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Strengthening of a 400 Year Old Ottoman Minaret

Renforcement d'un minaret ottoman de 400 ans

Verstärkung eines 400 Jahre alten ottomanischen Minaretts

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SUMMARY

The Minaret of Rotunda in Thessaloniki was built in 1590. The structure has a total height of 36m and is made of stone and brick masonry. The monument has been damaged by several strong earthquakes. During the earthquake of June 1978 the damage increased dangerously. This paper presents the procedure followed for the assessment of the bearing capacity of the structure, including in situ measurements, laboratory tests and geotechnical-seismological studies of the area. Finally, alternative strengthening solutions are analyzed and evaluated.

RÉSUMÉ

Le Minaret de la Rotonde à Thessalonique a été construit en 1590. La structure d'une hauteur de 36m est construite en maçonnerie de pierres et de briques. Le monument a subi des dégâts à cause de séismes puissants. Pendant le séisme de juin 1978, les dégâts ont augmentés dangereusement. Cet article présente la méthode d'estimation de la capacité portante de la structure et contient des informations sur la structure et l'état du monument, des mesures sur place, des essais en laboratoire et des essais géotechniques et sismiques sur place. Enfin, des solutions alternatives de renforcement de la structure sont analysées et évaluées.

ZUSAMMENFASSUNG

Das Minarett von Rotunda in Thessaloniki wurde 1590 gebaut. Es ist 36m hoch und besteht aus Stein- und Ziegelmauerwerk. Mehrere Erdbeben, besonders aber das vom Juni 1978 haben dem Monument bedrohlichen Schaden zugefügt. Der Bericht schildert die Methode, nach welcher die Tragfähigkeit des Minaretts durch Messungen am Objekt, Laborversuche und geotechnisch-seismologische Untersuchungen bestimmt wurde. Alternative Sanierungsmassnahmen werden durchdacht und bewertet.

1. INTRODUCTION - HISTORICAL BACKGROUND

The Minaret of Rotunda of Thessaloniki was built in 1590 in the courtyard of the Roman Rotunda (300 A.D.), when the building was converted from Christian church to Muslim mosque during the long period of Ottoman occupation of the city. The monument, erected in an area of relatively high seismicity, has been damaged due to several strong earthquake excitations as well as to weathering effects. During the earthquake of June 20, 1978 the damages were intensified. The 9th Ephorate of Byzantine Antiquities, officially in charge of the monument, carried out the necessary emergency works and entrusted the investigation of the structural condition and the design of repair and strengthening interventions of the monument to the Lab. of R/C Structures, Univ.of Thessaloniki, Greece. In the following chapters the phases of the research project are presented in short.

2. IN SITU AND LABORATORY RESEARCH AND MEASUREMENTS

The 9th Ephorate provided to the research team a complete series of architectural drawings, based on detailed surveying of the monument, the results of 64 hammer tests of bricks and mortar and a preliminary report [1]. In order to enrich the available data, a research project including constructional pathology surveys, in situ and laboratory measurements and tests, in situ measurements of the fundamental period of the structure and geotechnical and seismological studies of the territory was carried out.

