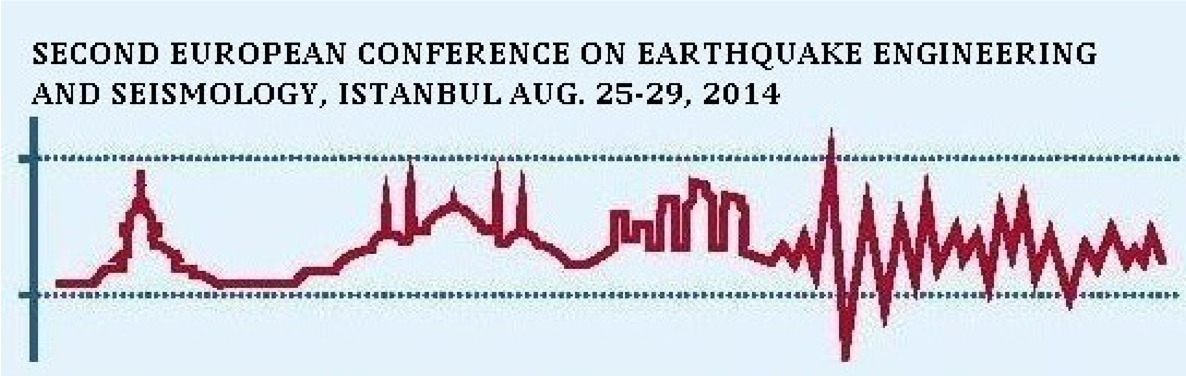

SEISMIC ASSESSMENT OF MASONRY BUILDINGS ACCORDING TO EUROCODE 6 AND 8

Dimitra N. STAVRELI [1] and Stephanos E. DRITSOS [2]

ABSTRACT

It is clear from several analyses that, in comparison with buildings designed according to new codes, the existing building stock has a lower seismic capacity. 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 mentioning that Eurocode 6 (2005) provides only for the design of new masonry buildings and does not consider 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 Organization (EPPO, 2012). These methods are applied a selection of 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 is 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 give high failure indices for multilevel buildings. On the other hand, Eurocode 6 (2005) results give similar failure index values 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 failure indices from the EPPO (2012) method are always higher than unity, even in the case of simple buildings where the respective values are expected, by definition, to be much lower than unity and where the rigorous method of assessment applied within the framework of Eurocode 8 (2005b) justified these low failure index values. Therefore, a correction factor to multiply the resistance index of the EPPO (2012) method is proposed in the present work. Furthermore, it is found that the results of the EPPO (2012) method are not significantly influenced by the number of storeys, which is found to be in contrast with the results of the rigorous analytical method that was used for the same buildings.

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INTRODUCTION

Seismic assessment methods for masonry buildings according to Eurocode 6 (2005), Eurocode 8 Part 3 (2005b) and an approximate method proposed by the Greek Earthquake Planning and Protection Organization (EPPO, 2012) are compared in this paper. It must be noted here that seismic actions are not specifically considered by Eurocode 6 (2005), as it concerns only the design of new masonry buildings. In addition, the out-of-plane deformation of the walls is not considered by Eurocode 8 (2005b), while Eurocode 6 (2005) simultaneously considers in-plane and out-of-plane action effects. Through a parametric study aimed to emphasise possible different results when assessing the seismic behaviour of a structure, these three methods are applied to selected buildings with particular characteristics.

To begin with, the investigation considers a six-storey building, for which failure indices according to Eurocode 6 (2005), Eurocode 8 (2005b) and the approximate method are computed and compared. Then, a sample of representative buildings are analysed, so the failure indices for "simple" masonry buildings (as defined in Part 1 of Eurocode 8, 2005a) can be determined in order that some very important conclusions can be drawn from the comparison of the three methods in such buildings. Moreover, the investigation of the influence of the number of storeys on failure indices of particular buildings was considered crucial. For this purpose, a differentiation between a six-storey building, a four-storey and a two-storey building is performed, so that the failure index resulting from the different methods could 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 were typical of that time. The walls of the basement and the ground floor storeys consist of three-leaf stone masonry in contrast to the rest of the storeys, which 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 the 5th floor of the building.

