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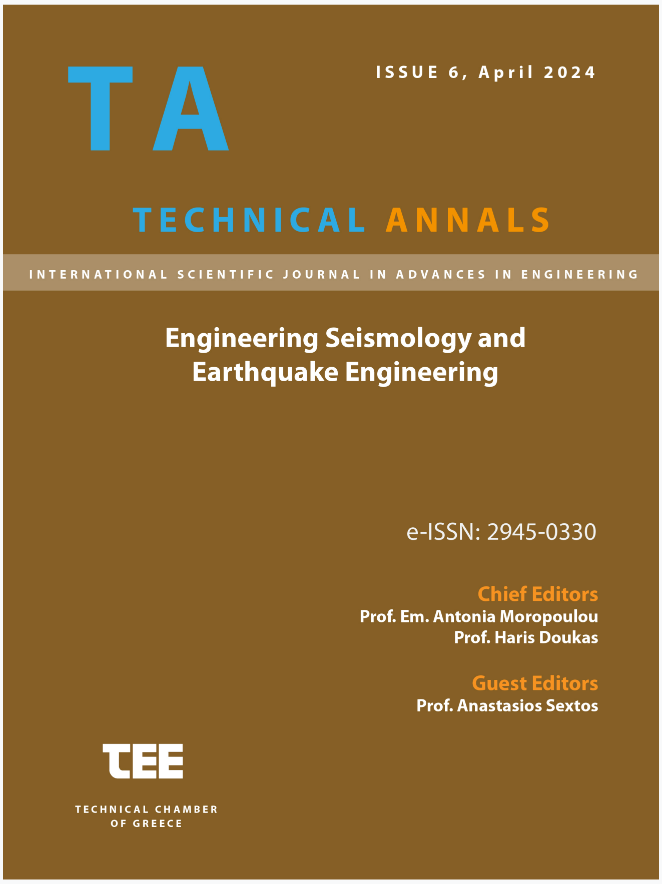

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Seismic Assessment and Proposal for Interventions of a Historical Masonry Building in Rhodes

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Abstract. The aim of this study is the seismic assessment of a historical two-storey masonry building, located in the city of Rhodes and the investigation of intervention ways to strengthen the building and improve the mechanical characteristics of its materials. The 3DR.PESSOS software was used for the simulation and analysis of the structure. For the evaluation of the seismic behaviour of the building, an elastic static analysis (lateral force analysis) was carried out based on the Greek Code for Structural Interventions of Masonry Structures. Elastic static analysis methods with global behaviour factor (q) or local ductility indices (m) were applied for performance levels B1 and B2. From the analyses and the code checks it was concluded that the building is vulnerable to in-plane and out-of-plane actions. For this reason, methods of intervention are being investigated to increase the diaphragm function of the building and to improve the mechanical characteristics of the masonry.

Keywords: Seismic Assessment, Historical Building, Unreinforced Masonry.

1 Introduction

Masonry buildings until the mid-19th century constituted the majority of the built environment not only in Greece but all over the world. Despite the fact that it was one of the oldest materials, knowledge regarding its mechanical behaviour was limited. In the 1970's, an intense research interest began, which continues to this day, mainly due to the need to preserve old structures that constitute cultural heritage monuments. It had been proven that the methods of preservation and strengthening that had been used in the past were ineffective and sometimes even dangerous [1].

The seismic codes currently in force for the assessment and retrofitting of masonry buildings are Eurocode 8 - Part 3 [2] and the more recent Greek Code for Structural Interventions of Masonry Structures [3], which has been in force since 2023. The purpose of the Greek Code for Structural Interventions of Masonry Structures is to establish criteria for the assessment of the load-bearing capacity of existing masonry structures and their redesign after possible interventions (repairs, retrofits). Other methodologies have been proposed that lead to interventions with respect for the cultural and archaeological characteristics of the structure [4]. Examples of strengthening schemes on cultural structures or bridges are presented in [5,6] while the application of non-destructive techniques for the investigation and rehabilitation of historical masonry structures or monuments are presented in [7-9]. Papanicolaou et al. [10] experimentally investigated the effectiveness of textile reinforced mortar (TRM) as strengthening material of unreinforced masonry walls subjected to out-of-plane cyclic loading, and it was concluded that TRM jacketing provides a substantial gain in strength and deformability.

In this study, an evaluation of a historical masonry building, located in Rhodes, is carried out based on the Greek Code for Structural Interventions of Masonry Structures. Specifically, it is checked whether the minimum requirements of its load-bearing capacity are met both during the assessment and during its redesign, after the proposed interventions. Under certain conditions, the minimum load-bearing capacity requirements for the assessment and redesign of an existing structure may be reduced compared to the provisions of the current design codes for new structures.

2 Case Study

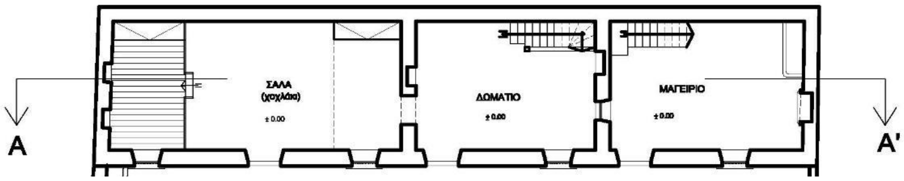
2.1 Historical data – Building location

In 1522, when Rhodes was occupied by the Ottomans, the decisive measure in forming the new living conditions for the next 400 years was the removal of the Greeks from the walled city. Thus, in order to meet the needs of the persecuted population, new residential nuclei, the "Marasia", were created. This form of "Marasias" was maintained until 1925, when the Italian buildings began to be built. The Marasias thus expanded and, with the new urban organisation, especially towards the end of the Italian period, were integrated for the first time into a single urban fabric. Despite the alterations brought by the construction activity of recent years in the city of Rhodes, the urban fabric of Marasia is still preserved, while several buildings of the late 19th and early 20th century are still standing, thanks to their designation as preserved buildings by the Ministry of Culture and the Ministry of Environment, Physical Planning and Public Works. Although no building within the urban fabric of Marasi can be considered to be earlier than the end of the 18th century, we can assume that the type of housing in Marasia was formed in the early years of the Ottoman period and was maintained unchanged with minor variations until the end of the 19th century [11].