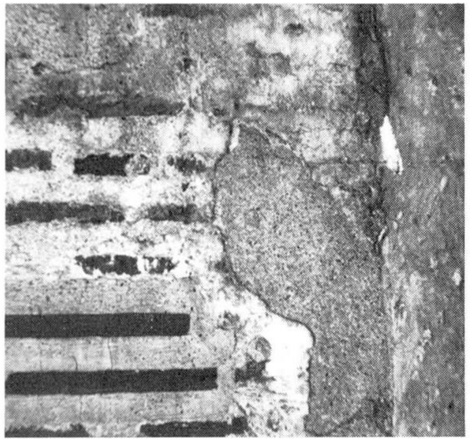
2.1 Geometry of the Structure

The Minaret is located at a distance of 3.50m western of Rotunda (see Fig.1a). It consists of a 4.50x4.50m square cross-sectioned base, 7.50m in height and is founded on a thick stiff red clay layer approximately 6.00m below the ground level. The base is supporting the trunk of the Minaret consisting of two successive twenty-sided trunkated conical parts of 22.00m total height crowned by the muezzin's balcony and the highest fifteen-sided cylindrical part 6.00m in height (see Fig.2). The external diameter of the trunk decreases gradually from 3.25m to 2.10m and finally to 1.75m at the highest cylindrical part. The narrow entrance (0.85x1.95m) is located at the northern face of the base. The internal helical staircase with gradually decreasing diameter (D=1.80-1.45m) leads up to the balcony at a height of 29.50m. The small muezzin's doorway (0.50x1.50m) is located at the southern face of the cylindrical trunk penetrated till the top by the narrow staircase (D=1.35m). Five narrow loopholes are located along the height of the southern side of the trunk.

The base of the Minaret is consisted of successive stone and brick masonry layers while the masonry of the trunk is pure brickwork built by flattened bricks (30x40x4cm) with thick mortar joints (4cm),(see Fig.1b). The balcony consists also of brickwork supported by successive horizontal wooden beams tangentially projecting from the cylindrical trunk. The staircase is masoned together with the surrounding trunk. At the edge of every step a wooden beam is located diametrically and is embedded in the masonry of the trunk, while along the axis of the staircase a masonry column (D=0.25m) is constructed (see Fig.1c).

2.2 Pathology - Emergency interventions

As it can be easily seen in Fig.2, the base of the Minaret is unplastered. The plaster covering has been collapsed at the major part of the trunk, while at the rest of the surface, as well as at the upper cylindrical trunk, it is heavily cracked. Limited superficial deterioration of masonry, especially at the northern face of the trunk has been detected and confirmed by the hammer tests. The wooden beams at the highest steps of the staircase have been heavily corroded due to rain water passing through the open top, as the wooden acute conical roof of the Minaret fall down, probably during an earthquake, at the begining of the



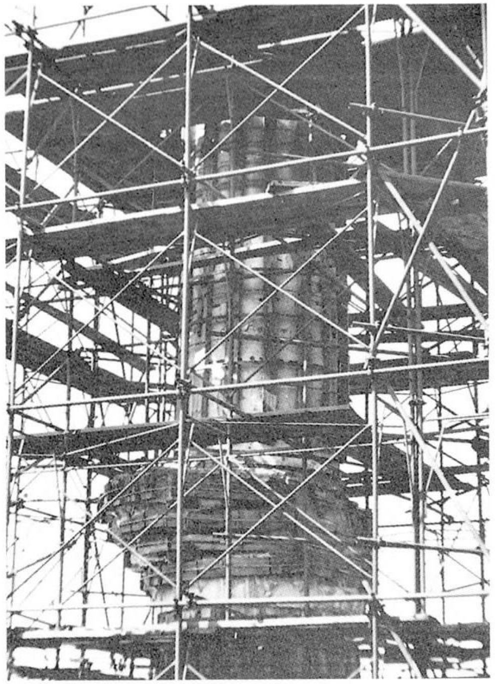


Fig.1

(a)Rotunda-Minaret: Southern view.(b)Masonry close-up: Merely visible horizontal crack.(c)Internal staircase. (d)Balcony and upper cylindrical part