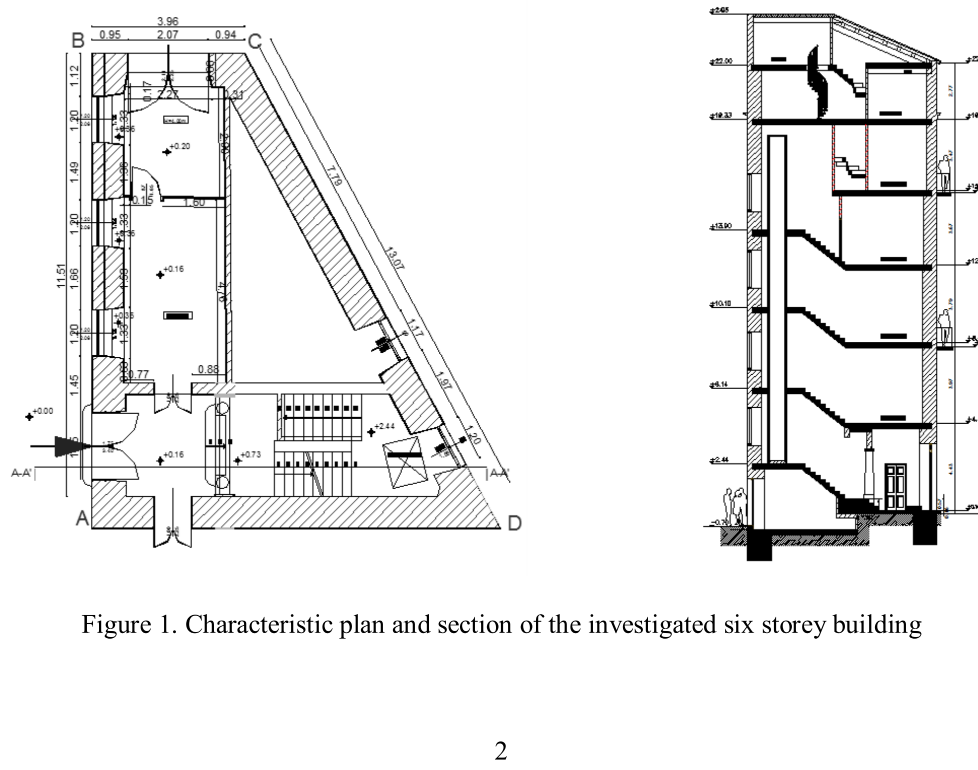

Figure 1. Characteristic plan and section of the investigated six storey building

As far as the floors are concerned, these consist of wood and are supported by wooden beams (of 20 by 18 cm cross section) spaced 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.

MASONRY MECHANICAL PROPERTIES

The mechanical properties of the materials were either determined from laboratory tests or by consulting 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 the three leaf masonry and 18 kN/m^2 for the 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 analyses. The live load of the floors and staircase was considered to be 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 are considered to have bending (out of plane) and membrane (in plane) stiffness. The foundation of the building was considered as 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 and the foundation factor were considered as 1.50 and 1.00 respectively. The building was checked for seismic loading according to the Greek Seismic Code (EAK2000, 2003) with a dead and live load combination of $1.35G+1.5Q$ as well as the seismic load combinations of $G+0.3Q\pm Ex\pm 0.3Ey$ and $G+0.3Q\pm 0.3Ex\pm Ey$.

FAILURE INDICES THROUGH EUROCODE 6

Through ECTools (2002) software, the cross sections of the 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 and
N_{Rd} is the design vertical load resistance of the masonry section and is given by the following equation:

$$N_{Rd} = \Phi_i * t * f_d \tag{2}$$

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 and
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 and
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 and
l_c is the length of area under compression.

It was decided that an average failure index of the cross sections per floor for bending and shear should be calculated. The average failure index in every floor (j), was determined 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 each section in bending or shear and
A is the cross sectional area of the pier or spandrel.

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

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. In addition, the failure index of the building can be considered the maximum value of flexure or shear. Therefore, λ_b = 11.4 is the failure index of the building according to Eurocode 6 (2005) and occurs in the 2nd storey.