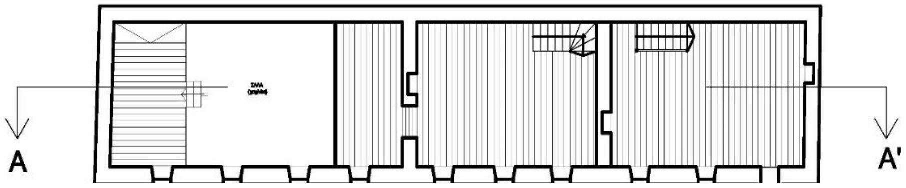
The building under consideration is located southeast of the Medieval City, within the urban plan of the city of Rhodes and specifically in the residential unit “Marasi Ag. Nikolaos”, where according to the ministerial decision of the Ministry of Culture [12] it is classified as a “Historic Site”. The building is also owned by the Archaeological Receipts and Expropriations Fund and since 1948 [13], it has been designated as a historical monument and is probably the earliest surviving example of a Marasio house.

2.2 Geometry, materials, loads

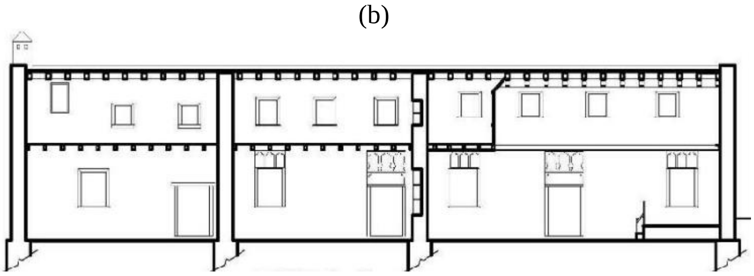
The masonry structure under investigation is constructed with two-leaf stone masonry of local Rhodesian porous stone, 50 cm and 55 cm thick. The structure is rectangular in plan, with three rooms and average external dimensions of 5.55 m wide and 24.60 m long. It consists of the ground floor, the mezzanine floor, which occupies 2/3 of the building, and the roof. The mezzanine consists of timber floorboard on wooden beams with dimensions of 10 cm x 15 cm at 50 cm spacing, while the roof consists of timber floorboard on wooden beams with dimensions of 15 cm x 20 cm at 50 cm spacing, covered with a light reinforced concrete slab. The height of the ground floor is 5.80 m and 3.25 m, while the height of the first floor is 2.55 m. Fig. 1. shows the ground floor plan and upper floor plan of the building.



(a)



(b)



(c)

Fig. 1. (a) Ground floor plan, (b) first floor plan and (c) longitudinal section A-A' [14].

For the materials, properties were taken from experimental data for similar buildings. Thus, the compressive strength of the stone f_{bc} was obtained equal to 30 MPa and the average compressive strength of the mortar f_{mc} equal to 1.0 MPa. The compressive strength of masonry f_{wc} which was calculated from Equation (1) [3, 15], was taken equal to 2.0 MPa.

$$f_{wc} = \xi\left[\left(\frac{2}{3}\sqrt{f_{bc}} - f_0\right) + \lambda\, f_{mc}\right] = 0.74\left[\left(\frac{2}{3}\sqrt{30} - 1.50\right) + 0.50 \cdot 1.0\right] = 2.0\, MPa \qquad (1)$$

where $\xi = 1 / [1 + 3.5\,(k - k_o)] = 0.74$, a coefficient which takes into account the adverse influence of the thickness of mortar joints, k is the percentage by volume of mortar in the masonry, equal to 0.40, k_o the maximum percentage of mortar considered not to cause a reduction in the strength of the wall, equal to 0.3, f_o the coefficient which takes into account the degree of carving of the stones, equal to 1.50, λ the coefficient of bonding between stone and mortar, equal to 0.5 for stone masonry.

The tensile strength of the masonry, f_{wt}, based on paragraph 6.5 of the Greek Code for Structural Interventions of Masonry Structures [3], was taken equal to 0.10 MPa and the mean characteristic initial shear strength of the masonry, f_{vk0}, based on Table 3.5 of EC6-1-1 [16], for natural stones, equal to 0.10 MPa. The self-weight of the masonry equal to 21 kN/m^3 and the material safety factor γ_m equal to 1.35 based on paragraph 4.5.3.1 of the Greek Code for Structural Interventions of Masonry Structures [3]. The modulus of elasticity E was calculated from the equation of Tasios [17], $E = \alpha f_{wc}$ =1600 MPa, where α = 800. According to EC8-1 [11] and EC8-3 [2], the stiffness is estimated as half of that for non-cracked elements, thus, the masonry modulus of elasticity was taken equal to 800 MPa. The shear modulus G was taken equal to 315 MPa and the Poisson's ratio equal to 0.30.

A live load equal to 2.0 kN/m^2 and a permanent load equal to 1.0 kN/m^2 was taken on all the slabs of the first floor and the roof. The building is located in an area with seismic hazard zone II (reference peak ground acceleration on type A ground a_{gR} = 0.24g according to EC8-1-1), soil class B (soil factor S = 1.20), importance class II (γ_I=1). For the geometric data and materials, the data reliability level was taken as "sufficient" (knowledge level *KL2*: normal knowledge according to EC8-3).