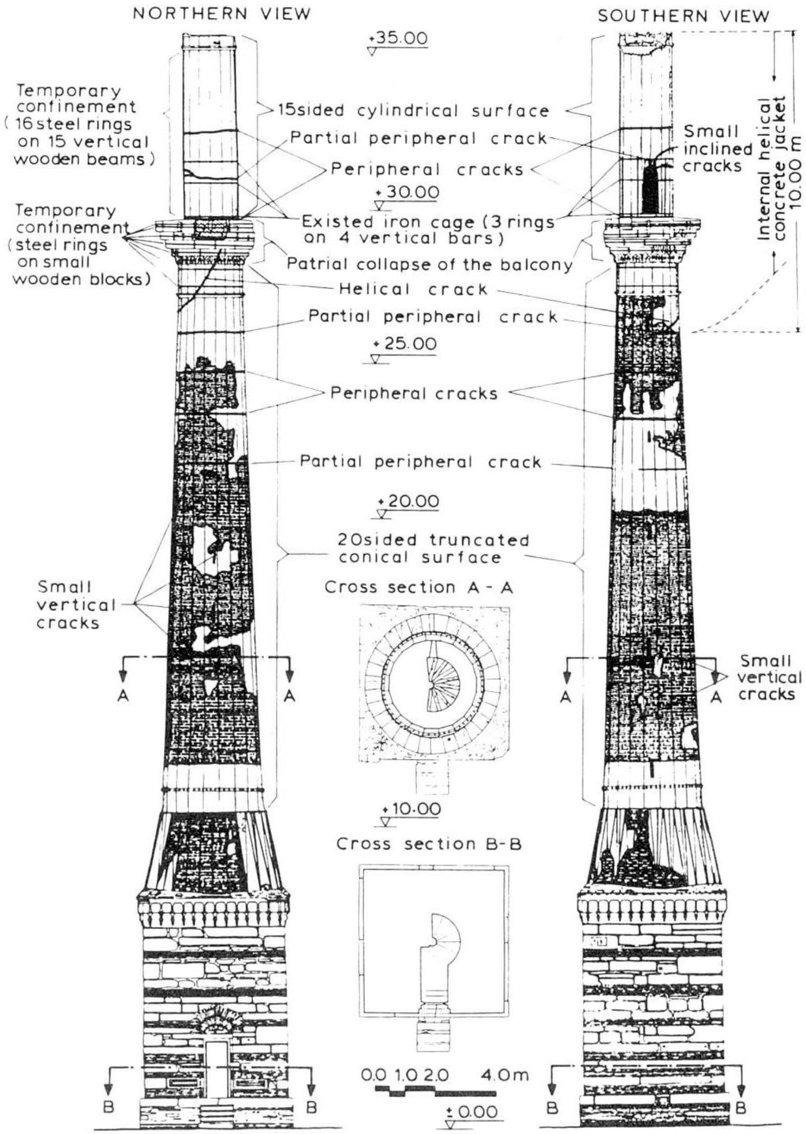


Fig.2 Northern and Southern elevations of the Minaret. Cracking pattern, external emergency interventions

present century. Successive horizontal peripheral cracks have been ascertained at the upper two fifths of the Minaret and have been drawn in Fig.2. There is one helical crack starting from the highest loophole, following the ascenting steps and cutting the balcony. All the cracks, being under vertical compression, are just visible (see Fig.1b). During the earthquake of June 20,1978 (epicenter 30km from Thessaloniki, magnitude 6.5 grades in Richter scale) a part of the balcony collapsed and the upper half of the cylindrical trunk was slightly translocated (see Fig.1d,2). It must be pointed out that, despite of the damages, no permanent horizontal drift of the structure has been ascertained and no foundation problem has been detected.

Emergency works were undertaken just after the 1978 earthquake including general external scaffolding (see Fig.1a) and temporary external confinement by steel rings and wooden beams as well as internal helical concrete jacket at the upper 10m of the Minaret (see Fig.1d,2).

2.3 Mechanical characteristics of bricks, mortar amd masonry

Non destructive tests in combination with a limited number of laboratory destructive tests were applied to make an estimation of mechanical characteristics of bricks, mortar and masonry. To this direction a new series of hammer and ultrasonic tests on bricks and mortar, beyond those performed by the 9th Ephorate, was carried out. Three masonry "triplets" were extracted from the base and the trunk of the Minaret (see Fig. 1b) and were used for the laboratory tests.The results are shown in Table 1.

