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ASSESSMENT AND STRENGTHENING MEASURES OF THE HISTORIC MASONRY COMPLEX: "PROSFYGIKA" IN CENTRAL ATHENS

Constantine C. Spyrakos [1], Eleni Panou-Papatheodorou[2], and Dimitrios Kokkinakos[3]

[1] Professor and Director of Lab for Earthquake
Engineering at NTUA
cspyrakos@gmail.com

[2] Civil Engineer M.Sc., Ph.D. Cand. N.T.U.A.

[3] Civil Engineer M.Sc. N.T.U.A.

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Abstract. *This paper presents the results of the analysis regarding the seismic risk assessment of a historic three-story masonry structure, part of a building complex in central Athens. The building complex has been declared as preserved because of its historic, architectural and structural characteristics.*

Based on data selected through in situ measurements both linear and nonlinear (pushover) analysis as well as kinematic analysis have been conducted. The analysis has revealed the need for limited strengthening measures in accordance with state-of-the-art practice on monumental structures.

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1 INTRODUCTION

The construction of the historic complex "Prosfygika" located in central Athens dates back to the 1930s. It consists of eight three-story blocks of flats, with a total number of 228 flats, constructed in an area of 14500m^2. The complex was constructed to house Greek refugees that arrived from the East Minor. Because of its historical, architectural and structural characteristics the complex has been declared a monument by the state.

Figure 1: "Prosfygika" historic complex.

The dimensions of each building are 69.60m (length of the facade) by 8.90m (width) by 10.60m (height). The load bearing walls are constructed of stone masonry and the floors of reinforced concrete. The load bearing masonry walls are interrupted by reinforced concrete floors, a characteristic that attributes to them a historical character in the evolution of building construction and is the primary reason for their enhanced seismic behavior. They represent the predecessor of the modern reinforced concrete construction with infill walls.

Figure 2: Façade of a typical building of the complex.

Today these buildings have deteriorated to a certain extent and proper maintenance is necessary. However, there is no significant damage on the load bearing walls of the structure.

2 ANALYSIS OF THE STRUCTURE AT ITS CURRENT STATE

Several in-situ tests at different locations of a representative structure tests were carried out in order to assess the mechanical characteristics of the load bearing stone masonry. Based on measurements, e.g., tests shown in Figure 3, the compressive strength of the stones was measured to be: f_{bc} = 50MPa. Based on the characteristics and the composition of the poor quality mortar, its compressive strength was taken as: f_{mc} = 1MPa.

With the aid of an endoscope, the composition of the masonry walls along their thickness (Figure 4) was examined showing relatively large voids of a construction closely resembling to a three-leaf masonry.

Figure 3: Rebound hammer test.

Figure 4: Endoscope test.

Three types of analysis were performed: a) Modal Response Spectrum Analysis, b) Non-linear Static (Pushover) Analysis and c) Kinematic Analysis.

Three different models of the structure were created, each one suitable for the corresponding analysis type. For the modal response spectrum analysis, the load bearing walls were modeled using shell elements [1, 2]. For the pushover analysis, frame elements were used. For the kinematic analysis parts of the load bearing walls were modeled as rigid bodies (macro-elements) [3].

The static loads used for the analysis were: Stone masonry self-weight: 26 kN/m^3; concrete self-weight: 25 kN/m^3; floor additional load: 1.50 kN/m^2; floor live load: 2.00 kN/m^2.

In order to calculate the compressive strength of the stone masonry, the following expression was used [4]:

$$f_{wc} = \xi \cdot \left[\left(\frac{2}{3} \cdot \sqrt{f_{bc}} - f_o \right) + \lambda \cdot f_{mc} \right] \tag{1}$$

where:

f_{wc} is the compressive strength of the masonry

V_m, V_w is the volume of the mortar and the volume of the masonry, respectively

f_{bc} is the compressive strength of the stones

λ is a bond factor between stones and mortar. It can be taken as equal to 0.50 for rough stones and equal to 0.1 for very smooth stones

f_0 is a factor (MPa), that takes into account the degree of carving of the stones. It takes the following values
0.00 for carved stone masonry
0.50 – 1.00 for stone masonry constructed of partially carved stones
1.50 – 2.50 for stone masonry constructed of stones without carving, depending on the construction quality

ξ is a factor that accounts for effect of the thickness of the mortar joints, where:
$\xi = 1 : [1 + 3.5 \cdot (k - k_0)] < 1.0$
$k =$ (mortar volume) / (masonry volume) $= V_m / V_w$
$k_0 = 0.30$
if $V_m / V_w < 0.30$ then the value received is $\xi = 1.00$

Considering the medium roughness of the stones, $\lambda = 0.25$. The stones used are without carving and medium construction quality, thus, $f_o = 2.0$. The ratio of the volume of the mortar to the volume of the masonry is lower than 0.30, thus, $\xi = 1.00$.