3 Numerical Analyses

The finite element method was used for the simulation of the building, where the masonry was modelled with 3319 quadrilateral shell elements in 3DR.PESSOS software [12], with a maximum dimension of 50 cm (Fig. 2.).

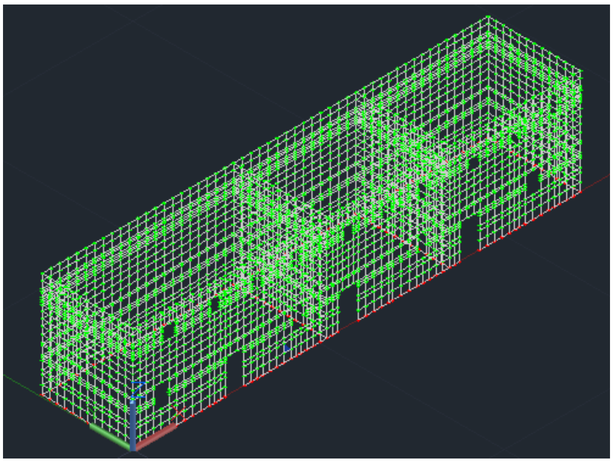
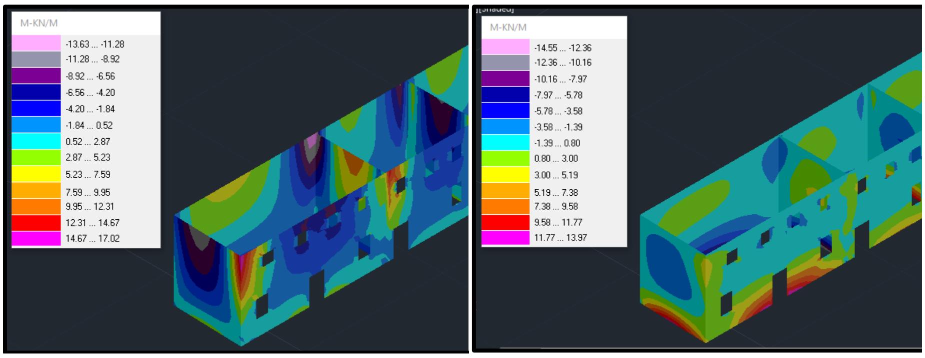


Fig. 2. 3D mathematical model of the building.

The wooden floors were not simulated as static entities in the program. They were taken into account through their self-weight, dead and live loads, so that these loads were distributed to the perimeter walls.

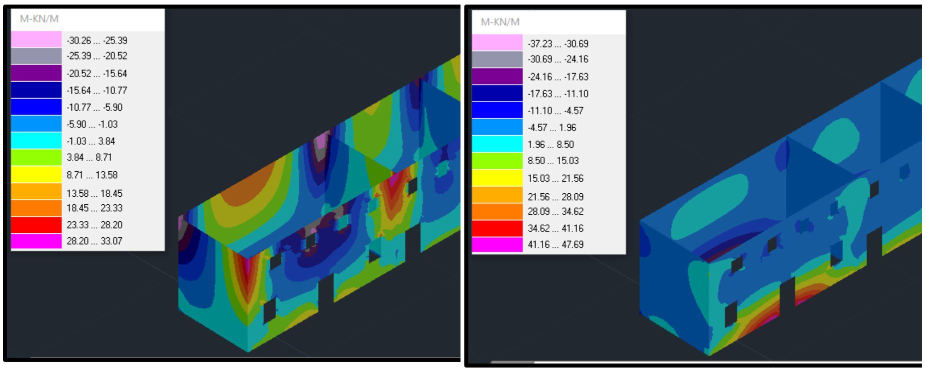
For the determination of the building's stresses and deformations, an elastic (equivalent) static analysis (lateral force analysis) was carried out. Both elastic static analysis methods with global behaviour factor (q) and local ductility indices (m) were applied for performance levels B1 and B2, according to the Greek Code for Structural Interventions of Masonry Structures [3].

Figs. 3 and 4 show the distribution of moments M_{xx} and M_{yy} for the two basic seismic combinations $G + 0.30\ Q + E_x + 0.30\ E_y$ and $G + 0.30\ Q + 0.30\ E_x + E_y$ using the method of the global behaviour factor $q = 1.50$ and for performance level B1.



(a) Bending moment distribution M_{xx}. (b) Bending moment distribution M_{yy}.

Fig. 3. Bending moment distribution for the seismic load combination $G+0.30Q+E_x+0.30E_y$.



(a) Bending moment distribution M_{xx}. (b) Bending moment distribution M_{yy}.

Fig. 1. Bending moment distribution for the seismic load combination $G+0.30Q+0.30E_x+E_y$.

It is observed that the largest values of out-of-plane bending moments about the vertical axis are developed at the connections with the transverse walls, while the largest values of out-of-plane bending moments about the horizontal axis are developed at the base of the walls.

4 Code Checks

In order to assess the seismic behaviour of the building, code checks were carried out. Elastic analysis methods were performed for the assessment. The following Tables present the results of the checks per wall, per pier and per level, with the highest value of the failure index λ. The values of the failure index are obtained from the analysis of the building using the elastic analysis method based on the global behaviour factor (q), for performance levels B1 and B2. Comparative results for each objective are presented below.

4.1 Results of elastic analysis method based on the global behaviour factor q = 1.5 and performance level B1

Table 1 presents the results of elastic analysis method based on the global behaviour factor q, for performance level B1 and q = 1.5 for level 1 (ground level) of the building, while Table 2 presents the corresponding results for performance level B2.

Table 1. Results of the code checks per wall, per pier for level 1, with the highest value of the failure index λ, *Green* (<=1.00): Adequacy, **Red** (>1.00): Inadequacy).

