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SEISMIC ASSESSMENT OF MASONRY BUILDINGS ACCORDING TO EUROCODE 6 AND 8

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ABSTRACT

It is clear from several analyses that the existing building stock has a lower seismic capacity, in comparison with the buildings designed according to new codes. The problem is more serious for masonry buildings, especially those that have been constructed in earlier years.

Despite the fact that the design code framework is clear through Eurocode 6 (2005) and Eurocode 8 (2005), the method of assessment of existing masonry structures is not obvious. It is worth to mention that Eurocode 6 (2005) is provided only for design of new masonry buildings not considering seismic actions.

This paper deals with a comparison of seismic assessment methods of masonry buildings according to either the framework of Eurocode 6 (2005) or Eurocode 8 Part 3 (2005b) and an approximate method proposed by the Greek Earthquake Planning and Protection Organizations (EPPO, 2012). These methods are applied to selected buildings with particular characteristics, through a parametric study aiming to emphasise possible different results when assessing the seismic behaviour of a structure. From the results of the present analysis, it was found that the extension of the implementation of assessment procedures of Eurocode 6 (2005) to buildings that are subjected to seismic actions, produces results that are very different to those that are given by the implementation of Eurocode 8 (2005b), when the criteria for the safety verification of the building are expressed in terms of deformations and storey drifts. Moreover Eurocode 8 (2005b) does not consider out of plane deformation of the walls while in the framework of Eurocode 6 (2005) in-plane and out-of plane action effects are simultaneously considered.

In the case studies investigated in the present work Eurocode 8 (2005b) results the highest failure indices in higher-level stories. On the other hand, Eurocode 6 (2005) results similar values of failure indices from storey to storey. When comparing the Greek Earthquake Planning and Protection method (EPPO, 2012) with Eurocodes 6 (2005) and Eurocode 8 (2005b), it appears that λ_b values for the EPPO (2012) method is always higher than unity even in the cases of simple buildings where respective values are expected by definition much lower than unity and the rigorous method of assessment applied in the framework of Eurocode 8 (2005b) justified these low λ_b values. Therefore a correction factor β multiplying the resistance index R of the EPPO(2012) method is proposed in the present work. Furthermore, it was found that the results of the EPPO(2012) method is not much influenced by the number of stories which was found to be in contrast with the results of the rigorous analytical method that was used for the same buildings.

INTRODUCTION

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This paper deals with a comparison of seismic assessment methods of masonry buildings according to either the framework of Eurocode 6 (2005) or Eurocode 8 Part 3 (2005b) and an approximate method proposed by the Greek Earthquake Planning and Protection Organizations (EPPO, 2012). It is worth to mention that Eurocode 6 (2005) is provided only for design of new masonry buildings not considering seismic actions. Moreover Eurocode 8 (2005b) does not consider out of plane deformation of the walls while in the framework of Eurocode 6 (2005) in-plane and out-of plane action effects are simultaneously considered. These methods are applied to selected buildings with particular characteristics, through a parametric study aiming to emphasise possible different results when assessing the seismic behaviour of a structure.

At first, the investigation considers a six-storey building for which the failure indices according to Eurocode 6 (2005), Eurocode 8 (2005b) and the approximate method are computed and compared. It is considered as necessary the expansion of the sample of representative buildings that are analyzed, so the failure indices for "simple" masonry buildings (as defined in Part 1 of Eurocode 8,2005a) are computed and some very important conclusions are provided from the comparison of the three methods in such buildings. Moreover, it is considered as crucial the investigation of the influence of the number of stories on the failure indices of particular buildings. For this purpose, a differentiation of the six-storey building to four-storey and two-storey building takes place, so that the failure index λ_b resulted from the different methods should be compared.

MAIN CHARACTERISTICS OF MULTI-STOREY BUILDING

The six storey building that was analysed is located at the historical centre of Corfu in Greece and was constructed during the period of the English "protection" of the Ionian Islands (1815-1864). Figure 1 presents a plan of the ground floor with the walls designated and a cross section through the six storey building.

The materials that were used for the construction of this building are typical of that time. The walls of the basement and the ground floor storeys consist of three-leaf stone masonry in contrast with the rest of the storeys that consist of compact brickwork of a reduced thickness. The thickness of the walls of the basement and the ground floor are 80 cm and thereafter gradually reduce to 35 cm in 5[th] floor of the building.

