Assessment and restoration of the first Greek power plant – Registered monument of industrial heritage

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C. Maraveas *, K. Tasiouli

C. MARAVEAS PARTNERSHIP – Consulting Engineers, Athens, Greece

A R T I C L E   I N F O

A B S T R A C T

The first electric power station in Greece is a registered monument of the international industrial heritage. The building consists of three longitudinal parts with a total area of 4800 m² approximately in plan and has two levels of a height of 3 m and 12 m respectively. The structural system consists mainly of stone masonry walls and a steel roof. Nowadays the building is scheduled to be reused as a Museum of Electric Power and the need for structural upgrade arose mainly from current seismic requirements. According to the structural assessment study, the prevailing problem of the building is the combination of the presence of very high walls, interrupted by transverse walls at a distance of approximately 80 m, and the complete lack of horizontal diaphragms. The building's architectural, historic and technological value is significant and its preservation, by minimization of interventions, posed several problems to the retrofit design. New steel frames connected to and cooperating with the masonry walls were designed to bear the vertical roof loads and restore the horizontal diaphragm at the roof level, while also reducing the seismic actions at the walls. The total required strength was achieved by additionally implementing vertical post-tensioning bars and FRP strips.

Introduction

The building was constructed in 1902 and some modifications to the initial structural system were made during the 1930s and 1950s in four basic construction stages. During the first decade of 2000 the roof sheeting was removed as part of the nationwide project of eliminating all existing asbestos in old buildings, which lead to significant degradation of material mechanical properties (Fig. 1).

The building consists of three longitudinal parts (A, B, C) (Fig. 2). The total plan has a rectangular layout with approximate dimensions of 85 m by 3 parts of 19 m, the roof level is 12 m above ground level and the mezzanine floor (which supports the machinery) is 3 m above ground level. Building A has an internal length of 84.40 m and width of 19.45 m. Building B is smaller, with internal dimensions 77.00 m × 19.50 m. Building C is comprised of two parts and was constructed in two stages during the 1930s, part C1 with internal dimensions 29.30 m × 15.64 m and part C2 with internal dimensions 41.80 m × 14.00 m. The perimeter load bearing masonry walls have a variable thickness and include local thickness strengthening buttresses at the axes of the existing roof trusses, every 5.30 m approximately. The thickness of the walls varies from 0.80 m at the top to 1.30 m at the base and the thickness of the buttresses is about 0.60 m. At building B parts of the walls have been demolished and replaced with reinforced concrete frames and brick masonry infill.

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 * Corresponding author at: 11 Metsovou Street, Athens 10682, Greece. Tel.: +30 2108223402.
   E-mail address: c.maraveas@maraveas.gr (C. Maraveas).

Nomenclature

f_b	Normalized mean compressive strength of the masonry
f_m	Compressive strength of the mortar
f_k	Characteristic compressive strength of the masonry
f_{vk}	Characteristic shear strength of the masonry
f_{xk1}	Characteristic flexural strength of the masonry, plane of failure parallel to bed joints
f_{xk2}	Characteristic flexural strength of the masonry, plane of failure perpendicular to bed joints
γ_M	Partial safety factor of the material
t	Thickness of the wall
h_{eff}	Effective height of the wall
l_w	Length of the wall
l_c	Length of the compressed part of the wall
Z	Elastic section modulus
Φ	Capacity reduction factor allowing for the effects of slenderness
$F_{p0.1k}$	Tensile resistance of prestress bar at 1% strain
P_{omax}	Maximum post-tensioning force of the bar
P_o	Remaining force at the prestress bar after prestress losses
M_{sd1}	Out of plane design bending moment, plane of bending parallel to bed joints
M_{sd2}	Out of plane design bending moment, plane of bending perpendicular to bed joints
$M_{sd\ (in\ plane)}$	In plane design bending moment
N_{sd}	Design axial force
V_{sd}	Design shear force