Wall	Pier	In-plane shear	In-plane bending	Out-of-plane bending, plane of failure parallel to the bedjoints	Out-of-plane bending, plane of failure per-pendicular to the be-djoints	Out-plane shear
		λ	*λ*	*λ*	*λ*	*λ*
1	1	**2.75**	*0.95*	**1.17**	*0.45*	**1.01**
2	2	**1.12**	*0.19*	**4.38**	*0.01*	**2.23**
3	3	**2.61**	*0.82*	**1.27**	*0.46*	**1.79**
4	4	**10.00**	**1.31**	**107.47**	*0.75*	**76.99**
	6	**5.07**	*0.76*	**2.05**	*0.66*	*0.96*
5	9	**3.01**	*0.61*	*0.82*	*0.40*	*0.61*
6	10	**1.57**	*0.07*	**2.49**	*0.23*	**3.32**
7	11	**13.85**	**1.22**	**39.86**	**3.71**	**36.51**
	13	**4.63**	*0.63*	**2.95**	**1.66**	**1.24**
	15	**3.43**	*0.57*	**3.14**	**1.33**	**1.29**
	17	**10.00**	**5.96**	**48.15**	**3.60**	**7.89**
8	18	**2.98**	*0.59*	*0.81*	*0.40*	*0.59*
9	19	**15.63**	*0.95*	**135.56**	**1.18**	**244.34**
	21	**4.33**	*0.74*	**1.51**	*0.43*	**1.59**
10	24	**1.79**	*0.29*	**4.31**	*0.30*	**3.25**

In Fig. 5a, the piers for which there is a deficiency are illustrated in red colour in the 3D model. Therefore, when evaluating the seismic behaviour of the building using the method of the global behaviour factor *q* and taking as an assessment target the performance level B1, it is concluded that the building is not sufficient and should be strengthened.

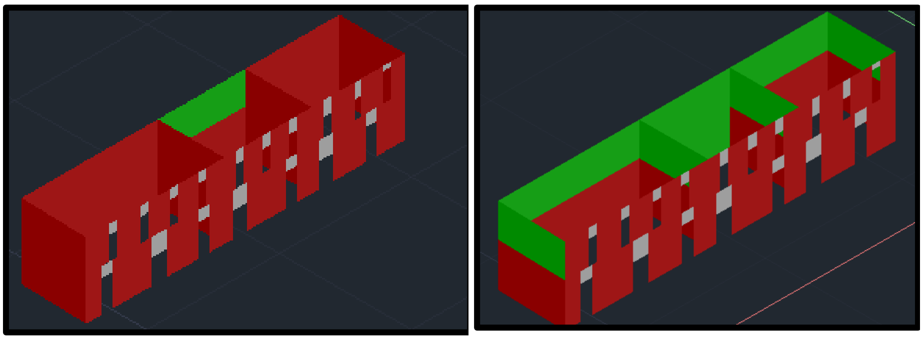
Table 2. Results of the code checks per wall, per pier for level 2, with the highest value of the failure index λ, *Green* (<=1.00): Adequacy, **Red** (>1.00): Inadequacy).

Wall	Pier	In-plane bending	Out-of-plane bending, plane of failure parallel to the bedjoints	Out-of-plane bending, plane of failure perpendicular to the bedjoints
		λ	λ	λ
1	25	*0.49*	*0.34*	**1.32**
2	26	*0.08*	*0.27*	**1.33**
3	27	*0.34*	*0.43*	**1.34**
4	28	*0.18*	*0.66*	**7.59**
	30	*0.89*	**1.02**	**5.29**
	32	*0.29*	*0.99*	**4.91**
	34	*0.89*	**1.17**	**8.83**
5	54	*0.24*	*0.27*	**1.22**
6	35	*0.04*	*0.44*	*0.99*
8	36	*0.29*	**1.97**	**8.30**
	38	*0.02*	*0.61*	**7.98**
	40	*0.67*	*0.96*	**6.92**
	42	*0.08*	*0.81*	**6.56**
	44	*0.33*	**1.52**	**6.69**
8	45	*0.21*	*0.31*	**1.22**
9	46	*0.43*	*0.77*	**4.13**
	48	*0.31*	*0.81*	**3.52**
	50	*0.57*	*0.85*	**3.17**
	52	*0.89*	**1.36**	**5.58**
10	53	*0.11*	*0.69*	**1.52**

4.2 Results of elastic analysis method based on the global behaviour factor q = 1.5 and performance level B2

The influence of the performance level on the seismic assessment results of the building was then investigated. The performance level B2 was selected and new elastic analyses were carried out based on the global behaviour factor (q) for q = 1.5. Fig. 5b illustrates in colour the piers for which there is inadequacy, which are obviously less than in the analysis for performance level B1 (Fig. 5a). By adopting a less stringent performance objective, i.e. a 50% probability of exceeding the seismic action in 50 years, on the basis of which the seismic action is reduced by about 40%, the more favourable behaviour of the masonry is evident, mainly in out-of-plane bending.

However, in this case more frequent and more extensive damage are expected for the same earthquake.



(a) Performance level B1 (b) Performance level B2

Fig. 5. Colour representation of inadequacies of the piers (green: adequacy, red: inadequacy).

5 Investigation of Interventions

The assessment of the seismic behaviour of the building according to the Greek Code for Structural Interventions of Masonry Structures [3] with the analysis method based on the global behaviour factor (q) and for the two performance levels, shows that the building has deficiencies and does not meet the required performance objective. For this reason, some interventions are being investigated which will improve the behaviour of the structure. Moreover, this is an elongated structure and the overturning check gives a failure index of 1.60, according to Equation 5.3.8a of the Greek Code for Structural Interventions of Masonry Structures [3]. In this case, it shall be ensured that the compressed parts of the walls can take up the whole of the horizontal and vertical loads.