Material	Compressive strength	Flex.tensile strength	Dynamic Elast.Modulus
Bricks	f_{bc} = 17.56MPa	f_{bt} = 3.52MPa(1)	E_{bd} = 12300MPa(3)
Mortar	f_{mc} = 1.28MPa	f_{mt} = 0.36MPa(1)	E_{md} = 1150MPa(3)
Masonry(5)	f_{wc} = 2.61MPa	f_{wt} = 0.11MPa(2)	E_{wd} = 2600MPa(4)

(1) Wide scattering, (2)Direct unstucking strength of bed joints, (3)Estimated from the ultrasonic tests, (4)Calculated by using a specific formula, (5)Pure shear strength of joints : f_{wso}= 0.19MPa

Table 1 Mechanical characteristics of bricks, mortar and masonry

Significant difference in the composition of mortars between the highest cylindrical part and the rest of the Minaret below the balcony has been found out, although the compressive strengths of the mortars were almost equal. Consequently, either the Minaret had been erected in two phases, or the part over the balcony had probably been collapsed and rebuilt. In Fig.3 the idealized masonry stress-strain curve adopted for the analyses and the stress-strain curves of concrete and steel for the strengthening interventions are shown.

2.4 Fundamental Period of the Minaret

To determine the in situ fundamental period of the Minaret, the ambient vibration method was used, based on the local traffic effect, with the aid of a sensitive portable system. Two measurements have been carried out, the first one before the emergency interventions (1979) and the second twelve years later (1991). The fundamental periods were found equal to 0.80 and 0.97 sec respectively. Under the reasonable assumption that the metal scaffolds, loosened by the time, have no influence on the response of the structure, the increase of the fundamental period must be attributed to the added mass of the internal concrete jacket on the top of the Minaret.

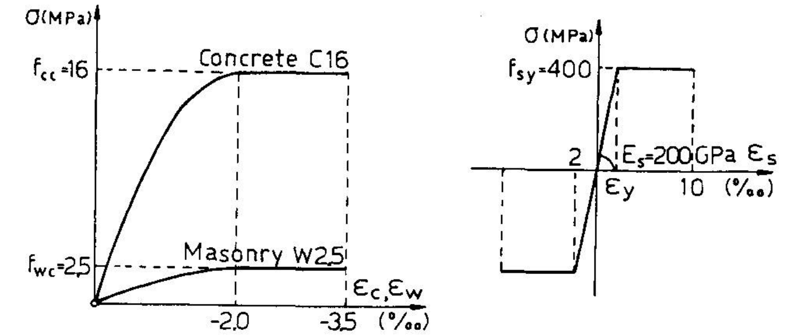


Fig.3 Idealized stress-strain curves for masonry and intervention materials (Concrete, Steel)