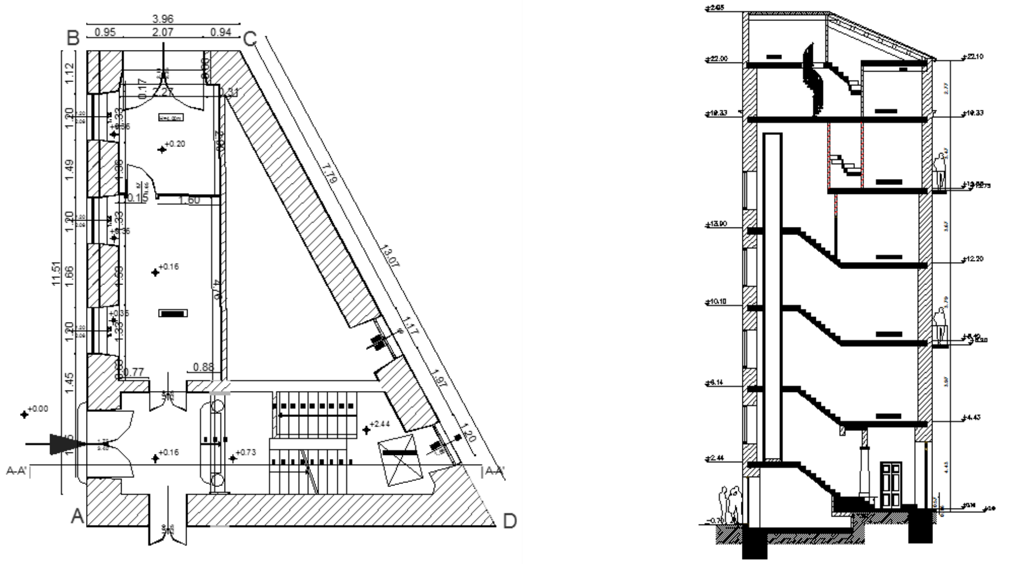

Figure 1 Characteristic plan and section of investigated 6-storey building

As far as the floors are concerned, these consist of wood and are supported by wooden beams (section 20x18 cm) at a distance of 40 to 60 cm, on which the flooring (with a thickness of 1.5 cm) is situated. The roof consists of a wooden truss arrangement and is covered by tiles.

MECHANICAL PROPERTIES OF MASONRY

The mechanical properties of the materials were either determined from laboratory tests or with reference to suitable references. The compressive strength for the three-leaf masonry was 10.9 MPa, while that of the brickwork was 4.47 MPa. The shear strength in both cases was 0.07 MPa. The elastic modulus for both materials was 6516 MPa and 2682 MPa respectively and the shear modulus was 2470 MPa and 1032 MPa respectively.

LOADING

The self weight of the two types of masonry was chosen as 21 kN/m^2 for three leaf masonry and 18 kN/m^2 for brickwork. The building has wooden floors and roof, which do not offer a significant diaphragm action. Therefore, it was decided that only the self weight and live load loads specified by the design code were be taken into account. As a result, dead loads of 1.30 kPa for the roof, 1.20 kPa for the floors and staircase and 0.80 kPa for internal walls were input in the analysis. The live load of the floors and staircase was considered as 2.20 kPa.

ANALYSIS ASSUMPTIONS

The building was modelled using the ETABS 9.5.0 (2009) computer program. The walls were simulated using four-joint shell finite elements and were different from floor to floor according to the wall thickness. These shell elements is considered to have bending (out of plane) and membrane (in plane) stiffness. The foundation of the building is considered pinned. A modal analysis was implemented. The analysis was a linear elastic multi-modal response spectrum analysis assuming a reduced stiffness for the cracked section (Eurocode 8, 2005b). The building that was analysed is located in Corfu, so the seismic zone is II with an importance factor of class II, as it is normal building for dwelling or offices. The soil category is C, while the behaviour factor q is considered 1.50 and the foundation factor is 1.00. The building is checked for seismic loading according to Greek Seismic Code (EAK2000, 2003) for the dead and live load combination of 1.35G+1.5Q as well as for seismic load combinations of G+0.3Q±Ex±0.3Ey and G+0.3Q±0.3Ex±Ey**.**

FAILURE INDICES THROUGH EUROCODE 6

Through ECTools (2002) software, the sections of piers and spandrels of the structure were checked for bending and shear. ECTools (2002) uses Eurocode 6 (2005) equations and failure indices in bending and shear can be determined. Failure indices (λ_i) of elements are determined by dividing action effects by capacity. Therefore they are unitless and values greater than unity indicate that damage would occur. Obviously, the greater the number is above unity, the greater the expected damage. According to Eurocode 6 (2005) λ_i in flexure with axial load is defined by the following equation:

$$\lambda_i = \frac{N_{sd}}{N_{Rd}} \tag{1}$$

where:
N_{sd} is the design vertical load on a masonry section
N_{Rd} is the design vertical load resistance of the masonry section and is given by the following equation:

$$N_{Rd} = \Phi i * t * fd \tag{2}$$

where:
Φ_i is the capacity reduction factor allowing for the effects of slenderness and eccentricity of loading
t is the thickness of the section
f_d is the design compressive strength of the masonry