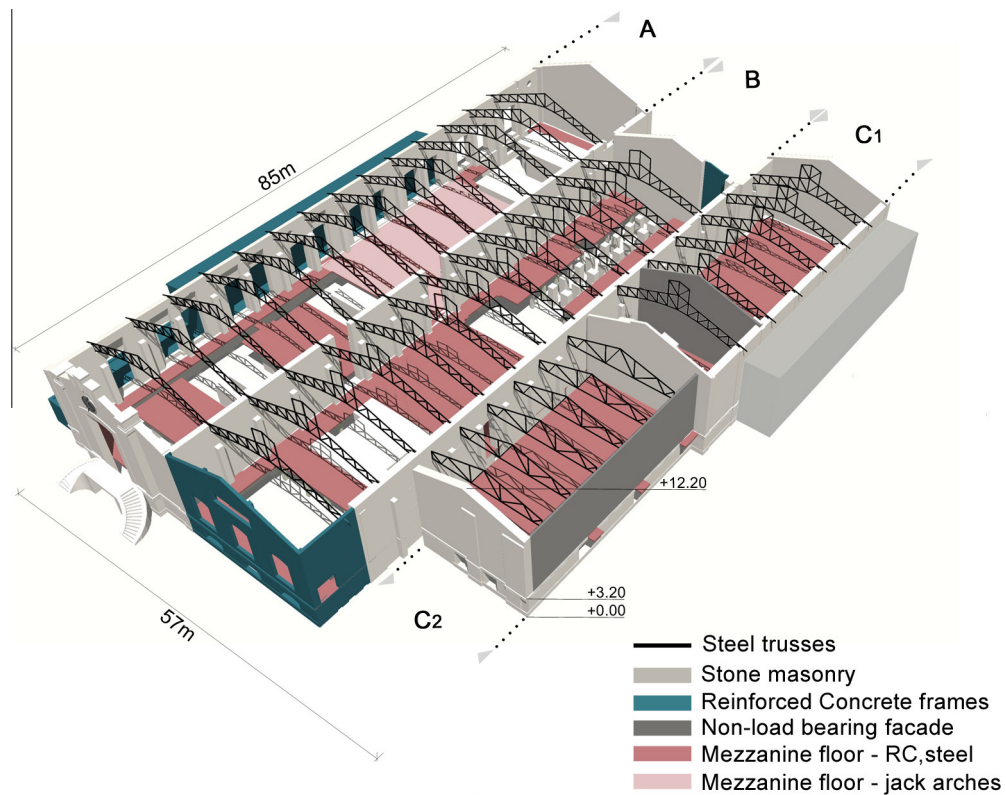Fig. 1. Exterior (a) and (b) Interior of buildings A, B and C2 on the mezzanine level.

Fig. 2. 3D Illustration of the structural layout of the existing building.

The initial mezzanine floor was comprised of jack arches supported by steel frames and masonry walls. Some parts of this type of floor structural system are still preserved today in building A only. The rest were replaced either with typical reinforced concrete frames or steel frames with metal sheeting.

The roof consists of steel trusses with fastened double-angles. The trusses have a variable height (1.30 m at the edges and 2.55 m at the middle), with the lower flange arc shaped, and are supported on the walls. In building C2 (Fig. 2) only, the longitudinal facade is not load bearing and a steel frame is supporting the steel roof trusses whose lower flange is not curved. The mezzanine floor of this part is structurally independent from the masonry. The existing configuration of the structure, which is based on the technological expertise of its time, is safe for bearing vertical loads but not seismic loads, as specified by current regulations. The new use of the building as a museum also increases the demands for structural system strength and serviceability.

Performance and design principles

At the preliminary stage of the design an extended in-situ survey [14] was made in order to establish structural details and estimate loads. Laboratory tests were also conducted on samples taken from the field to evaluate the mechanical properties of materials (Table 1). Specifically, tests were made for the compressive strength of stone masonry and concrete, hardness tests on steel elements and chemical analyses on mortars. In order to perform the capacity assessment the specifications of the Eurocodes were used.

As far as loads are concerned, in addition to the self-weight of the structure, the loads of the machinery were estimated and the wind and snow loads were calculated according to regulations [8–11]. On the basis of EN1998-1 [13], an inelastic

Table 1
Mechanical properties of materials.

Existing		
Masonry	Structural/reinforcement steel	Concrete
f_b = 55 MPa	AISI1010	C12/15
f_m = 0.50 MPa	S220	

response spectrum was adopted for soil type D (soil factor $S = 1.35$ and characteristic response spectrum periods $T_B = 0.2$ s, $T_C = 0.8$ s, $T_D = 2.0$ s), design ground acceleration $a_g = 0.16$ g, importance factor $\gamma I = 1.40$, behaviour factor $q = 1.50$ and damping ratio of 5%.

Simulation – Analysis of existing structure

STAAD.Pro V8i [5] software by BENTLEY was used for the analysis of the structure. A model was created which includes the main shell of the buildings and various additions and neighboring structures but no roof structures as at this stage they do not provide diaphragm, due to their negligible stiffness caused by lack of lateral support, and only transfer vertical loads. The assessment of the structural capacity of the existing steel roof trusses was achieved through separate models and only for loads arising from static forces and not seismic.