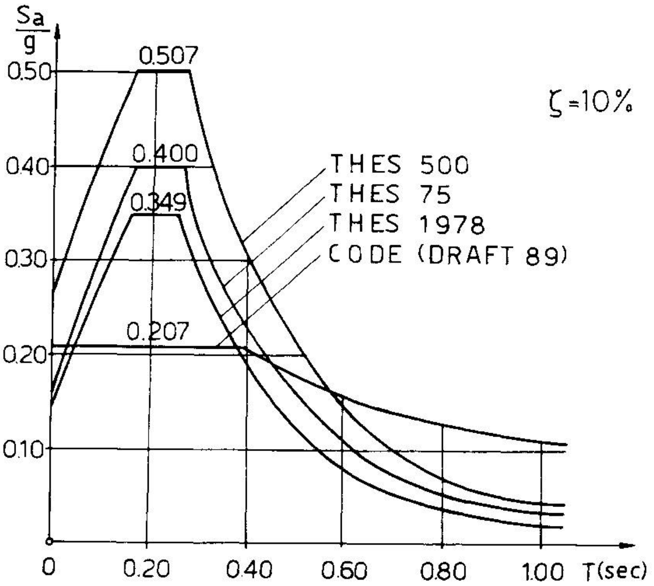


Fig.4 Design seismic accelelation spectrums

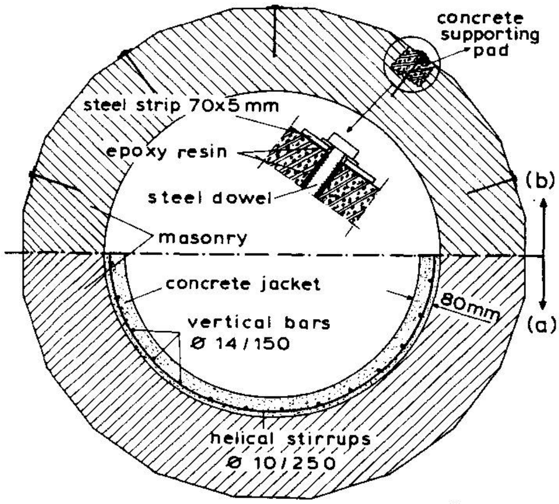


Fig.6 Semisections of Minaret trunk: (a)Internal concrete jacket, (b)External steel strips

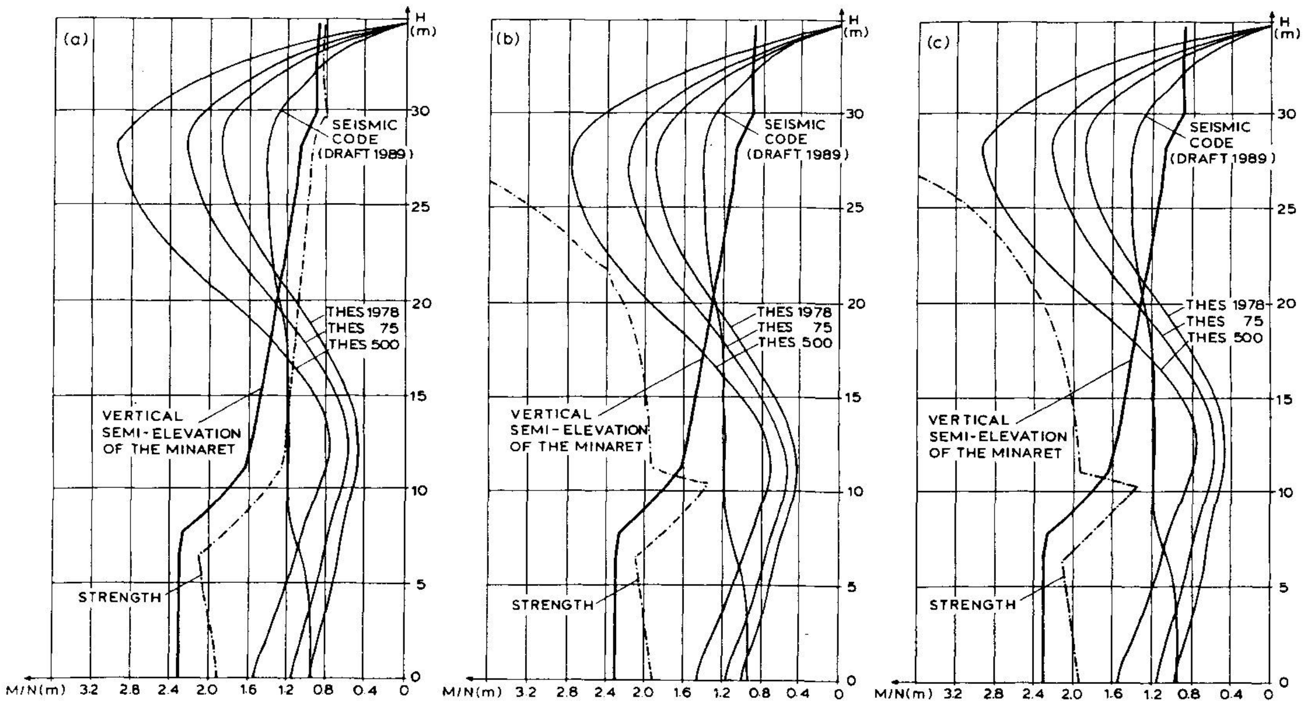


Fig.5 Eccentricity curves under seismic spectrums and strength eccentricity curve:(a)Virginal state,(b)Concrete jacket,(c)External steel strips