Grout injection is an effective method for strengthening masonry walls. This technique involves low-pressure injection of fine hydraulic lime grout into cracks, voids, and cavities within the masonry, aiming to create a more homogenous structure. This method was initially investigated to increase the compressive strength of the masonry. The new compressive strength of the strengthened masonry was calculated based on equation 8.2 of the Greek Code for Structural Interventions of Masonry Structures [3],

$$f_{wc,f} = f_{wc,0} + \Delta f_0 + \lambda\, n\, f_{gc} = 2.0 + 0.75 + 0.50 \cdot 0.10 \cdot 5.0 = 3.0\ MPa \qquad (2)$$

where $f_{wc,0}$ is the initial strength of the masonry, equal to 2.0 MPa, n is the ratio of the volume of the grout to the total volume of the mortar, which, is taken to be equal to 0.10 because precise data are not available, f_{gc} is the compressive strength of the grout equal to 5.0 MPa, λ is the bond coefficient between stone unit and mortar, which is taken to be 0.50 for rough stones, f_o is the coefficient (in MPa) which takes into account the degree of carving of the stones and takes a value of 1.50-2.50 MPa for clay mortar, depending on the building quality. In this study f_o was assumed equal to 1.50 MPa, while due to the grout injections the irregularity and inhomogeneity of the building due to the degree of carving of the stones is reduced and therefore a reduction of the f_o coefficient is required according to the following equation:

$$\Delta f_0 = f_0 : \left(1 + \frac{1}{10\,n}\right) = 1.50 : (1 + 1) = 0.75 \tag{3}$$

Thus, the new compressive strength of the masonry after grout injection and homogenisation is equal to 3.0 MPa. The increase in shear strength of masonry depends strongly on the composition, application technique and consumption of grout [3]. In the present study, a 10% increase in strength was considered, therefore the shear strength in the absence of vertical load (cohesion), f_{wv0}, of the strengthened masonry was taken to be equal to 0.11 MPa.

The behaviour of the structure against earthquake can also be improved by strengthening the stiffness of the building's diaphragm. The construction of a perimeter ring beam and a reinforced concrete slab at the roof is investigated. The installation of a second additional floorboard in the mezzanine, oriented perpendicular to that of the existing floorboard, is being considered. In the analysis of the strengthened structure, according to the Greek Code for Structural Interventions of Masonry Structures, the behaviour factor (q) can be taken equal to 2.0.

The results of the checks of the strengthened building, for a behaviour factor q = 2.0 and performance levels B1 and B2, are presented in the Tables and Figures below:

5.1 Results of elastic analysis method based on the global behaviour factor q' = 2.0 and performance level B1 for the strengthened building

Elastic analyses were carried out on the strengthened building for performance level B1. Table 3 shows the results of elastic analysis for in-plane action for level 1 and Table 4 for out-of-plane action. Table 5 shows the results of elastic analysis for out-of-plane action for level 2.

Table 3. Comparison of failure indices λ of piers for level 1, for in-plane action before and after interventions.

Level 1		In-plane shear		In-plane bending	
		Before	After	Before	After
Wall	Pier	λ	λ	λ	λ
1	1	**2.75**	**2.37**	*0.95*	*0.70*
2	2	**1.12**	*0.86*	*0.19*	*0.19*
3	3	**2.61**	**2.15**	*0.82*	*0.63*
4	4	**10.00**	**3.67**	**1.31**	*0.51*
	6	**5.07**	**3.16**	*0.76*	*0.43*
5	9	**3.01**	**1.96**	*0.61*	*0.39*
6	10	**1.57**	**1.12**	*0.07*	*0.06*
7	11	**13.85**	**9.21**	**1.22**	*0.58*
	13	**4.63**	**2.92**	*0.63*	*0.47*
	15	**3.43**	**2.37**	*0.57*	*0.39*
	17	**10.00**	**7.25**	**5.96**	*0.71*
8	18	**2.98**	**2.02**	*0.59*	*0.39*
9	19	**15.63**	**8.82**	*0.95*	**0.53**
	21	**4.33**	**2.73**	*0.74*	**0.45**
10	24	**1.79**	**1.33**	*0.29*	**0.24**

Table 4. Comparison of failure indices λ of piers for level 1, for out-of-plane action before and after interventions.

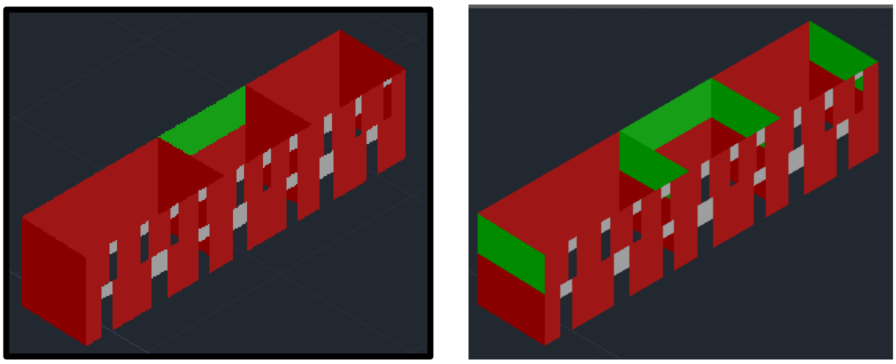
Level 1		Out-of-plane bending, plane of failure parallel to the bedjoints		Out-of-plane bending, plane of failure perpendicular to the bedjoints		Out-of-plane shear	
		Before	After	Before	After	Before	After
Wall	Pier	λ	λ	λ	λ	λ	λ
1	1	**1.17**	*0.66*	*0.45*	*0.26*	*1.01*	*0.87*
2	2	**4.38**	*1.89*	*0.01*	*0.03*	**2.23**	**2.34**
3	3	**1.27**	*0.65*	*0.46*	*0.29*	**1.79**	*0.96*
4	4	**107.47**	**4.66**	*0.75*	*0.27*	**76.99**	**7.09**
	6	**2.05**	**1.24**	*0.66*	*0.23*	*0.96*	**1.36**
5	9	*0.82*	*0.56*	*0.40*	*0.25*	*0.61*	*0.91*
6	10	**2.49**	**1.19**	*0.23*	*0.14*	**3.32**	**1.62**
7	11	**39.86**	**5.26**	**3.71**	**1.38**	**36.51**	**9.17**
	13	**2.95**	**1.27**	**1.66**	*0.62*	**1.24**	**1.36**
	15	**3.14**	**1.42**	**1.33**	*0.49*	**1.29**	**1.51**
	17	**48.15**	**23.32**	**3.60**	**1.34**	**7.89**	**25.31**
8	18	*0.81*	*0.56*	*0.40*	*0.26*	*0.59*	*0.92*
9	19	**135.56**	**2.36**	**1.18**	*0.62*	**244.34**	**4.03**
	21	**1.51**	*0.95*	*0.43*	*0.23*	**1.59**	**1.07**
10	24	**4.31**	**2.18**	*0.30*	*0.15*	**3.25**	**2.98**