The load bearing masonry walls were simulated with finite surface elements (plate elements), which include both membrane and bending function and are ideal for linear elastic shell analysis. The finite element meshing was implemented in such a way as to take into account recesses, openings, differences in thickness etc, in order to realistically simulate the structure geometry. The basement and mezzanine concrete structures and certain neighboring buildings attached to the main shell, which are included in the model because they structurally affect the walls, were simulated with the use of beam or plate elements and their diaphragmatic function was taken into account by imposing common displacements at the nodes of each diaphragm. The basement and mezzanine steel structures do not ensure diaphragmatic function, but their influence is taken into account at the model, since apart from load transferring they also have a kinematic dependence with the shell, by affecting the lateral direction of the walls (Fig. 3). Separate analyses were conducted for all those mezzanine structures that could not ensure diaphragmatic function (steel frames) in order to estimate their horizontal stiffness by calculating the corresponding displacement caused by a horizontal load. Equivalent linear elastic springs with axial stiffness equal to the horizontal stiffness of each frame were then used in the main model in order to simulate their kinematic dependence with the masonry. The aforementioned methodology was adopted in order to simplify the model and to reduce the calculation time. Following this analysis the steel frames structural assessment was made by imposing the calculated displacements to the separate models. Most of those structures proved to be adequate in resisting forces arising from the imposed displacements due to their low stiffness. Direct stiffness method was used to obtain the internal forces. The stiffness of the wall elements was taken as that of the full section for static load cases and as that of the half section for seismic load cases according to EN1998-1 [13].

Structural evaluation – Rehabilitation proposal

The use of plate elements for the simulation of walls in the aforementioned analysis leads to results which do not refer to individual members (equivalent columns, lintels) and thus EN1996-1-1 [12] checks could not be performed directly in force terms. Thus, the results were used initially qualitatively to locate the areas with high stress and then at the critical members
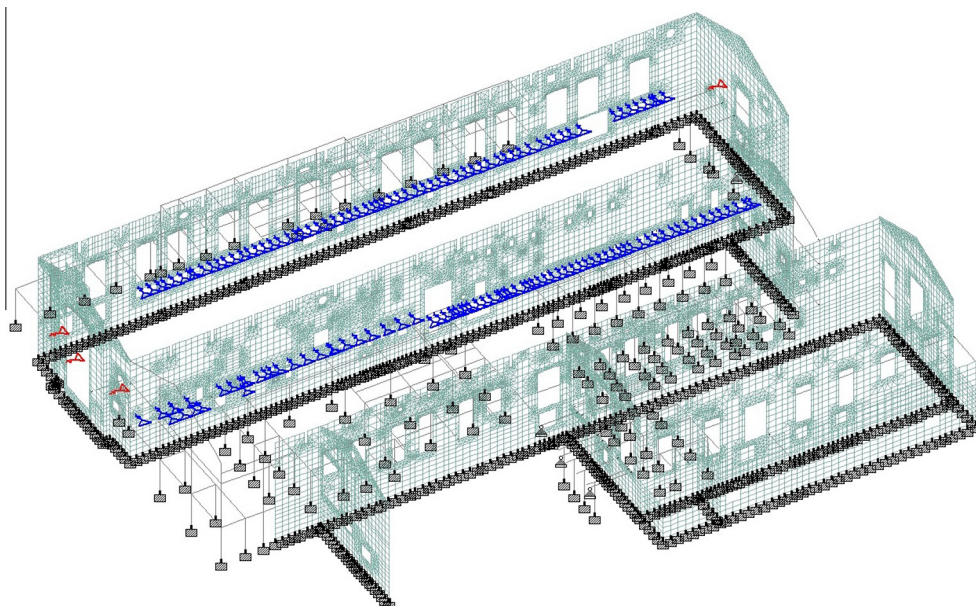

Fig. 3. 3D finite element model of the existing structure.

design checks were performed by integrating the stresses along a cut line. The design resistances of unreinforced masonry according to EN1996-1-1 [12] are summarized below:

$$N_{Rd} = \phi \cdot t \cdot f_k / \gamma_M \ (\text{kN/m}) \tag{1}$$

$$V_{Rd} = f_{vk} / \gamma_M \cdot l_c \cdot t \ (\text{kN}) \tag{2}$$

$$M_{Rd,1} = (f_{xk1} + \sigma_d) / \gamma_M \cdot Z \ (\text{kNm/m}) \tag{3}$$

$$M_{Rd,2} = f_{xk2} / \gamma_M \cdot Z \ (\text{kNm/m}) \tag{4}$$

Results showed that the seismic vulnerability of the structure is very high. The response of the structure in horizontal loads over the mezzanine floor is similar to that of a 10 m cantilever, due to the absence of horizontal diaphragm and transverse walls. Stability of the unreinforced masonry walls could not be ensured under the combination of large out of plane bending moments, small axial load and slenderness ratios approximately equal to 10.