2.5 Geotechnical and Seismological study of the territory

To determine the soil shear modulus (G) throughout the foundation depth, two bore holes were drilled very close to the Minaret and the Cross Hole technic was carried out. Resonant Column Tests on foundation red clay samples were also performed. Based on the obtained values of G, the deformational characteristics of appropriate soil springs, necessary for a reliable analytical model, were estimated. The analytical procedure was performed using four elastic acceleration response spectrums. Three of them were based on the seismic excitation of 1978 at the region of Thessaloniki. The motion at bedrock of this earthquake was multiplied by the factors 1.000, 1.267, 1.756 and then, using the SHAKE program [2], the ground motions at the site of Minaret were determined. The first motion (THES 1978, α_{max}=0.172g) corresponds to the earthquake of 1978 since the other two correspond to similar earthquakes for return periods of 75 (THES 75, α_{max}= =0.218g) and 500 years (THES 500, α_{max}=0.302). For the estimation of the latter maximum ground accelerations a statistical evaluation of the strong ground motions at the territory from 1500 till 1990 was performed. The adoption of such a long return period (500 years) was necessary because of the monumental character of the structure and the need of a long-life intervention. The fourth response spectrum corresponds to the new Greek Seismic Code (draft 1989). These response spectrums, after being appropriately smoothed and normalised, were multiplied by the foundation factor θ=0.8 and were devided by the behaviour factor q=1.5 to take design spectrums (see Fig.4).

3. BEARING CAPACITY AT VIRGINAL STATE. ALTERNATIVE STRENTHENING PROPOSALS

3.1 Analytical Model and Verification

The analytical model at virginal state was a vertical cantilever, consisted of 33 overground and 6 underground beam elements. The mass was assumed to be concentrated at the 6-DOF nodes of the model. The stiffness of the foundation was represented by horizontal, vertical nad rotational linear elastic springs. To simulate the mass and stiffness imposed to the structure due to concrete jacket intervention, additional mass at appropriate nodes and new beam elements connected to the same nodes were taken into consideration.

The type of analysis adopted was a linear dynamic spectral analysis, as it is recommended for tall slender type structures [3]. Throughout the analytical procedure, the SAP80/86 Structural Analysis Programs were used [4]. From pilot analyses it was proved that, for the adopted values of E_{wd} and stiffness of the soil springs corresponding to 0.8 G_{max}, the calculated fundamental periods were in very good agreement with those measured in situ.

3.2 Bearing Capacity at Virginal State

Since the shape of the structure practically does not permit the application of significant live loads, the only considerable vertical load is the self-weight of the structure. On the other hand, the Minaret is all around well protected against strong winds. Consequently, the only load combination to be considered is the seismic action and the dead load.

The structure at its virginal state was analyzed under the four design spectrums of Fig.4. The corresponding results are presented in Fig.5a by means of eccentricities of the dead load axial forces due to seismic bending moments (e=M/N). It must be pointed out that these diagrams are more informative than the conventional bending moments and axial forces diagrams since they include their combined action on the structure. From Fig.5a it is obvious that over the height of about 17m, where the eccentricity curves surround the semi-elevation of the structure, the masonry, having an almost zero tensile strength, is unadequate to carry the pair of (N,M) since the axial forces act outside of the cross section.

In order to determine more precisely the region of the structure which needs to be strengthened, the strength curve shown in Fig.5a was determined with the aid of a specific computer program which can calculate the ultimate bending moment, under a given axial force, of a polygonal section composed of various materials (masonry, concrete, steel) and given stress-strain relations of the materials (see Fig.3). It can be seen that under the height of 14m, the structure is adequate to carry out the combination of dead and seismic loads. In order to secure a reasonable safety factor, slightly greater than one, it was decided that the region to be strengthened extends from 12m height to the top of the structure.

It must be pointed out that these analytically obtained findings, concerning the region of inadequasy, match very well with the pathology of the structure (cracking pattern, probable collapse and reconstruction of the upper part).

3.3 Alternative Strengthening Proposals

The following alternative strengthening intervention proposals have been considered and evaluated:
- Internal vertical prestressing tendors
- Internal 3D steel truss
- External inclined lightly prestressed cables
- Internal concrete jacket
- External vertical steel strips nailed to the masonry.