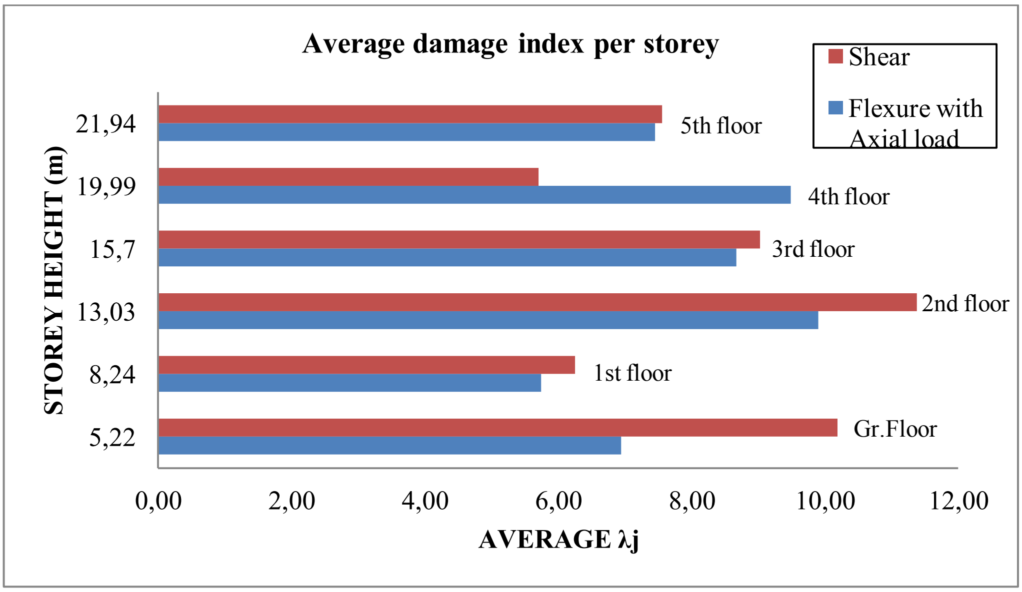

Figure 2. Average failure indices according to Eurocode 6 in flexure with axial load and shear per storey

Table 1. Building failure indices in flexure and shear

λ_b Flexure	λ_b Shear
9.90	11.4

FAILURE INDICES THROUGH EUROCODE 8

In this section, a computation of failure indices through the 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 as follows:

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

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

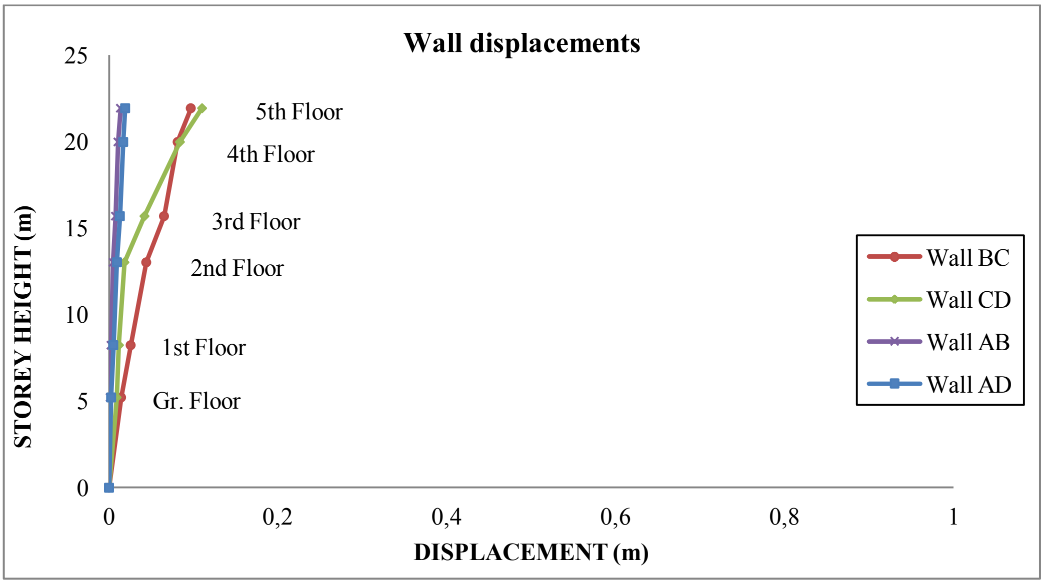

Figure 3. Wall displacements per storey

Figure 3 shows that the largest displacement occurs in wall CD, which is the longest wall. This is one of the most important influencing parameters that causes this wall to have the highest displacement. In addition, the supporting transverse walls to this wall are not perpendicular and are at an angle, so they do not offer a high resistance to this wall. It can also be observed that the angle of the line of displacements is normal in the first storeys, in contrast with upper storeys where the displacement line angle notably increases, especially in walls CD and BC, which also have the feature that their transverse walls are not perpendicular to them. This absence of normality from the 2nd floor onwards causes higher drifts.

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

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

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

According to Part 3 of Eurocode 8 (2005b), the λ_i failure indices are computed by dividing these drifts by their upper limit acceptance criteria 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. λ_i failure indices 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.

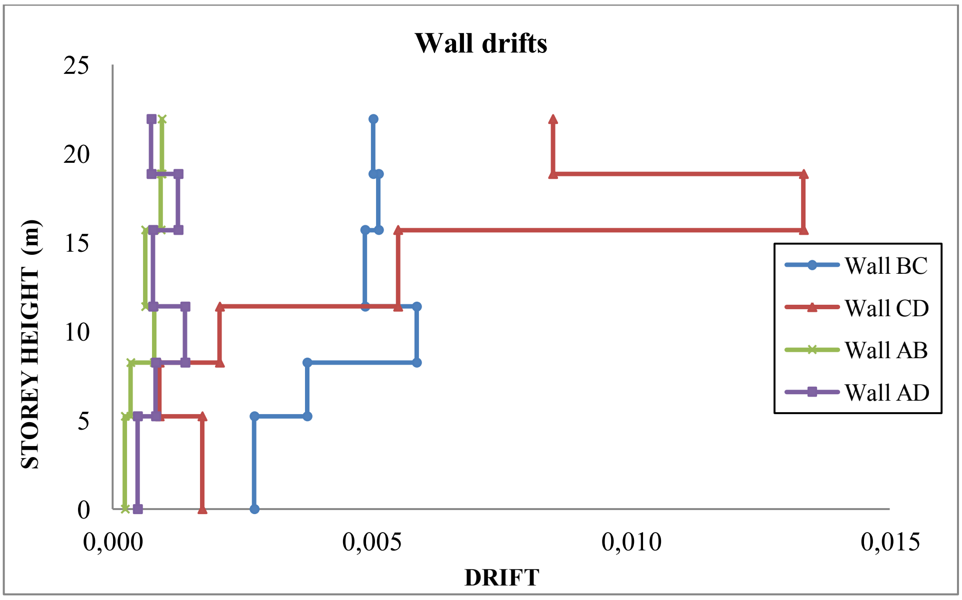

Figure 4. Wall drifts through Eurocode 8 (2005b)

Table 2 and Figure 4 show that the maximum failure index value and the maximum drift occurs in wall CD. Some significant failure indices also appear in wall BC in contrast with walls AB and AD, which have very low failure indices. All these findings are due to the characteristics of the walls that were described 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 for the building is the maximum λ_i value derived from Table 2 and, specifically, it is the value $\lambda_b = 3.3$, which corresponds to shear in the 4th storey.

COMPARISON OF EUROCODE 6 AND EUROCODE 8 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 all the 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 and there is not much of a difference from storey to storey. This is contrary to Eurocode 8 (2005b), where the maximum value appeared in 4th storey. Consequently, it is evident that Eurocode 8 (2005b) gives the highest failure indices in upper storeys, so these are the critical storeys for the assessment. On the other hand, Eurocode 6 (2005) gives almost the same results as far as the storeys are concerned.

ESTIMATION OF FAILURE INDEX OF THE BUILDING ACCORDING TO THE APPROXIMATE ASSESSMENT METHOD OF THE 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 a Seismic Intensity Index (H) and a Seismic Resistance Index (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 ($H1$) that is defined according to the seismic zone and the influence of neighbouring buildings index ($H2$), as shown in Eq. (8).