The main goal for the retrofit was to restore the lateral stability of the walls. Architectural and historic restrictions regarding the preservation of the original characteristics of the building eliminated any proposal along the lines of construction of new transverse intermediate walls or bracing, or the addition of intermediate floor levels. Any diaphragm could only be constructed at the roof level. In buildings A and B, the existing arced steel truss girders, which due to their significant axial stiffness could have provided some sort of lateral support, could not be reused as structural members due to their high level of corrosion, but nevertheless they have to be preserved as a historic part of the building. On the contrary, the steel truss girders in building C1 may be demolished and replaced, because since they were placed at a later date their historic value was deemed unimportant. In building C2 the existing roof has collapsed due to failure of the compression members of the steel truss girders, and has to be completely replaced.

Based on the aforementioned factors, 15 pairs of new steel rigid frames and horizontal X-shaped bracing at the roof were proposed for the structural rehabilitation of the overall stability of the walls and roof in buildings A and B (Fig. 4). The new frames were aligned symmetrically on both sides of the existing girders and their sections were chosen as standard H-shaped so that architecturally the intervention is as discreet as possible. The top flange of the girders was placed higher than the top chord of the existing truss girders in order to provide enough space for the inclusion of the horizontal bracing and so that the roof loads are transferred only to the new girders. The new steel columns are connected to the masonry walls with anchor bolts every 1 m approximately along the height in order to ensure kinematic constraint and cooperation with the walls. The role of these connections is primarily to facilitate the transfer of forces from walls to the new roof diaphragm and secondarily to support the walls and enhance their performance in out plane bending (Fig. 8). In building C the roof is completely reconstructed with trusses following the original geometry and new horizontal X-shaped bracing.

A new analysis was then conducted again using STAAD.Pro V8i [5] software by BENTLEY. For this analysis a complete model was created by adding the steel roof structures to the original model (Fig. 5). The steel roof structures were simulated with linear members (beam elements). The secondary members of the trusses were specified as truss members (only taking axial force), while the horizontal and vertical bracings were specified as tension only members (only taking tensile force). Purlin members were included in the model. The existence and contribution of the preserved old roof trusses was ignored. The all important structural cooperation of the walls with the roof steel structures, which in reality is achieved by connecting them along the height using anchor bolts at regular intervals, is simulated by using pairs of truss members of appropriate stiffness connecting the steel column nodes to the nodes of wall elements.

Post-stressing and strengthening masonry

With the implementation of the structural interventions described above, the roof diaphragm was retrofitted and an approximate 40% reduction was achieved in the out of plane moments at the base of the walls (Table 2, Fig. 6). However,