The presence of the internal staircase arises difficulties in applicating anyone of the internal interventions, the 3D steel truss being the most inconvenient since a lot wide drillings are needed. The vertical prestression was proved to be impracticable since the prestressing force needed to adequately decrease the seismic bending moment was too high for the low-strength masonry, especially at the top of the structure. Concerning the application of external inclined lightly prestressed cables, it must be pointed out that there is not enough space around the structure to give to the cables the appropriate inclination. On the other hand, the aesthetics of the monument would be rather inacceptable.

The existance of the internal concrete jacket at the top of the Minaret, which is very difficult to be removed, is a positive argument for the proposal of concrete jacketing to be chosen. Anyhow, it must be pointed out that the vertical reinforcing bars of the jacket have to penetrate the staircase helix to secure the function of the jacket as a tube. Moreover, to secure the unified action of the composite section of masonry and concrete, the bond on their interface must be increased by deepening the mortar joints. From the above mentioned, it is clear that concrete jacketing is an almost irreversible intervention.

The external reinforcing by means of steel strips nailed to the masonry, although it is not a conventional strengthening method, especially for relatively low strength materials, it is a method of rather low inconvenience and almost fully reversible.

After these prelimirary considerations and the rejection of the fisrt three strengthening proposals, detailed analytical investigation of the latter two proposals was performed and is presented in the following. As it was mentioned before, the region to be strengthened extends from 12m height to the top.

3.3.1 Internal concrete jacket

The helical concrete jacket, constructed after the 1978 earthquake at the upper 10m of the structure, had a thickness of 80mm and a longitudinal reinforcement $\Phi 14/150$, S400. It was decided that the already constructed jacket must be extended, having the same geometrical, mechanical and constructional characteristics (see Fig.6a), by welding the existing longitudinal reinforcements to the new ones. Using the modified analytical model mentioned in chapter 3.1, it was proved that the jacketing had a limited influence on the dynamic characteristics of the structure. In Fig.5b, the eccentricities of the axial forces due to seismic bending moments under the design spectrums of Fig.4 are shown. It is obvious that, in comparison to the relative curves of Fig.5a, the differences are small. The calculation of the ultimate bending moment of the cross sections under the acting axial force was based on the specific computer program mentioned in chapter 3.2. From the strength curve, shown in Fig.5b, it can be concluded that the strengthening proposal is adequate.

A calculation of the shear stress acting on the interface between masonry and concrete gives a maximum value of 0.06MPa, which can be easily sustained by the bond strength of the masonry to concrete interface (see also footnote (5) of Table 1), especially after deepening of the mortar joints before concreting. To secure the durability of the intervention, the use of austenitic stainless steel for the reinforcing bars is recommended.

3.3.2 External steel strips nailed to the masonry

After some pilot calculations, the use of ten, grade S400, austenitic stainless steel strips having an effective cross section of 50x5mm was adopted (see Fig.6b). Because of the relatively small increase in stiffness and the practically zero increase in mass due to steel strips, the response of the structure, under seismic loads was assumed to be equal to the one of the virginal state. Using the same computer program, the strength curve shown in Fig.5c was calculated and it can be seen that it is almost identical with that of Fig.5b, which means that the selected strengthening proposals are equivalent in strength.

The maximum shear force per unit length, which has to be transferred from each strip to the masonry, was found to be equal to 29KN/m. It is obvious that the greater the number of the dowels the easier the transfer of the shear force but, on the other hand, the greater the number of intervention points. Finally it was decided to nail the dowels every 0.50m along the strips. To prevent local failure of masonry under the dowel force, small concrete supporting pads will be embedded in the masonry. On the other hand, to avoid buckling under compression, the strips will be stuck on the masonry surface using epoxy resin.

It must be pointed out that the strengthening proposals mentioned above stand under the approval of the appropriate authorities (9th Ephorate of Byzantine Antiquities and Supreme Archaeological Council, Ministry of Culture).

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