Seismic behaviour of "Simple Masonry Buildings" according to EN 1998

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Abstract

"Simple Masonry Buildings" (SMB), according to Eurocode 8, are buildings of a regular plan, which fulfil specifications depending on the system of masonry (i.e. plain, confined or reinforced). These buildings are expected to perform good seismic behaviour and explicit safety verification is not mandatory. In the present study the assessment of seismic vulnerability of one- and two-storey "simple buildings" of unreinforced brick masonry is performed by means of finite element elastic spectral analyses using as a failure criterion the value of the principal tensile stresses developed under seismic load combinations; failure will occur when the predicted principal tensile stress exceeds the tensile strength of masonry. Two buildings are analysed for various ground conditions and for ground acceleration, α_g, given from the Greek Annex for Seismicity Zone I. Herein the results are presented in tabular mode; for each wall, the maximum tensile stress, the fibre in tension (inside or outside), and the load combination that causes the stress, are presented. It is proven that the buildings behave very well under the imposed seismic actions and no damage is expected. For ground acceleration $a_g.S > \alpha_{urm,g}$, the construction of unreinforced masonry (URM) buildings is not permitted, but here these buildings are analysed and their seismic behaviour is compared with that of SMB. It is found that the expected damage is insignificant even if they are in high seismicity zones with ground acceleration equal to 0.24g.
Keywords: simple masonry buildings, seismic performance, vulnerability.

1 Introduction

Until recently, masonry buildings were designed and built based on empiricism, without the use of any analysis procedures. Masonry buildings were stiff, regular

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structures. After the introduction of steel and reinforced concrete as structural materials, masonry buildings were constructed to be more flexible, with irregularities in plan and in elevation due to application of configuration rules suitable only for frame structures. The consequences of the lack of the use of any analysis results in the design of masonry buildings, and the adoption of modern plan layouts, was their poor seismic behaviour; this is the reason why masonry buildings are still considered to be vulnerable to earthquakes. Nevertheless, observations and vulnerability studies after earthquakes have proved that modern, low-rise masonry buildings, plain in plan, with reinforced concrete slabs, horizontal tie-belts at both the sills and the lintels and without large recesses, suffered no damage even after strong earthquakes, as described in detail by Fardis et al. [3], Karantoni and Bouckovalas [6], Karantoni [7] and Karantoni and Fardis [9]. Therefore, according to EN 1998-1-1 "Design of structures for earthquake resistance" [2], for a category of masonry buildings called "Simple Masonry Buildings" (SMB), explicit safety verification is not mandatory. These masonry buildings must comply with the specifications of EN 1996-1-1 "Design of masonry structures" [1], and EN 1998-1-1 and are expected to perform satisfactorily to earthquake excitation.

Stylianidis et al. [10] studied the response of a series of SMB by means of 3d-frame elastic analyses. In this paper, the results of linear elastic analyses of a series of SMB modelled by finite elements are presented. If the developed principal tensile stresses under the seismic loads exceed the tensile strength of masonry, failure occurs. This is the criterion adopted herein to predict the seismic performance of SMB under consideration. This criterion was also used to predict the seismic behaviour of the three buildings in Karantoni and Fardis [8] and it was found that the predicted damage is in very good agreement with the damage developed.

According to EN 1998-1-1, plain masonry buildings (URM) are not allowed if the acceleration in site, $a_g{\cdot}S$ is greater than $\alpha_{\mathrm{g,urm}}$, where a_g is the design ground acceleration on type A ground, and S is the soil factor. Greek Annex [5], adopted the recommended value $\alpha_{\mathrm{g,urm}}$=0.20g. Consequently, the URM buildings are permitted only in Seismicity Zone I for which stands a_g=0.16g, and only for ground of type A, B and C for which the product $a_g{\cdot}S$ is less than 0.20g. The number of floors depends on the product $k{\cdot}$g, where k is a factor that takes into account the mean length of the structure's shear walls. The buildings fulfil the minimum requirements of Greek Annex of EN 1996-1-1, [4], and EN 1998-1-1, [5], and a detail description of them is given in what it follows. The analyses performed for ground of type A as well as for ground of type E nevertheless URM are not permitted to be constructed, and indicate that no damage is expected.