Fig. 4. Render of the new pairs of steel frames aligned symmetrically with the existing steel truss girders in building A.
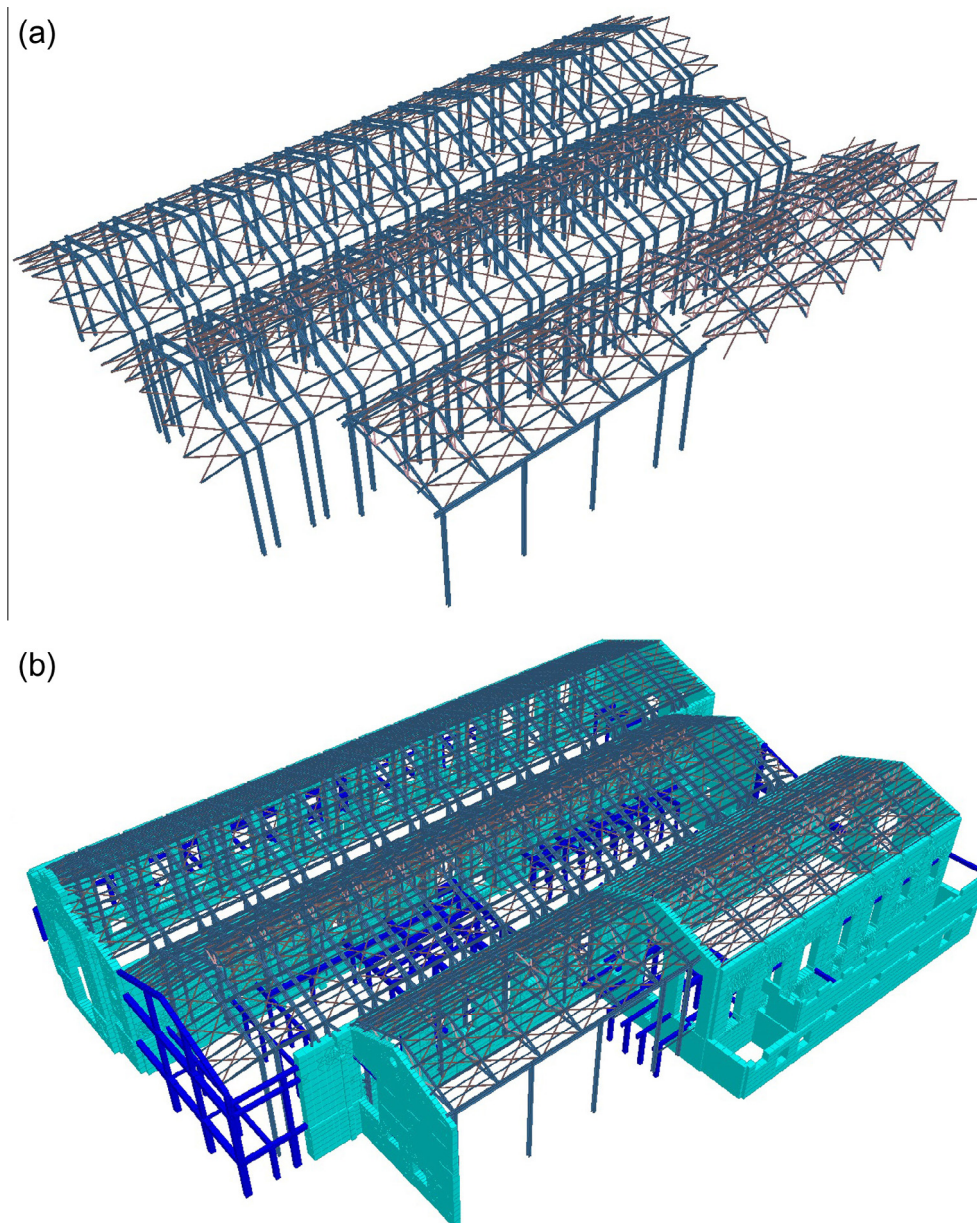
Fig. 5. Simulation of the structure (a) Structural layout of new steel structures (b) 3D model of the structure with the interventions.

Table 2
Design forces on the base of an equivalent column of the outer longitudinal facade wall.

t (m)	h_{eff}	l_w (m)	Analysis	N_{sd} (kN)	V_{sd} (kN)	M_{sd2} (kNm/m)	M_{sd1} (kNm)	$M_{sd\ (in\ plane)}$ (kNm)
0.90	9.65	3.90	Existing	−939.6	−369.2	−100.4	−2263.9	−23.1
			New frames	−975.4	−204.6	−27.8	−1400.3	87.3