As far as shear in Eurocode 6 (2005) is concerned, λ is defined by the following equation:

$$\lambda = \frac{V_{sd}}{V_{Rd}} \tag{3}$$

where:
V_{sd} is the design value of the applied shear load
V_{Rd} is the design shear resistance and is given by the following equation:

$$V_{Rd} = f_{vd} * t * l_c \tag{4}$$

where:
f_{vd} is the design shear strength of the masonry
t is the thickness of the section
l_c length of area under compression

It was decided that an average failure index of the sections per floor for bending and shear should be calculated. The average failure index in every floor (j), was computed from Eq. (5) and Figure 2 presents the results.

$$\lambda_j = \frac{\sum_{i=1}^{6} \lambda_i * A_i}{\sum_{i=1}^{6} Ai}, \quad j = 1 \ldots 6 \tag{5}$$

where:
λ_j is the failure index of the floor
λ_i is the failure index of section in bending or shear and
A is the area of section of pier or spandrel.

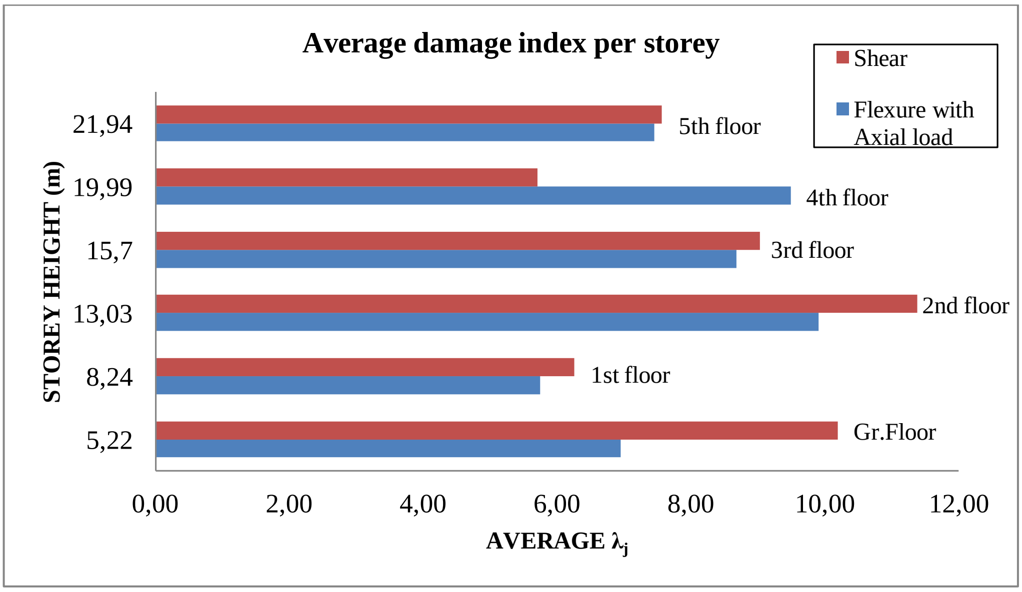

Figure 2 Average failure indices according to Eurocode 6 in flexure with axial load and shear per floor

Figure 2 shows that the failure indices in shear almost everywhere greater than those in flexure. It is also evident the fact that the values are in every storey greater than 5.00 both in flexure and shear. The maximum value of failure indices appears in 2nd storey of the building.

The failure indices of the building (λ_b) in flexure and shear, are the maximum failure indices that resulted from the above analyses and are presented in Table 1.

Table 1 Building failure indices in flexure and shear

λ_b Flexure	λ_b Shear
9.90	11.4

It can be seen from Table 1 that the failure indices in flexure and shear are considerably greater than unity and their values are very high.

Also, the failure index of the building can be considered the maximum value of flexure and shear, so λ_b = 11.4 is the failure index of the building according to Eurocode 6 (2005) and is observed in 2nd floor.

FAILURE INDICES THROUGH EUROCODE 8

In this section, a computation of failure indices through upper limit acceptance criteria of Eurocode 8 (2005b) takes place for shear and flexure with axial load respectively. The upper limit acceptance criteria of Eurocode 8 (2005b) were computed only for in-plane deformations and are the following:

For primary seismic walls in the "Significant Damage" performance level, the capacity of an unreinforced masonry wall controlled by flexure is expressed in terms of drift and taken equal to 0.008Ho/D, where Ho is the distance between the section where the flexural capacity is attained and the contraflexure point and D is the in-plane horizontal dimension of the wall (depth). The respective capacity in shear is equal to the value 0.004.

For the computation of displacements and because of the fact that the displacements that resulted are elastic, a behaviour factor q is used for their transformation from elastic to inelastic. Taking into account the Part 1 of Eurocode 8 (2005a), this factor was selected to be q = 1.50. Figure 3 presents the determined wall displacements.