Table 5. Comparison of failure indices λ of piers for level 2, for out-of-plane action before and after interventions.

Level 2		Out-of-plane bending, plane of failure parallel to the bedjoints		Out-of-plane bending, plane of failure perpendicular to the bedjoints	
		Before	After	Before	After
Wall	Pier	λ	λ	λ	λ
1	25	*0.34*	*0.52*	**1.32**	*0.37*
2	26	*0.27*	**1.56**	**1.33**	*0.20*
3	27	*0.43*	*0.58*	**1.34**	*0.39*
4	28	*0.66*	**1.89**	**7.59**	**1.57**
	30	**1.02**	**1.62**	**5.29**	**1.09**
	32	*0.99*	**1.65**	**4.91**	**1.01**
	34	**1.17**	*0.25*	**8.83**	**1.83**
5	54	*0.27*	*0.39*	**1.22**	*0.37*
6	35	*0.44*	*0.78*	*0.99*	*0.31*
7	36	**1.97**	*0.39*	**8.30**	**1.16**
	38	0.61	**1.54**	**7.98**	**1.11**
	40	0.96	**1.88**	**6.92**	*0.96*
	42	0.81	**1.93**	**6.56**	*0.91*
	44	**1.52**	**1.18**	**6.69**	*0.93*
8	45	*0.31*	*0.40*	**1.22**	*0.39*
9	46	*0.77*	*0.19*	**4.13**	**1.12**
	48	*0.81*	**1.10**	**3.52**	*0.95*
	50	*0.85*	**1.15**	**3.17**	*0.86*
	52	**1.36**	*0.11*	**5.58**	**1.51**
10	53	*0.69*	**1.36**	**1.52**	*0.35*

Fig. 6 illustrates in red colour the piers that fail in the original and the strengthened structure, respectively, for performance level B1. From Tables 5 to 7 and Fig. 6, it is observed that the failure indices for in-plane shear action for most of the piers have been significantly reduced after the interventions, however, shear failures of masonry remain. The same conclusion is reached for the out-of-plane action.

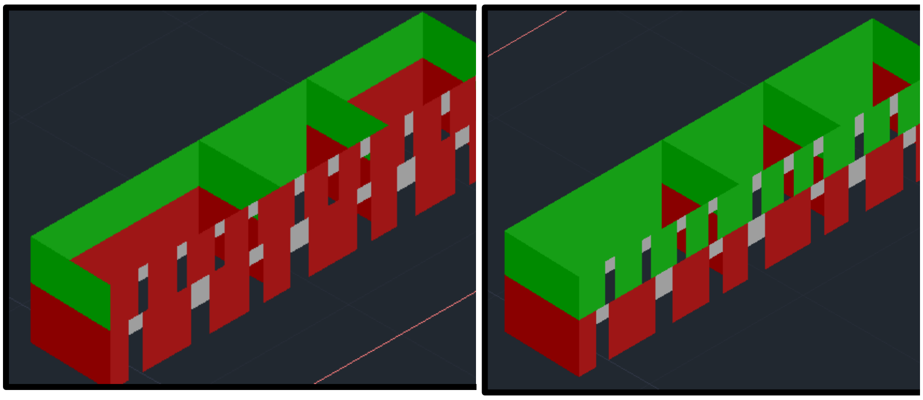
Significantly improved behaviour of the masonry appears mainly in the crown of the walls and in particular in out-of-plane bending for plane of failure perpendicular to the bedjoints. This can be attributed to the strengthening of the diaphragm function of the roof. Overall, despite the significant improvement observed in the failure indices, the building still exhibits deficiencies.



(a) Initial structure, q = 1.5 0 and performance level B1.

(b) Strengthened structure, q = 2.0 and performance level B1.

Fig. 6. Colour representation of failed piers (green: adequacy, red: inadequacy).



(a) Initial structure, q = 1.5 0 and performance level B2.

(b) Strengthened structure, q = 2.0 and performance level B2.

Fig. 7. Colour representation of failed piers (green: adequacy, red: inadequacy).

5.2 Results of elastic analysis method based on the global behaviour factor q' = 2.0 and performance level B2 for the strengthened building

As with the original building, the behaviour of the strengthened building was investigated for performance level B2, i.e. for an earthquake with a 50% probability of exceedance in 50 years. In Fig. 7, the piers that fail in the original and the strengthened structure, respectively, for performance level B2 are shown in colour in the 3D model.

As shown by the analysis of the strengthened building, for performance level B2, failure occurs only for in-plane shear actions, in the face piers and in the piers along the transverse direction (short side) of the building, only for the ground floor level. The favourable function of the diaphragm at the crown of the building is evident, where no pier shows deficiency in out-of-plane bending.