the effective height of the walls is approximately 9.5–11.0 m and the overturning design moments resulting mainly from the inertia forces of the wall remained high relatively to the design bending resistance of the unreinforced masonry which is almost negligible, so still the walls had to be strengthened. At the preliminary stage of the design the use of reinforced concrete or carbon fiber reinforced polymer (CFRP) strips was investigated for the strengthening of the flexural resistance of the masonry. The use of shotcrete is not recommended, as this type of intervention is irreversible and thus conflicts with the guidelines for interventions in historic structures. In addition, the application of CFRP strips (in 5 levels) does not ensure
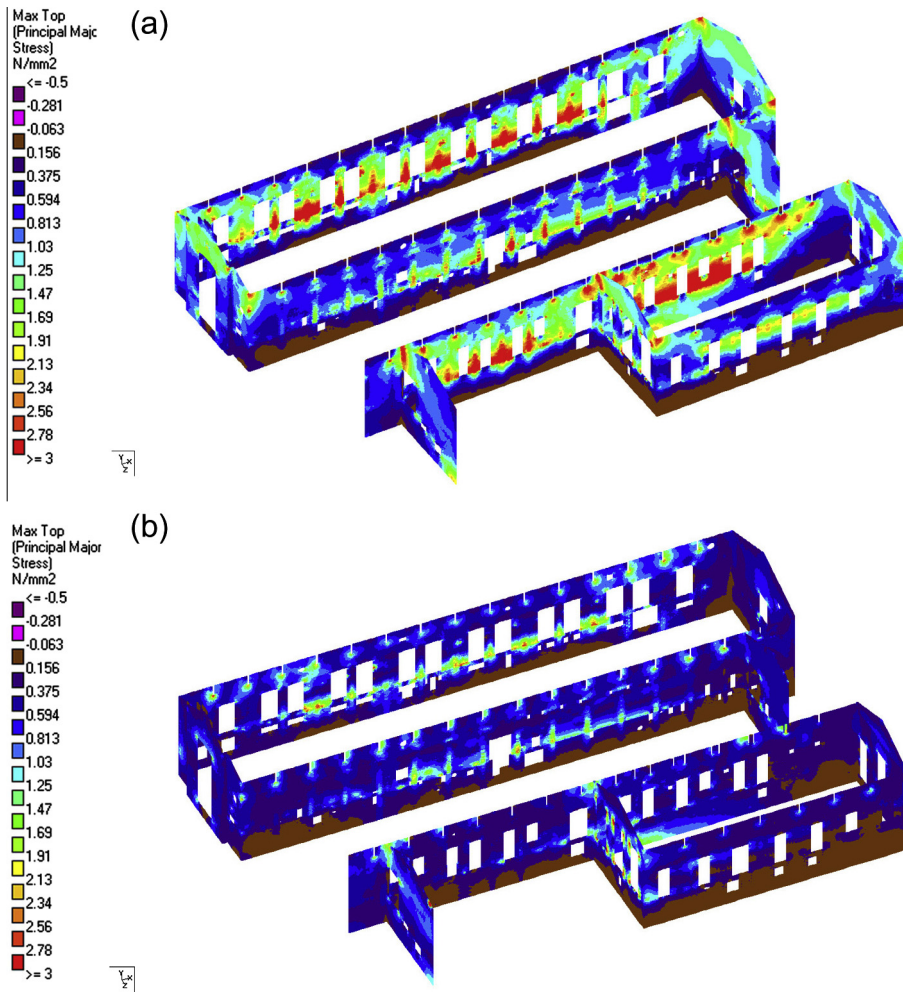
Fig. 6. Maximum principal (major) stresses (a) Existing (b) After the interventions. Favorable areas indicate tensile stresses lower than 0.1 MPa.

stability because the bond strength between the strips and masonry cannot be exactly estimated. Moreover, the outer facade of the building must remain uncoated on the base level, something which poses another restriction to the anchorage of CFRPs. Thus the solution proposed is to apply vertical post-tension to the masonry walls according to the following scheme (Fig. 8): Circular cavities of 150 mm diameter are drilled from the top of the masonry by center core drilling every 2 m approximately. An in situ concrete perimeter bond beam is then constructed with circular cavities. An SAS post-tensioning bar system Ø47 is inserted from the top of the wall into every cavity, then stressed at the live end which is placed on the perimeter bond beam at the top of the walls and finally grouted over. Before the application of prestress, cement injections for homogenizing the masonry are applied.
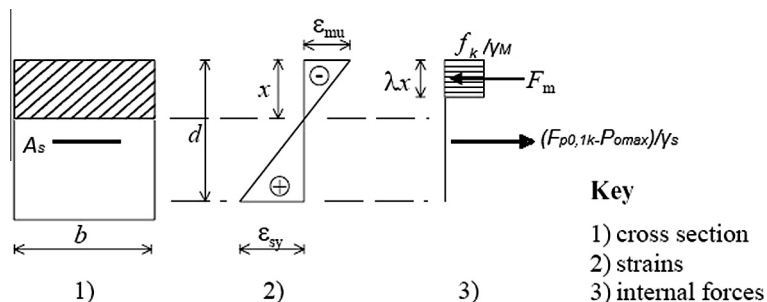

Fig. 7. Strain–stress distribution and internal forces at failure of prestressed masonry.

Post-tensioning "allows the utilization of masonry compression strength to compensate for seismic or wind induced lateral forces" [1] and also increases the shear strength of the wall due to the increase of compressive stresses.