In addition, in this study the same buildings analysed for Seismicity Zone II (with a_g=0.24g), condition which results $a_g{\cdot}S > \alpha_{\mathrm{g,urm}}$, i.e. where URM buildings are not allowed. The results demonstrate that the damage expected due to the design seismic loads is practically insignificant.

2 Simple masonry buildings in the present study

For the purpose of this study, two buildings, one 1-storey (Building **IA1**) (I or II for the Seismicity Zone I or II, respectively, A or E for the corresponding type of ground, 1 or 2 indicates the number of storeys) and one 2-storey (Building **IA2**) with plan layouts similar to that shown in Figure 1, were designed to implement the EN 1996-1-1 and EN 1998-1-1 specifications as well as the corresponding Greek Annexes. Namely:

- For $\alpha_{\mathrm{g,urm}}$ =0.20g the allowable number of storeys is 2 for k=1.305 which stands for the buildings of Fig. 1.
- The plan configuration of the building fulfil all the following conditions:
  a) The plan is approximately rectangular,
  b) The ratio between the length of the small and the length of the long side in plan is 0.72 > 0.25.
  c) The area of projections of recesses from the rectangular shape is 13.5m^2 and less than the allowable which is 15% of the total area in plan, which in buildings of the study equals to 23.55m^2.
- For shear walls stands:
  a) the effective thickness of shear walls, t_{ef}=0.24m and is equal to the required $t_{ef,min}$,
  b) the maximum ratio h_{ef} /t_{ef} of the wall effective height to its effective thickness equals 8.86 and do not exceed the maximum permitted value, $(h_{ef}/t_{ef})_{\max}$=12, and
  c) the ratio h/l of the greater clear height of the openings adjacent to the wall to the length of the wall is much greater than the minimum allowable $(l/h)_{min}=0.50$.
- The minimum cross section area is expressed as a minimum percentage, $p_{A,min}$, of the total floor area per storey and for the buildings under consideration is 6.6% for the x and y direction too.
- The shear walls of the building meet also all the following conditions:
  a) the building is stiffened by shear walls, arranged almost symmetrically in plan in two orthogonal directions,
  b) a minimum of two parallel walls is placed in two orthogonal directions, the length of each wall being greater than 30% of the length of the building in the direction of the wall under consideration,
  c) at least for the walls in one direction, the distance between these walls is greater than 75% of the length of the building in the other direction.
  d) at least 75% of the vertical loads are supported by the shear walls.
  e) shear walls are continuous from top to bottom of the building.
- There is not difference in mass and in horizontal shear wall cross-section in both orthogonal horizontal directions between adjacent storeys.
- No wall is connected with walls in the orthogonal direction at distance greater than the maximum allowable, which is 7.0 m.

EN 1996-1-1 recommends the construction of reinforced concrete tie-belts at the top of the walls and at vertical spaces no more than 4.00m from the floor. As the height of the buildings is typically 3.00m, only one tie-belt is necessary. It is common practice in Greece and other seismic prone areas the construction of tie-belts not only at the top of the walls but at the lintels and even at the sills, too. The buildings under consideration are supposed to have two tie-belts, one at the lintels and the other at the top of the walls.

The buildings, which for the reasons presented in the previous topic, cannot manipulated as “simple”, are buildings similar to the above but: the buildings IIA1 and IIA2 are supposed to be in Seismicity Zone II and in ground of the type A, with one- or two-storeys, respectively. In this case the product $a_g{\cdot}S$ is greater than 0.20*k*.g and according to EN 1998-1-1 these buildings should not be constructed of plain masonry. The same stands for the buildings IE1 and IE2 (one- or two- storey buildings in ground of type E, in Seismicity Zone I) as well as for the buildings IIE1 and IIE2 (the former buildings is Seismicity Zone II).

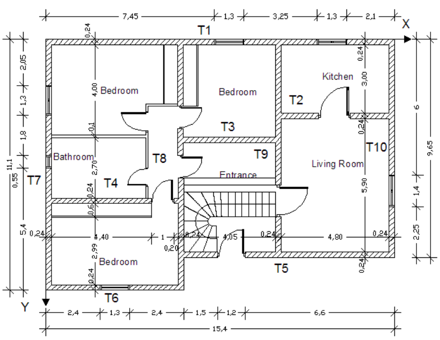


Figure 1: The “SMB” of the study, a typical plan of ground storey.