In order to calculate the compressive strength of the stone masonry, the following expression was used [4]:

$$f_{wc} = \frac{1}{\gamma_{Rd}} \cdot \left(2 \cdot \lambda_e \cdot \delta \cdot f_{c,e} + \lambda_i \cdot f_{c,i}\right) / \left(1 + 2 \cdot \delta\right) \tag{2}$$

where:

δ the ratio of the thickness of the outer leaf to the thickness of the inner leaf

$f_{c,e}, f_{c,i}$ the compressive strength of the outer leafs and the inner leaf, respectively

λ_e, λ_i empirical factors, that account for the interaction between the outer leafs and the inner leaf. They are taken as equal to 0.80 and 1.20, respectively

γ_{Rd} uncertainty factor, considered equal to 1.50

The building is constructed from a three-leaf masonry with a total thickness of 60cm and a thickness of 20cm for each one of the three leafs.

By applying expressions (1) and (2) the compressive strength of the masonry is calculated: $f_{wc} = 1.84 MPa$.

The modulus of elasticity was obtained from the masonry compressive strength according to the following expression:

$$E_{wc} = 1000 \cdot f_{wc} \tag{3}$$

Table 1 presents the data used for the analysis, as well as the results obtained from Equations (1 - 3):

Stones	Compressive Strength	f_{bc}	50	N/mm^2
Mortar	Compressive Strength	f_{mc}	1	N/mm^2

Masonry	Compressive Strength	f_{wc}	1.84	N/mm^2
	Tensile Strength	f_{wt}	0.20	N/mm^2
	Shear Strength	τ_0	0.028	N/mm^2
	Modulus of Elasticity	E_{wc}	1840	N/mm^2
	Specific Weight	W	26	kN/m^3

Table 1

The knowledge level was considered as "KL1: Limited knowledge" according to Eurocode 8-3 [5], since the number of the tests performed was limited. Thus, the value of the confidence factor is: CF_{KL1} = 1.35. However, in order to evaluate the influence on the results, the calculations were repeated for the highest knowledge level “KL3: Full knowledge”. In this case, the value of the confidence factor is: CF_{KL3} = 1.00.

The complex was constructed during a period when there were no seismic regulations in Greece. As a result no seismic analysis was performed at the time. The following assessment has been performed for the three different seismic loads according to the provisions of the three different seismic regulations that have been implemented in Greece over the last six decades. These three regulations are the following: (i) Seismic code of 1959 [6]; (ii) seismic code of 1985 [7]; (iii) Eurocode 8 (EC8) [5].

2.1 Modal response spectrum analysis

As stated in §3.1, for the modal response spectrum analysis the load bearing walls were modeled using shell elements. The model of the structure is presented in Figure 5:

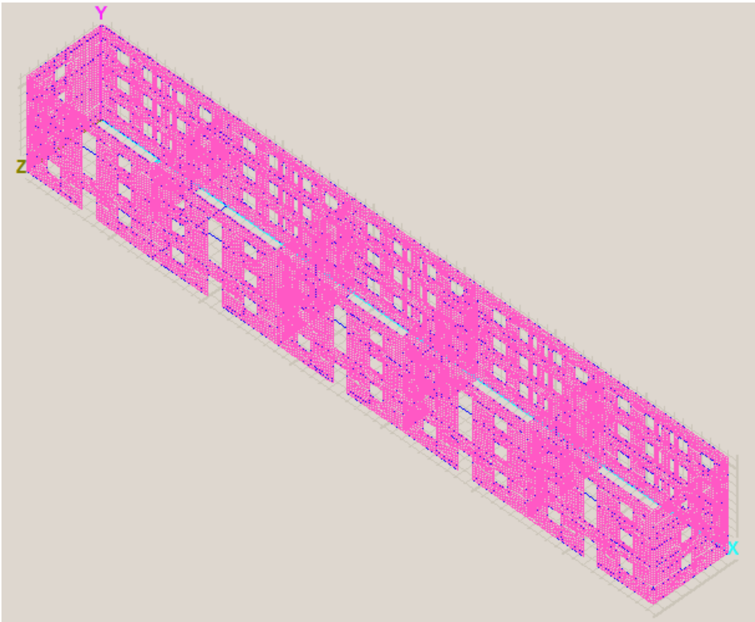

Figure 5: 3D model of the structure using shell elements.

According to the seismic code of 1959 [6] the maximum response acceleration of the structure is $\varepsilon = 0.07g$. The seismic code of 1985 [7] specifies as the maximum response acceleration of the structure the same value, that is $\varepsilon = 0.07g$, but in this regulation an importance factor of 1.5 is also taken into account for structures of major importance. As a result, the response acceleration was increased to $\varepsilon = 0.07 \cdot 1.5 = 0.105g$, in order to account for the historical importance of the structure.

The spectra, both elastic and design, are presented in Figure 6, the latter obtained by reducing the elastic by application of the reduction factor $q = 1.5$.

Figure 6: Response spectra: (a) design 1959 [6]; (b) elastic 1959 [6]; (c) design 1985 [7]; (d) elastic 1985 [7].