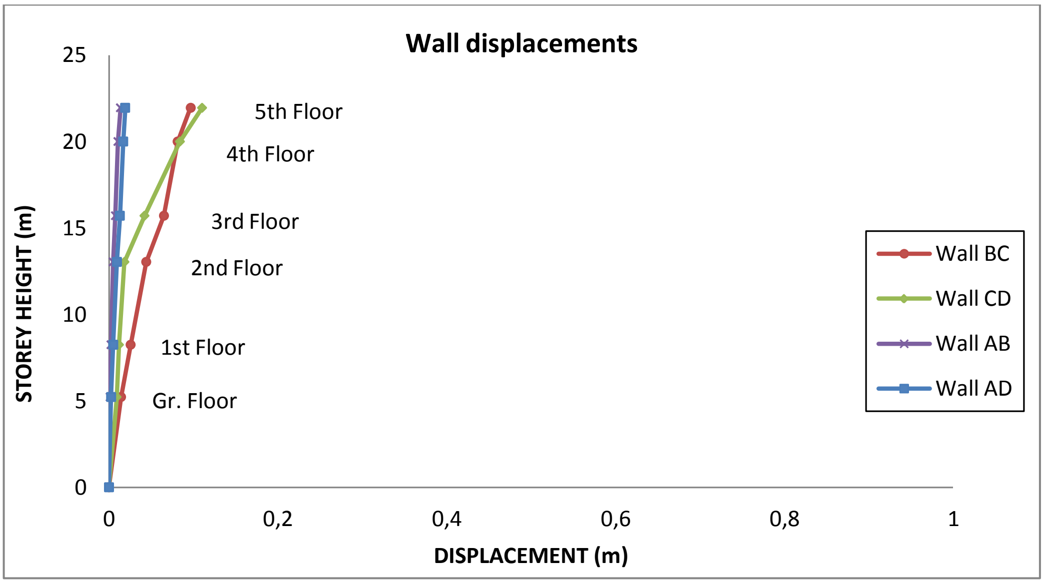

Figure 3 Wall displacements per storey

Figure 3 shows that the largest displacement occurs in wall CD, which has the biggest length. This is one of the most important reasons for which this wall has the larger displacements. Another reason could be the fact that the transverse walls are under an angle so they do not offer a high resistance to this wall. It is also observed that the gradient of the line of displacements is normal in the first stories, in contrast with higher stories in which the gradient is getting higher and especially in walls CD and BC, which have the feature that their transverse walls are not vertical to them. This absence of normality from 2nd floor causes higher drifts.

Having computed the displacements, the drift for each floor was determined through the following equation:

$$\text{drift} = \frac{ui-u(i-1)}{hi}, \qquad i = 1\ldots6 \tag{6}$$

where:
u is the in-plane displacement and
h is the floor height.

The failure indices λ_i are computed by dividing these drifts by their upper limit acceptance criteria according to Part 3 of Eurocode 8 (2005b) for every wall of the building. The upper limit acceptance criteria of Eurocode 8 (2005b) and the failure indices in flexure and shear for the "Significant Damage" performance level in the walls of the building are presented in Table 2. Failure indices exceeding unity have a red font colour.

Table 2 Failure indices λ_i through Eurocode 8 (2005b)

Wall	Storey	Displacement (m)	Drift	EC8 Flexure with Axial Load Upper Limit Acceptance Criteria	EC8 Shear Upper Limit Acceptance Criteria	λ_i Flexure with Axial Load	λ_i Shear
BC	Ground Floor	0.01428	0.00274	0.043	0.004	0.06	0.7
	1st Floor	0.02562	0.00375	0.067	0.004	0.06	0.9
	2nd Floor	0.0441	0.00587	0.106	0.004	0.06	1.5
	3rd Floor	0.0651	0.00487	0.128	0.004	0.04	1.2
	4th Floor	0.08127	0.00513	0.163	0.004	0.03	1.3
	5th floor	0.09681	0.00503	0.179	0.004	0.03	1.3
CD	Ground Floor	0.00903	0.00173	0.003	0.004	0.54	0.4
	1st Floor	0.01176	0.00090	0.005	0.004	0.18	0.2
	2nd Floor	0.01827	0.00207	0.008	0.004	0.26	0.5
	3rd Floor	0.042	0.00551	0.010	0.004	0.57	1.4
	4th Floor	0.084	0.01333	0.012	0.004	1.09	3.3
	5th floor	0.11025	0.00850	0.013	0.004	0.63	2.1
AB	Ground Floor	0.00126	0.00024	0.004	0.004	0.07	0.1
	1st Floor	0.00231	0.00035	0.006	0.004	0.06	0.1
	2nd Floor	0.00483	0.00080	0.009	0.004	0.09	0.2
	3rd Floor	0.00756	0.00063	0.011	0.004	0.06	0.2
	4th Floor	0.0105	0.00093	0.014	0.004	0.07	0.2
	5th floor	0.01344	0.00095	0.015	0.004	0.06	0.2
AD	Ground Floor	0.00252	0.00048	0.004	0.004	0.12	0.1
	1st Floor	0.00504	0.00083	0.006	0.004	0.13	0.2
	2nd Floor	0.00945	0.00140	0.010	0.004	0.14	0.4
	3rd Floor	0.01281	0.00078	0.012	0.004	0.06	0.2
	4th Floor	0.0168	0.00127	0.016	0.004	0.08	0.3
	5th floor	0.01911	0.00075	0.017	0.004	0.04	0.2