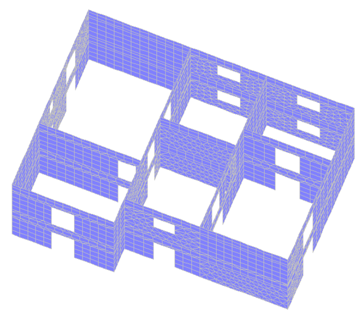


Figure 2: The finite element model of structural walls of the two-storey “SMB”.

3 Assumptions for the analyses

The bricks have a nominal compressive strength of f_{bc}=15 MPa, while the mortar is of the type M5 (i.e. f_m = 5 MPa). The characteristic compressive strength of the masonry $f_{wc,k}$ was estimated from the relation of EN 1996-1-1:

$$f_{wc,k} = K f_b^{0.7} f_m^{0.3} \tag{1}$$

while the design compressive strength $f_{wc,d}$ was estimated using the following formula:

$$f_{wc,d} = \frac{f_{wc,k}}{\gamma_m} \tag{2}$$

where:
K is a factor depended on the Group of the masonry blocks and the presence or not of vertical joint in the width of the wall,

f_b is the normalized compressive strength of the bricks,
f_m is the compressive strength of the mortar, and
γ_m is the partial safety factor for the masonry taken equal to 2.2
The elastic modulus E was taken by:

$$E = 1000 f_{wc,k} \tag{3}$$

where:
$f_{wc,k}$ is the characteristic compressive strength of the masonry
The design tensile strength of the masonry $f_{wt,d}$ supposed to be:

$$f_{wt,d}=0.1 f_{wc,d} \tag{4}$$

The above relations give:

$$f_{wc,d} = 2.2 \text{ MPa and } f_{wt,d}=0.22 \text{ MPa.}$$

We used the Finite Element Method to perform linear elastic analyses of the structures. The discretisation of the buildings were made with a sufficient number of elements with dimensions ~0.50 x 0.50 m and analysed using the computer program ACORD-CP. In Fig. 2 the discretisation of the two-storey building is presented. The finite elements used are a combination of thick plate and shear plane elements in order to consider both the in-plane and out-of-plane stresses of a structural wall under seismic actions. Elements of the same type were used to model the reinforced concrete slabs so that the diaphragmatic action of the floors and the roof can be account for. The elastic spectral analyses were based on the specifications of EN 1998 and the Greek Annex making the assumptions that the buildings were designed to house a family (ordinary building of II importance class). According to Greek Annex of EN 1998, the elastic response spectrum in use should be that of the Type 1 (Fig. 3) with the parameters given from EN 1998.

The fundamental eigenvalues of the buildings found to be lower than T_B =0.15sec and thus the equation (5), which describes the increasing branch of the design response spectrum, was used for the horizontal components of the seismic action.

$$\text{For} \quad 0 \le T \le T_B: \quad S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3} \right) \right] \tag{5}$$

where:

$S_d(T)$ is the design response spectrum;
T is the vibration period of a linear single-degree-of-freedom system, herein it was found from spectrum analyses;
a_g is the design ground acceleration of type A ground ($a_g = \gamma_I . a_{gR}$);
T_B is the lower limit of the period of the constant spectral acceleration branch, equals 0.15 of type A and E ground;
S is the soil factor, equals 1.0 for type A, and 1.4 for type E;
q is the behaviour factor, equals 1.5 for URM

The analyses were performed for nine loading combinations presented in Table 1; one combination for the gravity loads and eight combinations for the seismic loads, all with partial load factors taken from EN1991.

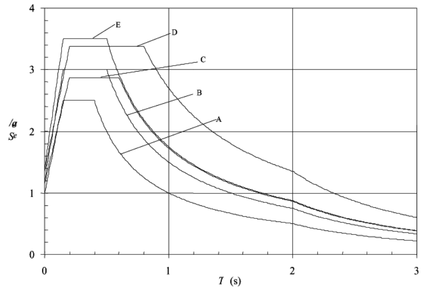


Figure 3: Type 1 elastic response spectrum for ground types A to E (5% damping).