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

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

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

Table 3. Seismic resistance Indices

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:

$$R = 0.20R1 + 0.15(R3 + R5) + 0.10(R4 + R7 + R8) + 0.05(R2 + R6 + R9 + R10) \quad (9)$$

When computing the indices for the investigated 6-storey building, it was found that:

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

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

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

COMPARISON OF THE EPPO METHOD FAILURE INDEX WITH THE FAILURE INDEX 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 $\lambda_b' = \frac{H}{\beta * R}$.

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

FAILURE INDICES FOR "SIMPLE" MASONRY BUILDINGS

With the aim to expand the sample of representative buildings that are analysed, the failure indices for "simple" masonry buildings have been computed. According to Part 1 of Eurocode 8 (2005a), "simple" buildings are those which belong to importance 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, it could be expected that seismic assessment of capacity of those buildings would result in failure indices somewhat lower than unity.

It is obvious that the configuration parameters of a building are too numerous. Therefore, it was initially decided to consider concrete slabs in order to ensure an effective diaphragm action.

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

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

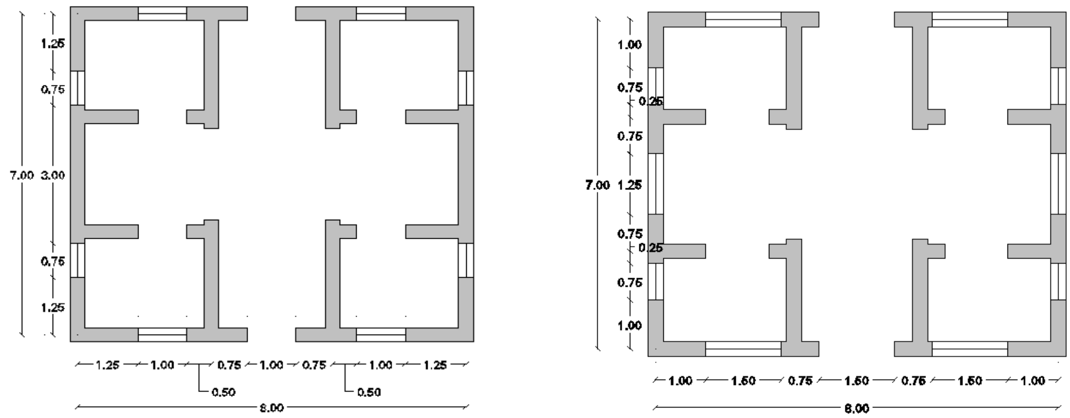

Figure 5. "Simple" building plans A, B, C (left) and D, E (right)

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

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

Building	Seismic Zone	Number of Storeys	Masonry Thicknes s (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	0.30	9.1	3.2	1.51	0.29	0.25	2.21
D	II	1	0.30	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 cross sectional area of the piers as percentage of the area of above floors and the seismic zone.

According to the above and having chosen buildings D and E that have a minimum area of piers for a particular seismic zone, a correction of the EPPO (2012) λ_b failure indices can be performed so that, according to Eurocode 6 (2005) and Eurocode 8 (2005b) respectively, the maximum values of the EPPO (2012) λ_b 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 pier area. Consequently, possible β values for the EPPO (2012) λ_b factors can be determined, as shown in Table 5.

Table 5. β correction factor values

Building	Seismic Zone	Number of Storeys	Masonry thickness (m)	Minimum pier area (%)	β	
					EC6	EC8
D	II	1	0.30	6.0	1.22	7.69
C	II	1	0.30	9.1	1.47	9.09
E	I	2	0.25	5.0	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 those of Eurocode 8 (2005b), the H/R factors do not exceed unity. Considering Eurocode 6 (2005), the results could be characterised as being fairly close to the H/R values and, thus, the obtained correction factors are close to unity. It can also be observed that according to Eurocode 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 comparison of six-storey buildings to four-storey and two-storey buildings is necessary, so that the failure λ of Eurocode 8 (2005b) and λ_b of EPPO (2012) can be computed. Taking also into account the respective results in "simple" one-storey and two-storey buildings, a very safe conclusion as far as the influence of the number of storeys on the failure indices will be feasible.