According to Eurocode 8-1 [5], for the horizontal components of the seismic action, the design spectrum, $S_d(T)$, is defined by the following expressions:

$$0 \leq T \leq T_{\mathrm{B}}:\ S_{\mathrm{d}}(T) = a_{\mathrm{g}} \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_{\mathrm{B}}} \cdot \left(\frac{2{,}5}{q} - \frac{2}{3}\right)\right] \tag{4}$$

$$T_{\mathrm{B}} \leq T \leq T_{\mathrm{C}}:\ S_{\mathrm{d}}(T) = a_{\mathrm{g}} \cdot S \cdot \frac{2{,}5}{q} \tag{5}$$

$$T_{\mathrm{C}} \leq T \leq T_{\mathrm{D}}:\ S_{\mathrm{d}}(T) \begin{cases} = a_{\mathrm{g}} \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_{\mathrm{C}}}{T}\right] \\ \geq \beta \cdot a_{\mathrm{g}} \end{cases} \tag{6}$$

$$T_{\mathrm{D}} \leq T:\quad S_{\mathrm{d}}(T) \begin{cases} = a_{\mathrm{g}} \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_{\mathrm{C}} T_{\mathrm{D}}}{T^2}\right] \\ \geq \beta \cdot a_{\mathrm{g}} \end{cases} \tag{7}$$

where:

$S_d(T)$ is the design spectrum; T is the vibration period; a_g is the design ground acceleration on type A ground $(a_g = \gamma_I \cdot a_{gR})$); γ_I is the importance factor; S is the soil factor; q is the behavior factor; T_B is the lower limit of the period of the constant spectral acceleration branch; T_C is the upper limit of the period of the constant spectral acceleration branch; T_D is the value defining the beginning of the constant displacement response range of the spectrum; β is the lower bound factor for the horizontal design spectrum $(\beta = 0.2)$.

The corresponding values of the parameters for the structure are: $a_{gR} = 0.16g$ for the seismic zone Z1; $\gamma_I = 1.0$; 1.2 for importance class II and III, respectively; $S = 1.2$ for ground type B; $q = 1.5$ for unreinforced masonry; $T_B = 0.15\sec$, $T_C = 0.5\sec$, $T_D = 2.5\sec$ for ground type B.

The response spectrum according to the provisions of Eurocode 8-1 [5] for $\gamma_I = 1.0$ is presented in Figure 7:

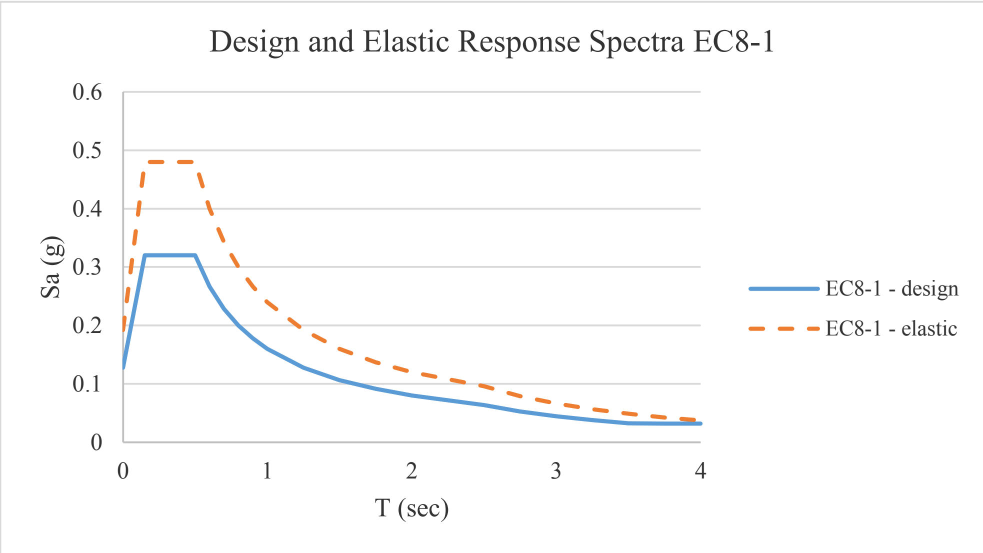

Figure 7: Design and elastic response spectra Eurocode 8-1 [5].

From the assessment of the structure for the seismic loading according to the seismic code 1959 [6] and for limit state of damage limitation (DL), a significant number of failures (in bending and shear) were observed (approximately 44% of the piers).

According to Eurocode 8-3 [5] the damage limitation (DL) is a limit state where the structure is only lightly damaged with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Permanent drifts are negligible. The structure does not need any repair measures. As a result, the structure is insufficient for limit state of damage limitation (DL) even for the lowest seismic loads. Consequently, the structure is assessed for the limit state of significant damage (SD) as elaborated in the following section.

2.2 Non-linear static (pushover) analysis

The pushover analysis is the most reliable analytical method to assess masonry structures, after the non-linear time history analysis, which is rarely implemented [3]. The pushover analysis can assess the structure more accurately for the limit state of significant damage (SD) and at the same time estimate the actual value of the behavior factor, q [8, 9, 10].