Figure 4 presents the wall drifts of the previous table.

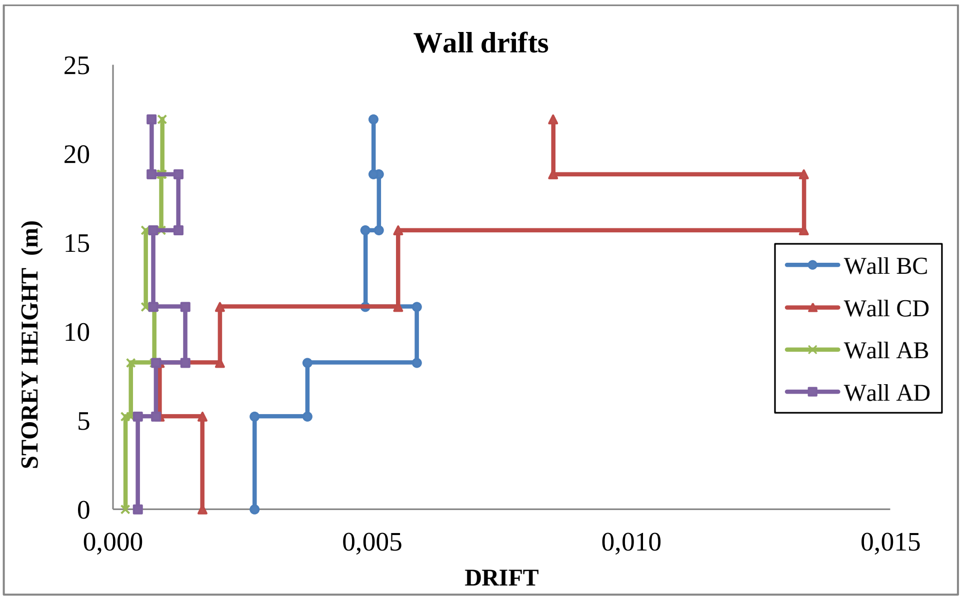

Figure 4 Wall drifts

Table 2 and Figure 4 show that the maximum value of failure index, as well as that of drift appears in wall CD. Some important failure indices appear also in wall BC in contrast with walls AB and AD that have very low failure indices. All these come from the characteristics of the walls that were referred above. The most serious problem appears in the fourth floor of the building and particularly in wall CD because of shear.

Consequently, the failure index λ_b is the maximum value λ_i that was derived from Table 2 and specifically it is the value $\lambda_b = 3.3$ which corresponds to shear in 4th storey.

EUROCODE 6 AND EUROCODE 8 COMPARISON OF FAILURE INDICES

From the above, a comparison between the failure indices of Eurocode 6 (2005) and Eurocode 8 (2005b) is feasible. To be more exact, the maximum failure index of Eurocode 6 (2005) was 11.4. As far as Eurocode 8 (2005b) is concerned, the maximum failure index was 3.3. Consequently, it can be seen that the failure index of Eurocode 8 (2005b) is much smaller than that of Eurocode 6(2005). The difference between the codes is about 245%.

This difference is due firstly to the fact that the failure indices of Eurocode 8(2005b) concern only in-plane drifts, which is in contrast with Eurocode 6 (2005) where the check concerns both in-plane and out of plane action effects. It is also clear that the failure indices of Eurocode 6 (2005) are an average for sections, so the check is much more conservative than that of Eurocode 8 (2005b), which focuses on the check of whole walls and their displacements per floor.

Another very important difference is that in Eurocode 6 (2005) the maximum value of failure indices appeared in 2nd storey, not having great differentiation from storey to storey contrary to Eurocode 8 (2005b) whose maximum value appeared in 4th storey. Consequently, it is evident that Eurocode 8 (2005b) appears the highest failure indices in high stories so there are the crucial stories for the assessment. On the other hand Eurocode 6 (2005) appears almost the same results as far as the stories are concerned.

