byzantina*, 14, 413-453.
- [6] Tafrafi, O (1913). "Topographie de Thessalonique." Librairie Paul Geuthner, Paris.
- [7] Theory and Modeling Guide, Adina R & D Inc, 2006.