As stated in §3, for the pushover analysis the load bearing walls are modeled using frame elements. The model of the structure is presented in Figure 8:

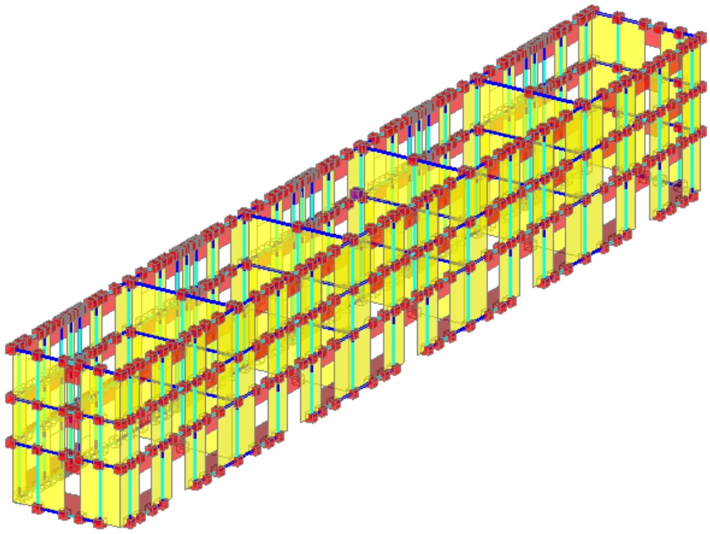

Figure 8: 3D model of the structure using frame elements.

In order to perform the pushover analysis the calculation of the elastic response spectrum for the three cases of seismic loading is used, that is the curves (b) and (d) of Figure 6. Thus, the maximum elastic response acceleration of the structure is $\varepsilon = 0.07 \cdot 1.5 = 0.105g$ for the seismic code of 1959 [6]. According to the seismic code of 1985 [7] the maximum elastic response acceleration of the structure is $\varepsilon = 0.105 \cdot 1.5 = 0.1575g$.

According to Eurocode 8-1 [5], for the horizontal components of the seismic action the elastic spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \le T \le T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \tag{8}$$

$$T_B \le T \le T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \tag{9}$$

$$T_C \le T \le T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T}\right] \tag{10}$$

$$T_D \le T \le 4s : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C T_D}{T^2}\right] \tag{11}$$

where:

$S_e(T)$ is the elastic response spectrum; η is the damping correction factor with a value of $\eta = 1$ for 5% viscous damping.

The elastic response spectrum according to the provisions of Eurocode 8-1 [5] is presented in Figure 7.

The pushover analysis is performed using two lateral load distributions: (i) a "unimodal" distribution (A) with lateral forces proportional to the fundamental mode of vibration; and (ii) a "uniform" distribution (B) with lateral forces proportional to mass regardless of elevation.

From the assessment of the structure for the seismic loading according to the seismic code 1985 [7] and for limit state of significant damage (SD), the capacity curve of the structure has been calculated. An indicative capacity curve for the multi degree-of-freedom system is presented in Figure 9:

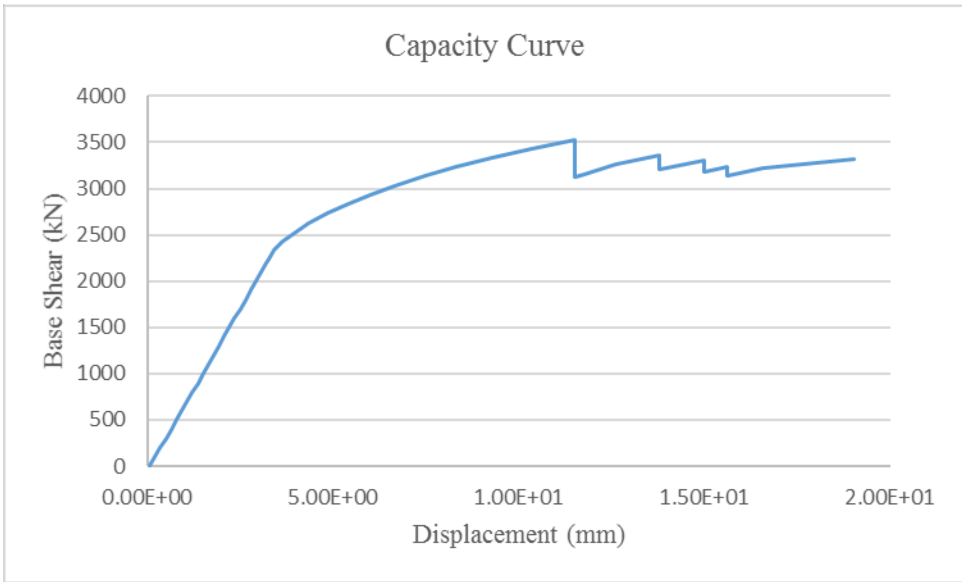

Figure 9: Indicative capacity curve for the seismic load of the combination Y+0.3X for the unimodal lateral load distribution.