In the present design the bars were partially prestressed so as to act also as reinforcements for the cross section. The design concept was to bear the imposed seismic loads by allowing some cracking to occur within the bar force increase. At the serviceability and ultimate limit states any tensile stresses should preferably be avoided.
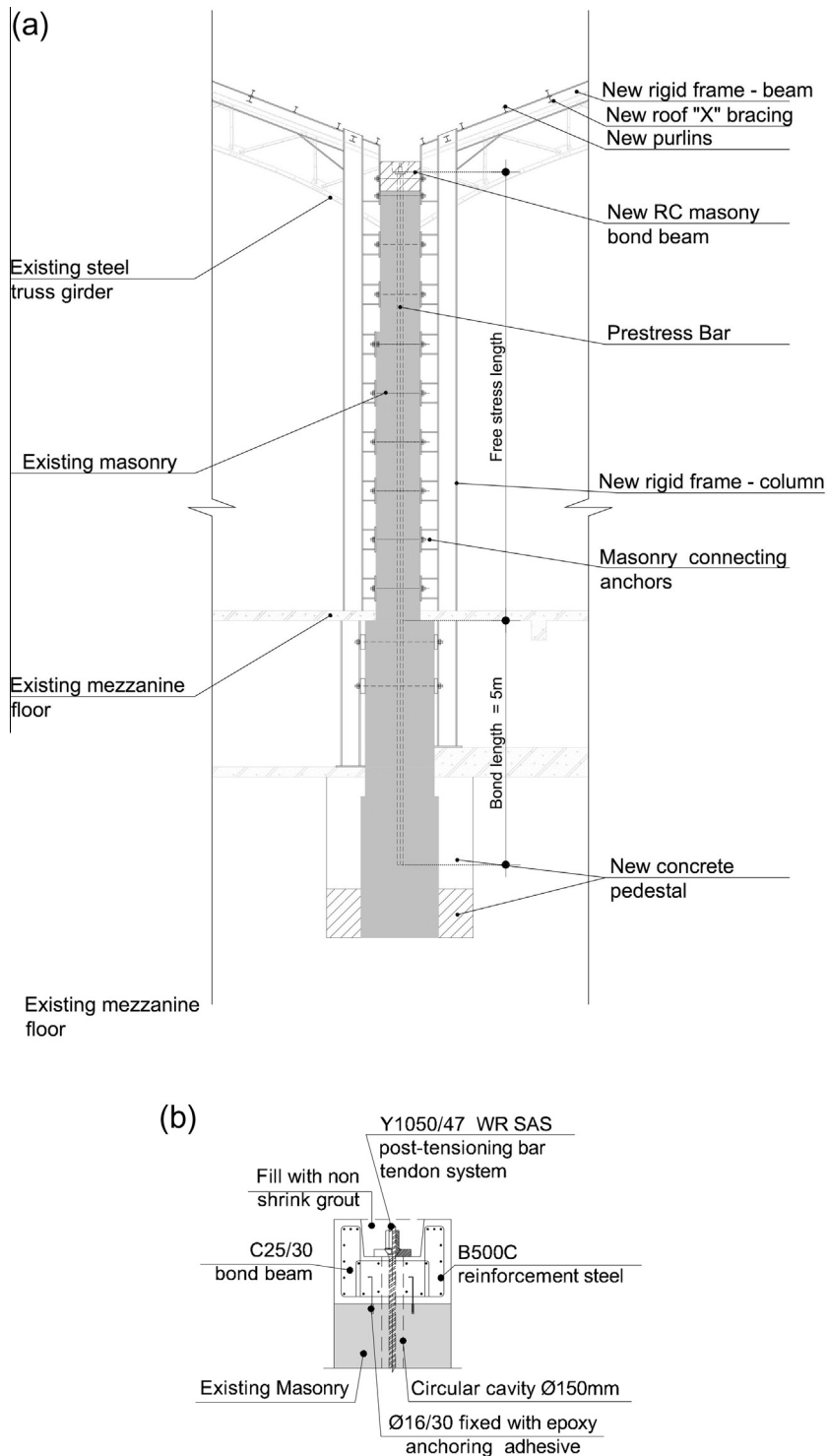

Fig. 8. (a) General layout of structural interventions (b) Structural detail of post-tensioned masonry on the top of the wall (live end).

According to EN1996-1-1 [12], the design of prestressed / reinforced masonry members should be based upon the assumptions that the stress distribution over the compressive zone is uniform and does not exceed f_k/γ_M, the limiting compressive strength of masonry is $\varepsilon_{mu} = -0.0035$, the tensile strength of masonry is ignored and the tensile strain of the reinforcement ε_s is limited to 0.01 (Fig. 7).

The initial prestress force was estimated t taking into consideration the following factors. The local bearing and lateral bursting tensile forces at the end of the anchorage while post-tensioning must be limited to an acceptable level so as to avoid ultimate load failure. According to BS 5628-2 [7] the local bearing stress after locking of the prestress bar should not exceed $0.65 f_k/\gamma_M$ for the design of new masonry walls. In the present Design, initial stresses equal to $0.25 f_k/\gamma_M$ [6] (approximately 940 kN per prestress bar) were adopted in order firstly to take account of the uncertainties in the estimated masonry design resistance and secondly in order to limit the final (post losses) stresses in the cross section to $0.30 f_k/\gamma_M$, so as to have bending and not axial forces as the critical design parameter. Additionally, depending on the slenderness ratio of the wall, the axial capacity reduction factor Φ allowing for the effects of slenderness was taken into consideration where appropriate.