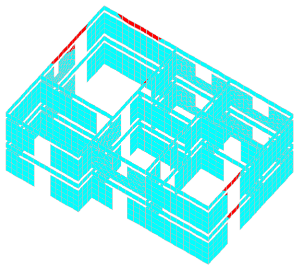
Table 1: The load combinations used in the analyses.

	Name	Combination
Gravity loads	L0 =	1,35*G + 1,5*Q
Load combinations used for the seismic analyses	L1 =	G + 0.3*Q + Ex + 0.3*Ey
	L2 =	G + 0.3*Q + Ex – 0.3*Ey
	L3 =	G + 0.3*Q - Ex + 0.3*Ey
	L4 =	G + 0.3*Q - Ex – 0.3*Ey
	L5 =	G + 0.3*Q + Ey + 0.3*Ex
	L6 =	G + 0.3*Q + Ey – 0.3*Ex
	L7 =	G + 0.3*Q - Ey + 0.3*Ex
	L8 =	G + 0.3*Q - Ey – 0.3*Ex

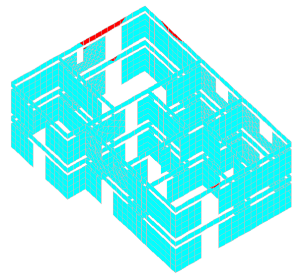
G = dead loads
Q = live loads
Ex and Ey are the seismic loads along the x and y directions, respectively

4 Seismic behaviour of "simple masonry buildings"

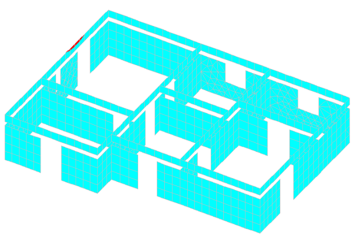
A series of analyses performed and some characteristic results are presented in Fig. 4 where the red (dark) colour depicts the areas of the walls that develop principal tensile stresses higher than the tensile strength of the masonry. In the following tables the developed maximum principal tensile stresses to the SMB IA1 and IA2, as well as to the rest buildings of the study, are presented for all structural walls in order to derive conclusions that are as clear as possible. In Fig. 4 and in Tables 2 and 3, L0-L8 represents the load case, which is denoted in Table 1, and (+) or (-) denotes the external or internal fibre of the wall respectively. For each wall, the first row shows the most critical of the four loading combinations of seismic loading along the x axis with partial load factor equal to 1.0 (i.e. load combinations L1-L4). The second row refers to the action along the y axis (load combinations L5-L8). The principal tensile stress, which is higher than the tensile strength of the masonry, is typed with red letters. It is noteworthy that the ground acceleration 0.24g in seismic zone II is 64% higher than that in seismic zone I., i.e. 0.16g.



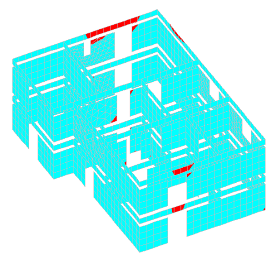
(a) Building IA1, L0



(b) Building IA2, L2



(c) Building IIA2, L2



(d) Building IIE2, L4

Figure 4: Overstressed areas of masonry (in red (dark) colour).

In more detail:

- The one-storey SMB IA1 can sustain successfully the design seismic loads. Two nodes of structural walls T1 and T7 fail, but the reason is the eccentricity imposed by the long span slab, which rests on these. For comparison, the principal tensile stresses (p.t.s.) for the load combination L0 are also presented in the Table for these walls (see also Fig.4(a)).
- In two-storey SMB IA2 all the walls of the ground storey (with the exception of only one node of the wall T7 (see Fig. 4(b)) develop stresses under f_{wt}. Because of the vertical loads of the upper storey, the p.t.s. are reduced. The behaviour of the walls of the upper storey is similar to this of the building IA1 with little higher p.t.s. One node of the wall T10 and two nodes of walls T1 and T7 develop p.t.s. higher than f_{wt}.
- The p.t.s. are higher for the seismic direction parallel each wall, it is explained by the translational mode of vibration which governs their dynamic behaviour.
- The one-storey buildings IE1, IIA1 and IIE1, exhibits the same seismic behaviour as the SMB IA1, but according to EN1998 and its Greek Annex it is not allowed to be constructed from plain masonry.
- The two-storey buildings IE2 and IIA2 behaves almost the same because the design response spectrum $S_d(T)$ has almost the same value, 0.23g and 0.25g, respectively; in the ground storey one node of the walls T7 and T10 develops

p.t.s. higher than f_{wt}; in the first floor five nodes develop p.t.s. higher than f_{wt}, totally (see Fig. 4(c)).