According to Eurocode 8-3 [5] the significant damage (SD) is a limit state where the structure is significantly damaged with some residual lateral strength and stiffness, while the vertical elements are capable of sustaining vertical loads. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair.

In Figures 10-13 four bilinear capacity curves of the equivalent single-degree-of-freedom system are presented, out of the sixteen analysis performed in total. The seismic loading of this analysis is that of the seismic code 1985 [7]. The result is that the structure is sufficient for this level of seismic loading.

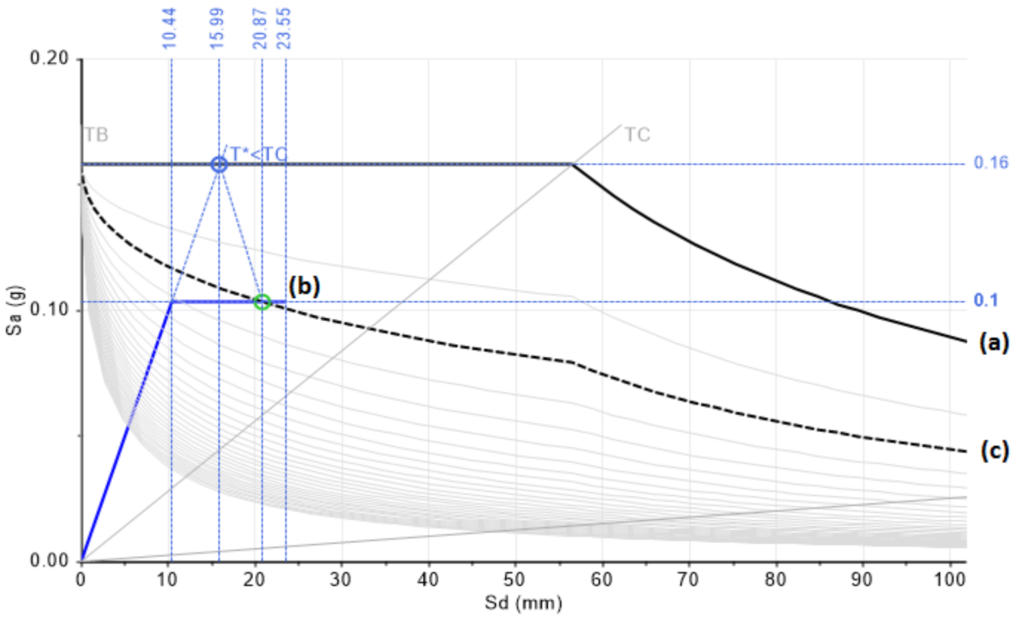

Figure 10: Bilinear capacity curve and target displacement for the seismic load for the combination X+0.3Y with the unimodal lateral load distribution. (a) 1985 [7] elastic spectrum; (b) bilinear capacity curve; (c) 1985 [7] nonlinear spectrum.

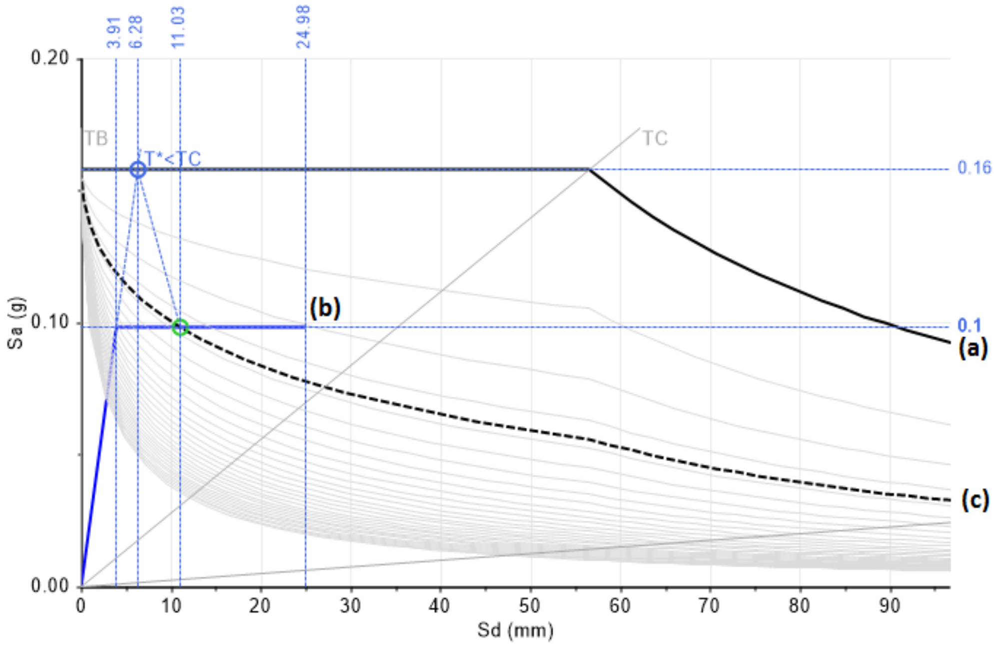

Figure 11: Bilinear capacity curve and target displacement for the seismic load for the combination Y-0.3X with the unimodal lateral load distribution. (a) 1985 [7] elastic spectrum; (b) bilinear capacity curve; (c) 1985 [7] non-linear spectrum.