In conclusion, the method of homogenising the masonry mass with grout injections and increasing its compressive strength, in terms of in-plane shear action, does not achieve adequacy in all the piers of the structure, however, the seismic behaviour of the masonry is clearly improved. At the same time the contribution of the diaphragm at the crown level is decisive in out-of-plane flexural failure.

The strengthened structure is still inadequate regarding in-plane shear, even if grout injection and the diaphragm insertion improved the behaviour of the structure. Other methods that could be applied for strengthening of the structure could be the application of a layer of shotcrete with added reinforcement mesh to the walls which would increase both out-of-plane and in-plane strength, the application of fibre reinforced polymer sheets to the walls, or the application of fibre reinforced mortar jacketing. These methods would significantly increase the shear capacity of the walls, however, since the examined structure is a listed building, all of these methods should be approved and take listed building consent for repair works from the appropriate authorities.

6 Conclusions

In the present study, the seismic behaviour of a historical two-storey masonry building, located in the city of Rhodes, was evaluated. Subsequently, interventions were proposed to strengthen the structure and improve the mechanical characteristics of its materials. An elastic equivalent static analysis was carried out based on the Greek Code for Structural Interventions of Masonry Structures and the elastic dynamic analysis methods of global behaviour factor (q) and local ductility indices (m) were applied for performance levels B1 and B2. The analyses lead to the following conclusions:

- The original building shows significant deficiencies in both in-plane and out-of-plane action. The piers with deficiencies are significantly reduced when performance level B2 is chosen over B1. When performance level B1 is selected, only 3% of the piers are sufficient, whereas when performance level B2 is selected, the percentage of sufficient piers reaches 23%.
- Due to the inadequacy of the original structure, methods of interventions are being investigated in order to improve the diaphragm function of the building and upgrade the mechanical characteristics of masonry. The diaphragms of both the mezzanine and the roof of the building are strengthened, while the method of homogenising the masonry mass with grout injections is used to increase its compressive strength. The application of injections led to an increase in the compressive strength of the masonry by 50%, while also ensuring better bonding between mortar and natural stones.
- In the strengthened structure, in terms of in-plane shear, adequacy is not achieved in all the piers of the structure, but the seismic behaviour of the masonry is clearly improved. At the same time, the contribution of the diaphragm at the crown level is decisive in out-of-plane flexural failure.

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Annex A – Detailed Calculations

Detailed calculations in vertical loads, in-plane and out-of-plane bending moment and shear check of piers 6 and 32 are presented below. Piers 6 and 32 are highlighted in yellow in Fig. 9.

A.1 Vertical loads check

The maximum normal stress is calculated from the equation $\sigma_c = N_{sdmax}/A_w$, where $N_{sd,max}$ is the maximum axial force of each pier, resulting from the analysis of the building for the load combination 1.35G+1.50Q, and A_w is the area of the pier at the control level. Pier 6 (Fig. 8) has length, L=2.425m, height H=3.20m and width t = 0.55m.

For load combination 1.35G+1.50Q the maximum axial load is N_{sdmax} = 261.95kN, so the maximum normal stress is: $\sigma_c = N_{sdmax}/A_w$,= 196.40 kPa,

The mean compressive strength of the masonry is f_{mc}= 2.0 MPa and the safety factor γ_w = 1.35 for normal knowledge level (Greek Code for Structural Interventions of Masonry, 2021, §4.5.3.1). Therefore, the design compressive strength is:
$f_d = f_{mc}/\gamma_w$ = 2 MPa/1.35 = 1481.48 kPa

The failure index is $\lambda = \sigma_c / f_d$ = 0.13 < 1, meaning that the pier strength is adequate.

A.2 In-plane checks

The in-plane shear check, according to the global behaviour factor method and for load combination G+0.30Q+Ex+0.30Ey, is presented in detail for pier 6 (Fig. 8).
The results at the bottom of Pier 6, from the lateral force analysis (linear) of the structure using the software 3DR.PESSOS 2022, for the q factor approach and for the load combination *G+0.30Q+Ex+0.30Ey*, are the following:

F_x (V_{sd}) = 99.43 kN,
F_y (N_{sd}) = 135.91 kN,
F_z = 10.11 kN,
M_x ($M_{sd,x}$) = 14.32 kNm,
M_y ($M_{sd,y}$) = 1.63 kNm
M_z ($M_{sd,z}$) = 79.29 kNm

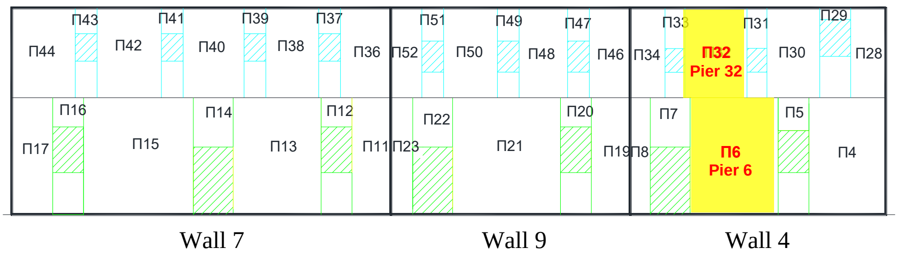

Fig. 8. Numerical representation of piers, for levels 1 and 2.

Axial force and bending moment check

The flexural design strength of the pier, considering an inactive area, is
$M_{Rd} = N_{sd}\ (1 - 1.15v_{sd})\ L/2 = 151.76\ kNm$

The acting bending moment at the bottom of the pier (from analysis) is $M_{sd,z}$ = 79.29kNm.