For the design of the cross section the initial force was reduced taking into account the prestress losses. Prestress losses which result from the combination of shrinkage in the masonry, anchorage draw-in and friction were taken into consideration. It should be mentioned that elastic deformations of masonry were neglected because the sequence of stressing allows for the elastic deformation to take place during tensioning and thus no loss on prestress will occur. The prestress losses were calculated at approximately 35% of the initial force. Due to the uncertainties in the estimation of the elastic modulus of the masonry, it is recommended that after the initial stressing the estimated prestress force is verified.

Following the above and according to Paulay and Priestley [4] the design resistance of walls was calculated according to:

• For slenderness ratio of the wall <12

$$N_{Rd,s} = t \cdot f_d \ (\text{kN/m}) \tag{5}$$

$$M_{Rd1} = \left(N_s d + \frac{P_0}{Y_S} + \frac{F_{p01k} - P_{0max}}{Y_S}\right) \cdot \left(\frac{t}{2} - \frac{a}{2}\right) \leq 0.40 \cdot f_d \cdot l_w \cdot t^2 \ (\text{kN}) \tag{6}$$

$$a = \frac{\left(N_{sd} + \frac{P_0}{Y_S} + \frac{F_{p01k} - P_{0max}}{Y_S}\right)}{0.85 \, f_d \, l_w} \leq t \tag{7}$$

• For slenderness ratio of the wall >12

$$N_{Rd,s} = \phi \cdot t \cdot f_d \ (\text{kN/m}) \tag{8}$$

$$M_{Rd1} = \left(N_s d + \frac{P_0}{Y_S} + \frac{F_{p01k} - P_{0max}}{Y_S}\right) \cdot \left(\frac{t}{2} - \frac{a}{2}\right) - \frac{\left(N_{sd} + \frac{P_0}{Y_S}\right)h_{eff}^2}{2000 \cdot t} \leq 0.40 \cdot f_d \cdot l_w \cdot t^2 \ (\text{kN}) \tag{9}$$

A major challenge for the design was the anchorage of prestress bars during post tensioning, as they are inserted into the non accessible core of drilled circular cavities. Thus a procedure akin to rock anchoring was adopted by grouting the fixed/ bond length before stressing. The required length was calculated according to EN1996-1-1 [12] for confined concrete infill stronger than C25/30 as long as the cavity is at least 150 mm in diameter and the grouting is specified as equivalent of C30.

Supplemental strengthening of the shear resistance of specific walls and most of the equivalent beams was achieved with the application of CFRP strips with a sectional area of 100 mm × 1.40 mm on each side of the walls. For design in shear with CFRP the effective strain was limited to 0.004 [2] in order to account for loss of bonding.

Finally, in order to take account of the variability of the estimation of the mechanical characteristics of masonry, a Sika TRM system was applied in all the surfaces [3]. It is comprised of a Glass Fiber Reinforced Polymer Grid (GFRP) and high strength mortar. The system is anchored to the masonry with special anchors of SikaWrap C type. These anchors also play the role of transferring small tensile stresses which may arise during post tensioning.

Conclusions

Most historic monuments of industrial heritage constructed during the first decade of 1900 in Europe have the same structural system, that of stone masonry walls in longitudinal layout in plan and approximately 6–10 m high which support steel truss girders on the roof. The main characteristic of unreinforced masonry walls is that their bearing resistance to vertical loading is very high while their tensile resistance is negligible. Current seismic requirements in Greece drive the need for structural upgrade of these structures, but due to the building architectural, historic and technological value the need for minimum and reversible interventions is crucial. In the present Design, new steel rigid frames and truss girders with stiff horizontal X-shaped bracing are proposed in order to ensure a diaphragm in the roof level, reduce out of plane bending moments in the walls and bear the vertical loads of the roof. Post-tensioning with high tensile threaded bars is applied in order to allow the utilization of masonry in compression which results in out of plane minimization of overturning and in plane shear strengthening. Due to architectural restrictions the prestress bars are placed into the core of circular cavities

and in the first step the bottom part of the cavity is grouted so as to ensure anchorage during post-tensioning with the proper bond length. The whole mass of masonry is homogenized with grouting before the application of post-tensioning. Supplementary GFRPs for strengthening in shear of the equivalent beams and some walls are used and Sika TRM system is applied on the wall surface to account for small random tensile stresses. With the implementation of the proposed structural scheme the building is capable of bearing of the loads specified by the current regulations and its architectural form is optimally preserved.

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