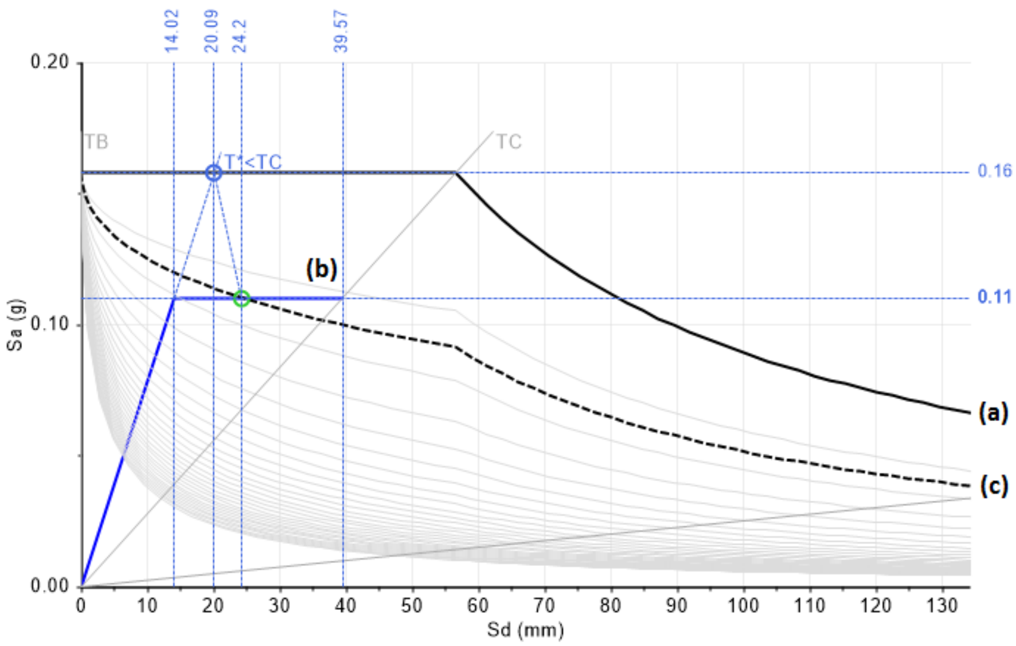

Figure 12: Bilinear capacity curve and target displacement for the seismic load for the combination -X+0.3Y with the uniform lateral load distribution. (a) 1985 [7] elastic spectrum; (b) bilinear capacity curve; (c) 1985 [7] nonlinear spectrum.

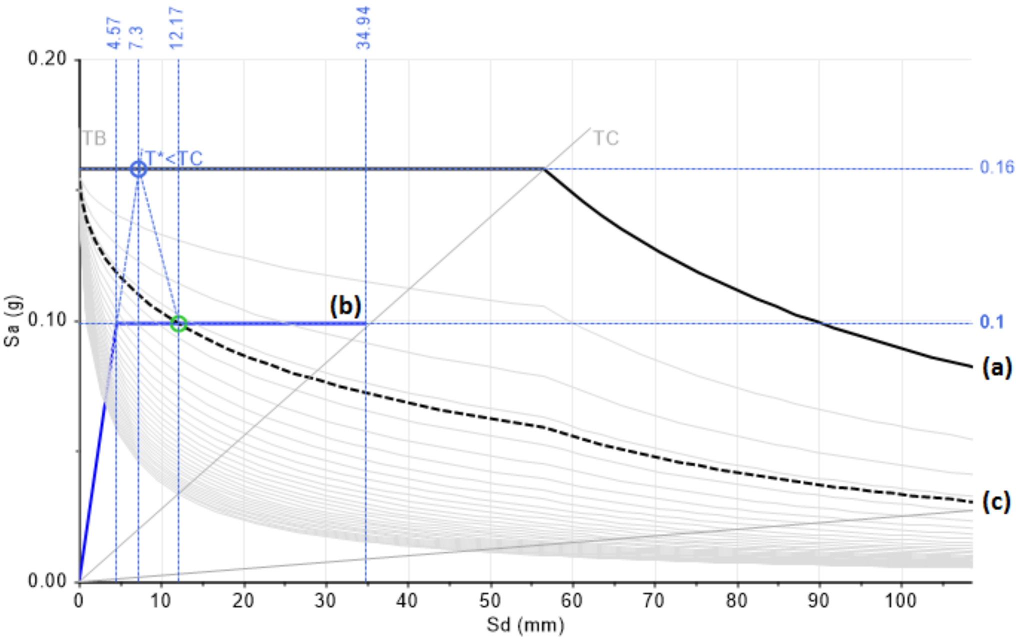

Figure 13: Bilinear capacity curve and target displacement for the seismic load for the combination -Y-0.3X with the uniform lateral load distribution. (a) 1985 [7] elastic spectrum; (b) bilinear capacity curve; (c) 1985 [7] non-linear spectrum.