ESTIMATION OF FAILURE INDEX OF THE BUILDING ACCORDING TO APPROXIMATE ASSESSMENT METHOD OF GREEK EARTHQUAKE PLANNING AND PROTECTION ORGANIZATION

The Greek Earthquake Planning and Protection Organization has proposed an approximate method for the assessment of the seismic capacity of masonry buildings (EPPO, 2012). This is based on a rational comparison between an index of Seismic Intensity (H) and an index of Seismic Resistance (R). The failure index of a building is defined as in Eq. (7).

$$\lambda_b = \frac{H}{R} \tag{7}$$

The seismic intensity for the building depends on the seismic action index of the building (H1) which is defined according to the seismic zone and the influence of neighbouring buildings index (H2) according to the Eq. (8).

$$H = h1*H1+h2*H2 \tag{8}$$

where h1, h2 are weighting factors which take the values 0.75 and 0.25 respectively.

The seismic resistance of the building depends on the indices R1 which is the ground floor shear resistance index, R2 which is the load bearing wall openings index, R3 which is the ring beam index, R4 which is the diaphragm index, R5 which is openings near corners index, R6 which is masonry damage index, R7 which is the connection between transverse walls index, R8 is the perimeter wall out of plane stress index, R9 which is ground floor plan regularity index and R10 which is height regularity index. The weighting factors for these indices are presented in Table 3.

Table 3 Indices of seismic resistance

Index	Name	Weighting factor (ri)
R1	Ground floor shear resistance index	0.20
R2	Load bearing wall openings index	0.05
R3	Ring beam index	0.15
R4	Diaphragm index	0.10
R5	Openings near corners index	0.15
R6	Masonry damage index	0.05
R7	Connection between transverse walls index	0.10
R8	Perimeter wall out of plane stress index	0.10
R9	Ground floor plan regularity index	0.05
R10	Height regularity index	0.05

Table 3 shows that the final seismic resistance depends on a number of indices and their corresponding weighting factor. Table 3 can be summarised by Eq. (9), as follows:

$$R = 0.2R1+0.15(R_3+R_5)+0.10(R_4+R_7+R_8)+0.05(R_2+R_6+R_9+R_{10}) \tag{9}$$

Computing the indices in the investigated 6-storey building the results are found as:

- H1 = 2.4, H2 = 1.00 $\rightarrow$ H = 2.05
- R1 = 0.142, R2 = 0.57, R3 = 0.50, R4 = 0.40, R5 = -1.00, R6 = 1.00, R7 = 1.00, R8 = 0.93, R9 = 0.50, R10 = 1.00 $\rightarrow$ R = 0.342

Consequently, the value of the failure index that results through this method is :

$$\lambda_b = H/R = 6.00$$

COMPARISON OF FAILURE INDEX λ_b OF EPPO METHOD WITH THE FAILURE INDEX λ_b OF EUROCODES 6 AND 8

According to the values of failure indices that resulted through Eurocode 6 (2005), Eurocode 8, (2005b) and the approximate seismic method (EPPO, 2012), an improvement of the failure index of the EPPO method λ_b is proposed by introducing a correction factor "β" in the form of $\lambda_b' = \frac{H}{\beta * R}$.

Obviously taking into account that λ_b values have been found equal to $\lambda_b = 11.4$ and $\lambda_b = 3.33$ according to Eurocode 6 (2005) and Eurocode 8(2005b) respectively, the correction factor β is equal to $\beta = 0.53$ and $\beta = 1.79$ respectively.

FAILURE INDICES FOR "SIMPLE" MASONRY BUILDINGS

Aiming to expand the sample of representative buildings that are analyzed, the failure indices for "simple" masonry buildings are computed. According to Part 1 of Eurocode 8 (2005a), "simple" buildings are those which belong to important classes I and II and an explicit safety verification is not mandatory for them. Consequently, according to certain rules, these buildings are considered safe without checking the integrity of them by any specific seismic analysis. Therefore, one could expect that seismic assessment of capacity of those buildings would result in failure indices quite lower than unity.

It is obvious that the parameters of configuration of a building are too numerous. It was at first decided, the consideration of concrete slabs that ensure diaphragm operation.

In order to broad the investigation of the present work, it was decided to investigate two buildings with different minimum area of sections per direction as a percentage of the area of the above floors (Fig. 5 right) and three buildings with area of section greater than the minimum (Fig. 5 left).

Figure 5 presents typical plans and includes the dimensions of piers and spandrels.