Therfore, the failure index is $\lambda = M_{sd,z} / M_{Rd} = 0.52 < 1$, meaning that the pier strength is adequate.

Shear check

Capacity design

The shear strength according to the capacity design rule for Pier 6 in Wall 4, level 1, is calculated according to the following equation:

$$V_f = \frac{L\ N}{2H_0}(1 - 1.15v_{sd}) = \frac{2.425\ m\ 135.91\ kN}{2 \cdot 5.47m}(1 - 1.15 \cdot 0.0687) = 27.73\ kN$$

where H_o= 5.47 m is the shear length, that is the length between the two sections where the bending moment is maximum and zero respectively,

L= 2.425m is the length of the pier,
t= 0.55m is the width of the pier,
$N_{sd(G+0.30Q)}$= -135.91 kN is the acting axial load and
$v_{sd} = N_{sd} /(L \cdot t \cdot f_d) = 0.0687$ is the normalised axial load.

Shear strength

The in-plane shear strength of the wall is the minimum of the following two mechanisms for shear failure:

a) Due to diagonal tensile cracking, according to the Greek Code for Structural Interventions of Masonry, 2021, §7.2.2i

$$f_{vd,t} = \sqrt{f_{wtd} \cdot (f_{wtd} + v_{sd} \cdot f_d)} = 142.09\ kPa$$

where $f_{vd,t}$ is the shear strength of the masonry associated with diagonal tensile cracking and f_{wtd} = 100 kN/m^2 is the mean tensile strength of the masonry.

b) Due to horizontal joint slipping, according to the Greek Code for Structural Interventions of Masonry, 2021, §7.2.2ii

The average shear strength of the masonry, $f_{vd,s}$, which takes into account the presence of the vertical load is:

$$f_{vd,s} = f_{vm0} + 0.4\frac{N_{sd}}{L´t} = 135.69\ kPa \leq 0.065\ f_b = 1950\ kPa$$

where f_{vmo} = 100 kN/m^2 is the shear strength of masonry in case of absence of vertical loads for natural carved stones,

L' = 1.89 m is the length of the compressive area of the pier,
f_b = 30 MPa is the normalised compressive strength of the stone unit, according to EN 1996-1-1.

Therefore, the in-plane shear strength of the wall is the minimum of the two:

$f_{vd} = min(f_{vd,t}, f_{vd,s}) = 135.69\ kPa$

and the shear strength of the pier is

$V_v = f_{vd} \cdot L' \cdot t = 140.90\ kN$

Final shear check

The design shear force V_{sd} shall be compared with the minimum of the values of V_v and V_f. If $V_v \leq V_f$, it is assumed that the shear force is critical for wall failure and the wall is controlled by shear, otherwise it is assumed that the moment is critical and the wall is controlled by bending (Greek Code for Structural Interventions of Masonry, 2021, §7.2.3). Because $V_v > V_f$, the bending moment is critical for failure of the pier by elastic forces. Therefore, the final shear capacity of the pier is:
$V_{Rd} = \min(V_f, V_V) = 27.73$ kN,

The acting shear force at the base of the pier is V_{sd} = 99.43 kN, according to the results from analysis.
Therefore, the failure index is $\lambda = V_{sd} / V_{Rd} = 3.59 > 1$, that is the capacity of the pier is inadequate.

A.3 Out-of-plane actions checks

The out-of-plane check, according to the global behaviour factor method and for load combination *G+0.30Q+0.30Ex+Ey*, is presented in detail for Pier 32 of level 2 (Fig. 8). The length of Pier 32 is L=1.71 m, its height is H= 2.46 m and its width is t = 0.55m. The results at the bottom of Pier 32, from the lateral force analysis (linear) of the structure, for the q factor approach and for the load combination *G+0.30Q+0.30Ex+Ey*, are the following

F_x (V_{sd}) = 0.079 kN,
F_y (N_{sd}) = 35.22 kN,
F_z = 1.34 kN,
M_x ($M_{sd,x}$) = 5.61 kNm,
M_y ($M_{sd,y}$) = 25.98 kNm
M_z ($M_{sd,z}$) = 2.67 kNm

Out-of-plane bending, plane of failure parallel to the bedjoints (Greek Code for Structural Interventions of Masonry, §7.3, Eq.7.6a)

The bending moment capacity of the cross-section is

$$M_{Rd1,o} = \frac{1}{2}\ell t_w^2 \sigma_o \left(1 - \frac{\sigma_o}{f_d}\right) = 9.36\ kNm$$

where $\ell = L$= 1.71 m is the length of the pier,
t_w = 0.55 m is the width of the pier,
N_{sd} = -35.22 kN is the axial load,
$\sigma_o = N_{sd} /(\ell \cdot t_w)$ = 37.44 kN/m^2 is the mean compressive stress due to axial load at the cross section and
$M_{sd,x}$=5.61kNm is the acting bending moment.

Therefore, the failure index is $\lambda = M_{sd,x} / M_{Rd1,o}$ = 0.60 < 1, meaning that the pier strength is adequate.

Out-of-plane bending, plane of failure perpendicular to the bedjoints (Greek Code for Structural Interventions of Masonry, §7.3, Eq.7.6b)

The bending moment capacity of the cross-section is

$$M_{Rd2,o} = \frac{1}{6} f_{wt,d} t^2 \ell = 6.38\ kNm$$

where $f_{wt,d} = f_{wt}/\gamma_w$ = 100 kN/m^2/ 1.35 = 74.07 kN/m^2 is the tensile strength of the wall and γ_w = 1.35 is the safety factor for normal knowledge level. The bending moment about the vertical axis at the base of the pier, from the elastic analysis, is $M_{sd,y}$ = 25.98 kNm.

Therefore, the failure index is $\lambda = M_{sd,y} / M_{Rd2,o}$ = 4.07 > 1, meaning that the capacity of the pier is not adequate.