It should be also stated that the structure is insufficient for the seismic loading according to Eurocode 8 [5].

2.3 Kinematic analysis

The kinematic analysis is an assessment method that evaluates structural behavior for the development of local failure mechanisms. The kinematic analysis should always be implemented before any analysis that considers the structure behaving as a whole [3, 9, 11]. Initially a local failure mechanism is considered. Then the kinematic analysis calculates the ground acceleration for which it can be activated, considering each part of the mechanism as a rigid body (composition of macro-elements). If the acceleration that activates the failure mechanism is lower than the design ground acceleration, then strengthening measures should be implemented in order to avoid local failure.

At all the floor levels there are rigid diaphragms. As a result there is a small number of failure mechanisms that could be activated. Indicative forms of failure mechanisms are presented in Figures 14-18.

The result of the kinematic analysis is that the acceleration that activates the mechanisms is even higher than the acceleration of Eurocode 8-1 [5]. To be more specific the ground acceleration according to the provisions of EC8-1 [5] is $a_g = 1.0 \cdot 0.16 = 0.16g$. While, the minimum acceleration that activates a mechanism is $a_g = 0.847g$ (Mechanism 1). Thus, no measures to avoid local failure mechanisms are needed.

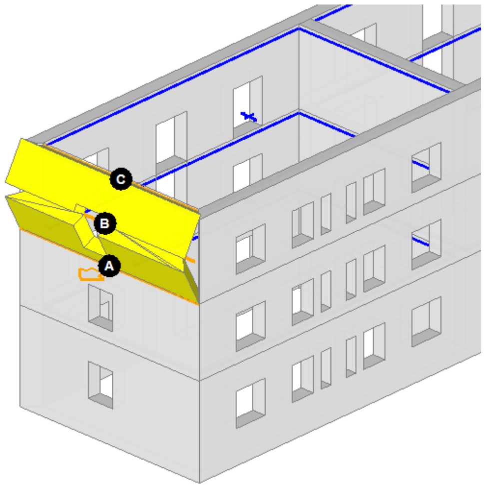

Figure 14: Mechanism 1.

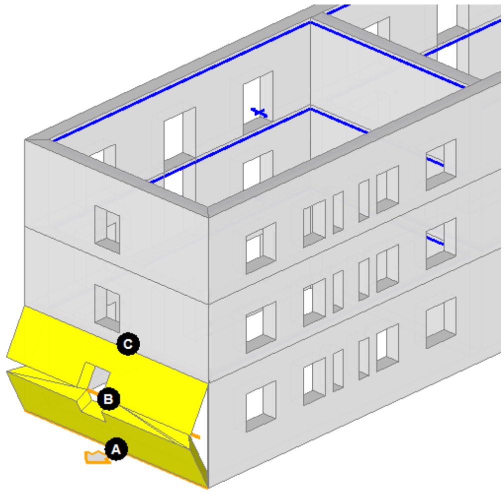

Figure 15: Mechanism 2.

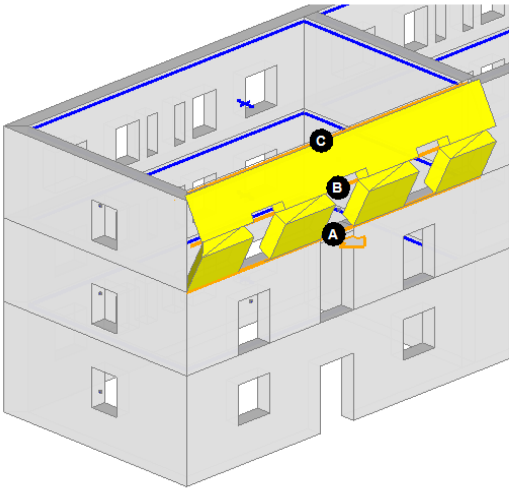

Figure 16: Mechanism 3.

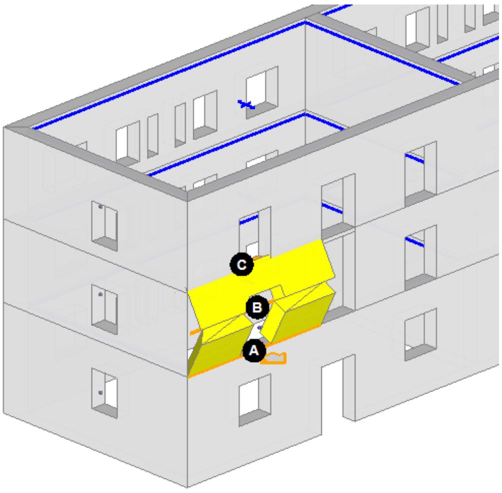

Figure 17: Mechanism 4.