The second level pre-earthquake assessment method 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 "simple" buildings and 2 storey, 4 storey and 6 storey buildings

Building	λ_b EC8 (Flexure & Axial Load)	λ_b EC8 (Shear)	λ_b EPPO
"Simple" 1 storey (A)	0.24	0.20	1.57
"Simple" 2 storey (B)	0.29	0.25	1.65
2 storey (C)	0.48	0.60	5.20
4 storey (D)	0.92	1.47	5.70
6 storey (E)	1.09	3.36	6.00

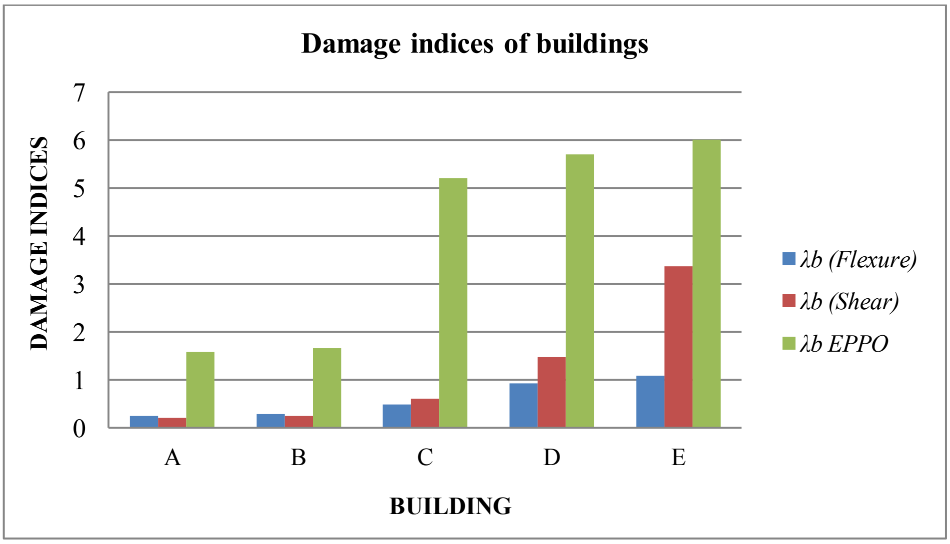

Figure 6. Failure indices in "simple" 1 storey, "simple" 2 storey, 2 storey, 4 storey and 6 storey buildings

From the above table and figure, it can be seen that the EPPO (2012) λ_b indices gradually icrease when adding storeys until a maximum value, which is 6.00 for the 6 storey building. This is in contrast with the results of the failure indices of Eurocode 6 (2005), which steadily increase as the number of storeys increase. On the contrary, the failure indices calculated through Eurocode 8

(2005b) rapidly 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 taken into account the number of storeys and, more importantly, the influence of the number of the storeys should be reflected in the other 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 defined in Part 1 of Eurocode 8 (2005a).

Within 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) they are compared in terms of deformations (drifts). Moreover, Eurocode 8 (2005b) does not consider out-of-plane wall deformation while the framework of Eurocode 6 (2005) considers both in-plane and out-of plane action effects simultaneously.

In the case studies investigated in the present work, it was found that Eurocode 8 (2005b) gives the highest failure indices for higher level buildings. On the other hand, Eurocode 6 (2005) results in similar failure index values from storey to storey.

When comparing the Greek Earthquake Planning and Protection method (EPPO, 2012) with Eurocode 6 (2005) and Eurocode 8 (2005b), it appears that failure index values for the EPPO (2012) method are always higher than unity, even in cases of "simple" buildings where respective values, by definition, would be expected to be much lower than unity and the rigorous method of assessment applied in the framework of Eurocode 8 (2005b) justifies these low failure index values. Therefore, a correction factor multiplying the resistance index of the EPPO (2012) method has been proposed by the present work.

Finally, it was found that the results of the EPPO (2012) method are not much influenced by the number of storeys, 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 can be concluded that it is necessary that the number of storeys should have greater influence when determining the seismic resistance of a building.

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