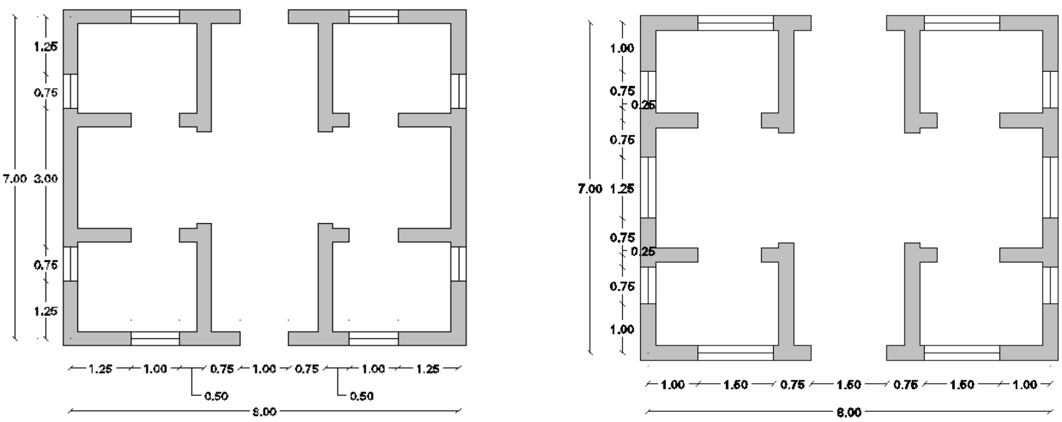

Figure 5 "Simple" building plans A, B, C (left) and D, E (right)

The main parameters of the buildings that were investigated are given in Table 4 along with the average of failure indices of Eurocode 6 (2005), the maximum values of failure indices of Eurocode 8 (2005a) and failure indices λ_b of EPPO method (EPPO, 2012).

Table 4 Main characteristics and failure indices of investigated "simple" buildings

Building	Seismic Zone	Number of Storeys	Masonry thickness (m)	Minimum pier area (%)	λ_b				
					EC6 Flexure	EC6 Shear	EC8 Flexure	EC8 Shear	EPPO
A	I	1	0.25	7.58	1.82	1.95	0.24	0.20	1.57
B	I	2	0.25	7.58	2.14	1.67	0.29	0.25	1.65
C	II	1	0.3	9.1	3.2	1.51	0.29	0.25	2.21
D	II	1	0.3	6.0	4.16	1.96	0.38	0.32	2.40
E	I	2	0.25	5.0	2.78	2.17	0.38	0.33	1.70

Table 4 shows that the factors that influence the failure indices of simple buildings are the number of storeys, the area of sections of piers as percentage of the area of above floors and the seismic zone.

According to the above section and having chosen buildings D and E that have a minimum area of piers for a particular seismic zone, a correction of failure indices λ_b EPPO can be performed so that, according to Eurocode 6 (2005) and Eurocode 8 (2005b) respectively, the maximum values of λ_b EPPO will define if a building can be determined as "simple" or not.

Following the same method, the maximum factors β derived from flexure or shear for buildings B and C can be determined, as presented in Table 5. The aim here is to compare the failure indices of buildings that have the same parameters with the exception of the percentage of piers area. Consequently, the possible β values for factors λ_b EPPO can be determined, as shown in Table 5.

Table 5 Correction factors β

Building	Seismic Zone	Number of Storeys	Masonry thickness (m)	Minimum pier area (%)	β	
					EC6	EC8
D	II	1	0.3	6	1.22	7.69
C	II	1	0.3	9.1	1.47	9.09
E	I	2	0.25	5	0.78	5.26
B	I	2	0.25	7.58	0.99	6.67

From Table 5, it is evident that the factors obtained from Eurocode 6 (2005) and Eurocode 8 (2005) are as expected and are in line with the six-storey building results, that is, there are much lower values when Eurocode 6 (2005) is compared to Eurocode 8 (2005b). Therefore, since Eurocode 6 (2005) values are far more conservative than Eurocode 8 (2005b), the H/R factors do not exceed unity. Regarding Eurocode 6 (2005), the results could be characterised as being fairly close to H/R values and, thus, the obtained correction factors are close to unity. It can also be observed that according to Eurocode's 8 (2005b) values (which indicate that "simple masonry buildings" are structurally more than adequate), H/R failure indices are too conservative for one or two storey buildings, as the derived correction factors are much higher than unity. Through the inadequacies of Eurocode 8 (2005b), it can be concluded that H/R indices should be suitably modified in order to also assume values less than unity, as in the case of "simple masonry buildings".

INFLUENCE OF NUMBER OF STOREYS ON THE FAILURE INDICES OF THE BUILDING

A differentiation of the six-storey building to four-storey and two-storey building is necessary, so that the failure λ of Eurocode 8 (2005b) and λ_b of EPPO (2012) will be computed. Taking also into account the respective results in "simple" one-storey and two-storey building a very safe conclusion as far as the influence of the number of stories on the failure indices will be feasible.