- In the two-storey building IIE2 all the walls except T2 and T4 and T3 in the ground, develop p.t.s. higher than f_{wt} in only one or two nodes, Fig 4(d).

The distribution of the damage over the storeys of the two-storey buildings is not uniform. The upper storey seems to be more vulnerable than the ground storey. This result is expected in low-rise masonry buildings because of the existing small compressive stresses due to low gravity loads that allow the development of higher tensile stresses at the upper floor, although the horizontal inertia forces are lower than those in the ground floor where the compressive stresses are higher due to double gravity loads. Vulnerability studies after Aegion and Pyrgos (in Greece) earthquakes, [6], [7], [9], have shown that in two-storey buildings more or less similar to these of the present study, the level of damage of the upper storey is a little higher than the lever of damage of the ground storey. This behaviour is not expected in higher rise masonry buildings where the horizontal seismic loads at the lower storeys are considerably increased and the beneficial compressive stresses cannot equalize the tensile ones.

Table 2: Principal tensile stresses of one-storey buildings.

Building Wall	SMB IA1	IE1	IIA1	IIE1
T1	0.26 (+) L3 0.26 (+) L6	0.26 (+) L4 0.26 (+) L6	0.26 (+) L4 0.26 (+) L6	0.27 (+) L4 0.27 (+) L6
	0.47 (+) L0			
T2	0.06 (-) L3 0.05 (-) L6	0.07 (-) L3 0.06 (-) L6	0.07 (-) L3 0.06 (-) L6	0.08 (-) L3 0.08 (-) L6
T3	0.03 (+) L4 0.03 (+) L8	0.04 (+) L4 0.04 (+) L8	0.05 (+) L4 0.04 (+) L8	0.06 (+) L8 0.05 (+) L8
T4	0.08 (+) L2 0.07 (+) L7	0.09 (+) L2 0.08 (+) L7	0.09 (+) L4 0.08 (+) L7	0.10 (+) L4 0.08 (+) L7
T5	0.17 (+) L2 0.17 (+) L7	0.17 (+) L2 0.18 (+) L7	0.17 (+) L2 0.18 (+) L7	0.18 (+) L2 0.19 (+) L7
T6	0.10 (+) L1 0.11 (+) L5	0.11 (+) L1 0.12 (+) L5	0.11 (+) L2 0.12 (+) L5	0.11 (+) L2 0.13 (+) L5
T7	0.27 (+) L1 0.27 (+) L8	0.28 (+) L2 0.27 (+) L7	0.28 (+) L2 0.27 (+) L8	0.29 (+) L2 0.28 (+) L4
	0.49 (+) L0			
T8	0.11 (+) L3 0.11 (+) L6	0.13 (+) L3 0.12 (+) L6	0.13 (+) L3 0.12 (+) L6	0.14 (+) L3 0.12 (+) L6
T9	0.06 (-) L4 0.05 (-) L8	0.07 (-) L4 0.06 (-) L8	0.07 (-) L4 0.06 (-) L8	0.10 (-) L2 0.07 (-) L2
T10	0.19 (+) L1 0.19 (+) L5	0.20 (+) L3 0.19 (+) L5	0.20 (+) L3 0.19 (+) L5	0.21 (+) L3 0.19 (+) L6

Table 3: Principal tensile stresses of two-storey buildings.