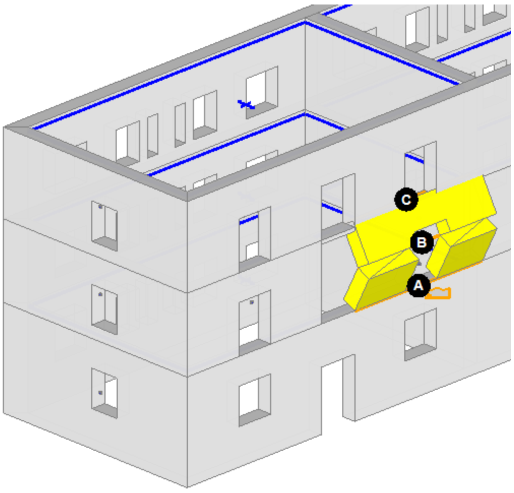

Figure 18: Mechanism 5.

3 RETROFIT AND STRENGTHENING MEASURES

Concerning the interventions, it is common practice to implement limited interventions that respect the architectural and historical characteristics of the structure, even without complying with the provisions of the current seismic code. This issue is elaborated in [12].

From the analysis, it is clear that the structure is sufficient when analyzed with seismic loads specified in the older versions of the seismic code. As a result the interventions should aim to strengthen the structure to comply as close as possible with the current code [7] with the minimum possible interventions.

The proposed interventions that would enhance the seismic behavior of the structure are the following: (1) Consolidation of the three-leaf masonry walls by injecting grout with the proper composition, (2) Jointing at both sides of the masonry. The extent of these interventions should be determined after detailed in-situ investigation of the load carrying members of the structure. Also, rehabilitation of reinforced concrete members is needed, because they have deteriorated over time [8, 13].

The behavior of the strengthened structure is examined in the following.

From the assessment of the strengthened structure for the seismic loading according to the Eurocode 8 [5] and for limit state of significant damage (SD), the capacity curve for the structure before and after applying the proposed interventions is presented in Figure 19:

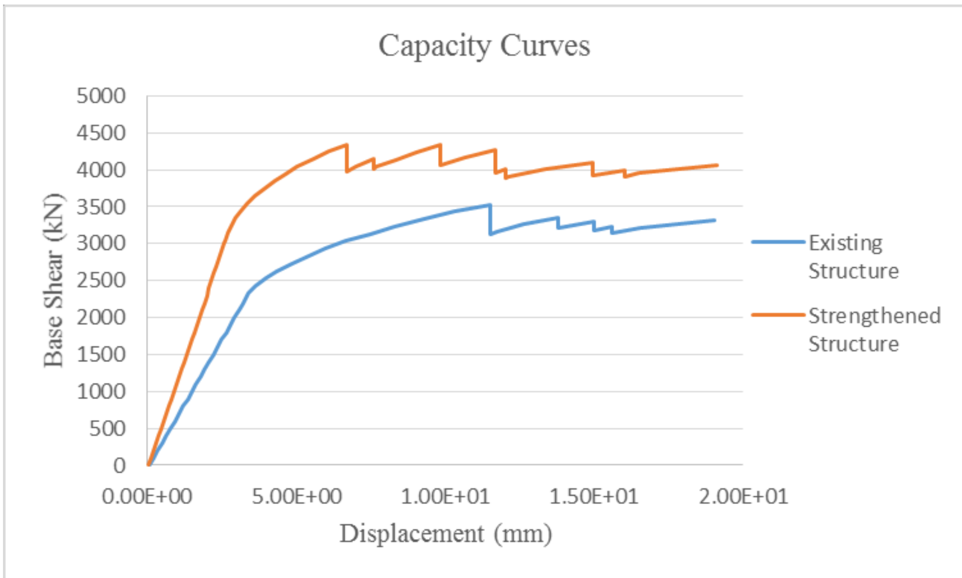

Figure 19: Indicative capacity curves before and after applying the proposed interventions for the seismic load of the combination Y+0.3X with the unimodal lateral load distribution.

Pushover analysis demonstrated that the structure is still insufficient for the seismic loading according to Eurocode 8 [5].

If further strengthening of the structure is desired, so that it can comply with the provisions of the current seismic code [5], additional strengthening measures will be required, provided that they respect the historical character of the structure [3, 12, 14].

4 CONCLUSIONS

For the assessment of the historic masonry complex “Prosfygika” in central Athens a series of in-situ tests were performed in order to estimate the mechanical characteristics of the materials of the load bearing stone masonry walls.

A series of analyses were performed: (1) Modal response spectrum analysis, (2) Pushover analysis and (3) Kinematic analysis, in order to evaluate the capacity of the existing and the strengthened structure. The results are the following:

- Assessing a masonry structure with modal response spectrum analysis underestimates its capacity.
- Pushover analysis can assess the capacity of the masonry structure more accurately, leading to limited strengthening measures.
- The historic masonry complex "Prosfygika" is sufficient for the seismic loading required by the older versions of the Greek seismic code (1959 [6] and 1985 [7]), but it is insufficient for the seismic loading required by Eurocode 8 [5].
- Further strengthening of the historic masonry structure can be achieved through strengthening measures that respect its architectural and historical characteristics.

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