The second level pre-earthquake assessment is particularly conservative in cases of one or two storey buildings, as the relevant inadequacies are greater than unity while, according to Eurocode 8 (2005b) deficiency results, it is highly likely that there are adequacies in such buildings. The latter is evident not only from the "simple masonry buildings" inadequacies, but also from the six storey building variations (4 storey and 2 storey), where the difference when compared to the six storey deficiencies is significant. For this purpose, the failure indices of five buildings using Eurocode 8 and the approximate assessment method (EPPO, 2012) are presented in Table 6 and Figure 6.

Table 6 Failure indices of 2-storey, 4-storey & 6-storey building

Building	λ_b EC8 (Flexure & Axial Load)	λ_b EC8 (Shear)	λ_b EPPO
"Simple" 1 storey	0.24	0.20	1.57
"Simple" 2 storey	0.29	0.25	1.65
2 storey	0.48	0.60	5.20
4 storey	0.92	1.47	5.70

6 storey	1.09	3.36	6.00

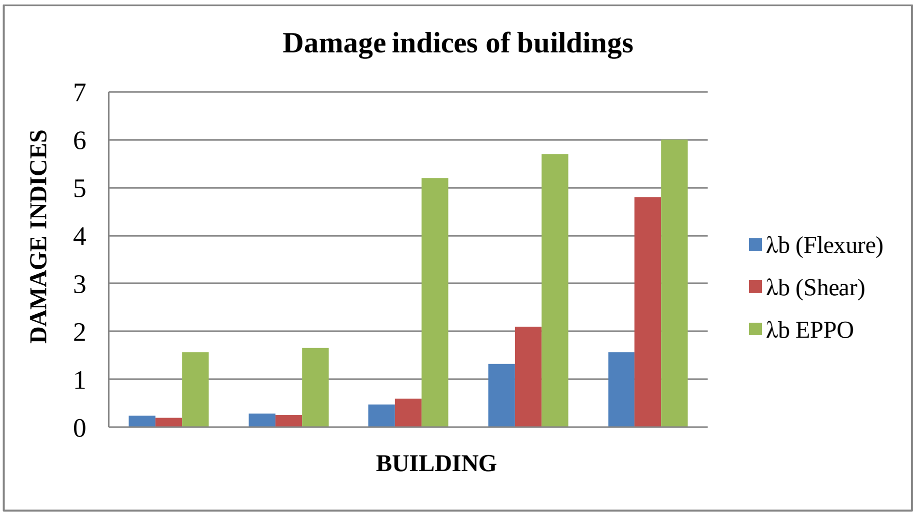

Figure 6 Failure indices in 2-storey, 4-storey and 6-storey buildings

From the above table and figure, it can be noted that the λ_b EPPO (2012) index barely changes when adding storeys until a maximum value, which is 6.00 for the 6 storey building. This is in contrast with the results of failure indices of Eurocode 6 (2005) and Eurocode 8 (2005b) that differ more and more as the number of stories grows up. On the contrary, the failure indices calculated through Eurocode 8 (2005b) steadily increase when storeys are added. From the above, it can be concluded that the R1 index (shear resistance index) of the approximate assessment method (EPPO, 2012) should be revised when the number of the storeys is taken into account but, most importantly, the influence of the number of the storeys is reflected in the other the seismic resistance indices of the building.

CONCLUSIONS

When assessing the seismic capacity of masonry buildings in earthquake regions, Eurocode 6 (2005) appears to be much more conservative when compared to Eurocode 8 (2005b), especially for one or two storey buildings and “simple masonry buildings” as these are defined in Part 1 of Eurocode 8 (2005a).

In the framework of Eurocode 6 (2005) action effects are compared with relevant resistances in terms of forces, while in the framework of Eurocode 8(2005b) are compared in terms of deformations (drifts). Moreover Eurocode 8 (2005b) does not consider out of plane deformation of the walls while in the framework of Eurocode 6 (2005) in-plane and out-of plane action effects are simultaneously considered.

In the case studies investigated in the present work Eurocode 8 (2005b) results the highest failure indices in higher-level stories. On the other hand, Eurocode 6 (2005) results similar values of failure indices from storey to storey.

When comparing the Greek Earthquake Planning and Protection method (EPPO,2012) with Eurocodes 6 and 8, it appears that λ_b values for the EPPO (2012) method is always higher than unity even in the cases of simple buildings where respective values are expected by definition much lower than unity and the rigorous method of assessment applied in the framework of Eurocode 8 (2005b) justified these low λ_b values.

Therefore a correction factor β multiplying the resistance index R of the EPPO (2012) method is proposed in the present work. Furthermore, it was found that the results of the EPPO (2012) method is not much influenced by the number of stories which was found to be in contrast with the results of the rigorous analytical method that was used for the same buildings. As a result, and through the present investigation, it was found that it is necessary that the number of stories should have greater influence in the seismic resistance of the building (R).

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