Ground storey				
Building	IA2	IE2	IIA2	IIE2
Wall				
T1	0.10 (+) L1 0.13 (+) L5	0.12 (+) L1 0.17 (+) L5	0.14 (+) L2 0.16 (+) L6	0.27 (+) L2 0.23 (+) L7
T2	0.04 (+) L4 0.08 (+) L8	0.05 (+) L4 0.12 (+) L8	0.05 (+) L4 0.12 (+) L8	0.07 (+) L4 0.18 (-) L8
T3	0.04 (-) L1 0.04 (+) L5	0.05 (-) L1 0.06 (-) L8	0.05 (-) L1 0.06 (-) L5	0.08 (-) L3 0.10 (-) L5
T4	0.05 (+) L2 0.06 (+) L5	0.06 (+) L2 0.10 (-) L5	0.06 (+) L2 0.11 (-) L5	0.13 (+) L1 0.19 (-) L7
T5	0.06 (-) L4 0.09 (-) L8	0.07 (+) L4 0.13 (+) L8	0.10 (+) L4 0.13 (+) L8	0.23 (-) L4 0.19 (+) L7
T6	0.09 (+) L4 0.13 (+) L6	0.12 (+) L3 0.17 (+) L5	0.15 (+) L3 0.17 (+) L5	0.31 (+) L3 0.23 (+) L6
T7	0.25 (+) L2 0.20 (+) L7	0.28 (+) L2 0.21 (+) L7	0.29 (+) L2 0.21 (+) L7	0.35 (+) L1 0.23 (+) L7
T8	0.13 (+) L4 0.10 (+) L5	0.16 (+) L4 0.12 (+) L5	0.17 (+) L1 0.12 (+) L5	0.25 (-) L3 0.16 (+) L5
T9	0.11 (-) L2 0.07 (-) L7	0.13 (-) L2 0.08 (-) L7	0.14 (-) L1 0.08 (-) L7	0.26 (+) L4 0.09 (-) L2
T10	0.20 (+) L3 0.15 (+) L6	0.23 (+) L3 0.16 (+) L6	0.24 (+) L3 0.16 (+) L6	0.32 (+) L4 0.21 (+) L8
1st storey				
Building	IA2	IE2	IIA2	IIE2
Wall				
T1	0.28 (+) L3 0.29 (+) L6	0.28 (+) L4 0.29 (+) L6	0.28 (+) L4 0.29 (-) L6	0.28 (+) L4 0.30 (+) L6
T2	0.10 (-) L3 0.13 (-) L6	0.13 (-) L3 0.17 (-) L8	0.14 (-) L6 0.18 (-) L8	0.19 (-) L4 0.25 (-) L8
T3	0.04 (-) L1 0.04 (-) L5	0.05 (+) L3 0.05 (-) L5	0.05 (+) L1 0.05 (-) L5	0.10 (+) L1 0.07 (+) L5
T4	0.09 (+) L2 0.11 (-) L5	0.11 (+) L2 0.17 (-) L5	0.11 (+) L2 0.18 (-) L5	0.14 (+) L1 0.26 (-) L5
T5	0.18 (+) L1 0.19 (+) L7	0.19 (+) L2 0.20 (+) L7	0.19 (+) L2 0.20 (+) L7	0.19 (-) L2 0.23 (-) L7
T6	0.16 (-) L1 0.22 (-) L5	0.19 (-) L1 0.27 (-) L5	0.19 (-) L1 0.28 (-) L5	0.23 (-) L1 0.35 (-) L5
T7	0.43 (-) L2 0.39 (-) L7	0.48 (-) L2 0.43 (-) L7	0.49 (-) L2 0.43 (-) L7	0.57 (-) L2 0.48 (-) L7
T8	0.20 (-) L1 0.13 (-) L5	0.27 (-) L1 0.17 (-)..L5	0.29 (-) L1 0.18 (-) L5	0.40 (-) L1 0.24 (-) L5
T9	0.18 (+) L2 0.11 (+) L7	0.23 (-) L2 0.14 (+) L7	0.25 (+) L2 0.15 (+) L7	0.33 (+) L2 0.19 (+) L2
T10	0.35 (-) L3 0.3 (-) L6	0.41 (-) L3 0.34 (-) L6	0.43 (-) L3 0.34 (-) L6	0.53 (-) L3 0.40 (-) L6

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5 Conclusions

Based on linear elastic spectral analyses using the finite element method it is concluded that:

The “simple masonry buildings” of plain masonry according to EN1998 should perform very good seismic behaviour.

Buildings of plain masonry that fulfil the design requirements to be characterized as SMB but will be constructed in regions with higher than the permitted values of the product a_g.S exhibits also good seismic behaviour.

The maximum horizontal span of 7.0 m of shear walls is rather long if they are external, because the eccentricity of the reaction forces from the slab is significant.

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