Study on the behavior of a high reinforced concrete building with different kinds of partitioning masonry walls

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Abstract. The paper is a study on different types of masonry partitioning walls in a highrise reinforced concrete walls building. It will be built in Bucharest, Romania. The masonry walls are made of clay bricks for the first 4 cases studied and aerated concrete blocks for the last 2 cases. They are designed to bare the in plane and perpendicular efforts they are subjected to in the elastic stage. The effect these masonry walls have on the structures is shown. They have an important influence over the building's natural vibration periods and the slabs reinforcement design and shear stresses. The plastic hinges development is shown for all 6 cases. The pushover diagrams are compared separately for directions X and Y.

1. Introduction
This paper studies the behavior of different non-bearing walls (clay bricks and aerated concrete blocks) and their influence on the building. The partitioning walls have different volume weights and elasticity modulus values. These influence both the stresses in bearing elements and the structure’s stiffness. In literature it is shown that both clay bricks and concrete blocks walls show a similar response when the effect on seismic design parameters of certain design variables, like horizontal reinforcement ratio, and axial pre-compression level, is analyzed [1]. Both types of bricks are considered useable for walls [2]. Compared to clay solid bricks and concrete hollow blocks, autoclaved aerated concrete (AAC) blocks are easier to process and have smaller volume density. The strength of AAC is lower, the blocks dimensions are large and the homogeneity is good [3]. The AAC masonry is used in many parts of the world because of their deformability and their low weight that reduces the inertia force [4]. According to tests, the masonry prisms strength decreases as the bricks height/thickness ratio increases. Masonry prisms subjected to compression strength tests show diagonal cracks close to the corners and vertical cracks in the center [5]. Using masonry walls as partitions for high concrete structures may increase the lateral stiffness [6]. The masonry infill walls increase the buildings stiffness and strength. They may experience failure under in plane and out of plane seismic loads [7]. Autoclaved aerated concrete (AAC) is a popular choice of infill material due to its light weight. Infill masonry walls may experience failure under in plane and out of plane seismic demands [8]. Linear and nonlinear analysis show autoclaved aerated concrete blocks can also be used for bearing walls for low buildings in seismic areas [9]. Laboratory tests show that strength increases and displacement ductility decreases for reinforced concrete frames, if the infill masonry walls are stifferly connected to the structure [10]. Masonry infill walls may create soft stories. A
building with a soft story shows important variations in the buildings dynamic behavior compared to one without [11].

2. Building description

The building in study is situated in Bucharest, Romania. The codes in force used to design the building are: [12–19]. This is a high seismic area, the seismic acceleration is 0.30g (g is the gravity acceleration) [18]. The structure is composed of reinforced concrete walls, beams and slabs. The building contains a basement and 8 stories above it. Story height is 3m for all 8 stories above the basement. The non-bearing walls used are masonry made of burned clay bricks with vertical holes in cases 1 to 4 and aerated autoclaved concrete blocks in cases 5 and 6. These walls will be placed on beams and on slabs. Both concrete bearing and partitioning walls thickness is 25cm. The beams dimensions are 25·50cm. The slabs thickness is 14cm. This structure is stiff because of the concrete walls. It is interesting to establish if nonbearing walls do have an important stiffening effect on it, or if they do not have such a notable consequence. There are no small columns or tie beams in the nonbearing walls in this study. Partitioning walls lengths are P1 (2m), P2 (8m), P3 (3m) and P4 (5m).

The concrete used for the bearing elements is class C25/30 [16], with elasticity modulus $E_C=31000N/mm^2$. The masonry partitioning walls used in the 6 different cases have: $E_{M1}=6000 \text{ N/mm}^2$ for case 1, $E_{M2}=5250 \text{ N/mm}^2$ for case 2, $E_{M3}=3700 \text{ N/mm}^2$ for case 3, $E_{M4}=1800 \text{ N/mm}^2$ for case 4, $E_{M5}=1880 \text{ N/mm}^2$ for case 5 and $E_{M6}=1800 \text{ N/mm}^2$ for case 6. The masonry clay bricks with vertical holes (CB) are 240·115·88 (mm). Autoclaved concrete walls (AAC), the blocks are 600·240·100 (mm). Reinforcement bars are S345 with elasticity modulus $E_S=210000N/mm^2$ [16].

![Figure 1. 3D building image](image1)

![Figure 2. Building partitioning walls and bays dimensions](image2)

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3. Theory elements used

The seismic load is introduced by coefficient $c_s$. The base force $F_b$ is calculated using [17-18]: $\gamma_{I,e} = 1.2$ is the building’s importance-exposure coefficient, $\beta_0 = 2.5$ is the maximum value in the elastic spectrum and $q$ is the structure’s behavior factor for reinforced concrete uncoupled walls structures, with medium ductility. $q=3\cdot k_u \cdot \alpha_u / \alpha_1 = 3 \cdot 1 \cdot 1.35$ [18], $m = \text{building’s mass}$ [18], $\lambda = 0.85$, $a_g = 0.30g$ [18].

$$F_b = \gamma_{I,e} \cdot \beta_0 \cdot a_g / q \cdot m \cdot \lambda = c_s \cdot m \cdot g = 0.19 \cdot m \cdot g \ [kN] \quad (1)$$

The masonry strengths calculated here are: horizontal compression $f_{dh}$, vertical compression $f_d$, shear strength for horizontal direction $f_{vd,0}$, shear strength for inclined direction $f_{vd,i}$. The design strengths are determined from their characteristic values specific for each case, as seen in table 1: $f_{d,h}$, $f_k$, $f_{vk,0}$ and $f_{vk,i}$ in N/mm$^2$ [12](equations (2) to (7)). Masonry shear strength $f_{bt}$ is calculated according to the standard compression strength $f_b$. $f_{bh}=0.035 \cdot f_b$ for CB and $f_{ba}=0.080 \cdot f_b$ for AAC [19]. $f_b$ values for cases 1 to 6 are: 15N/mm$^2$, 12.5N/mm$^2$, 7.5N/mm$^2$, 5.0N/mm$^2$, 8.0N/mm$^2$ and 4.0N/mm$^2$ [19]. In equations (6) and (7), $\sigma_{od}$ is the axial vertical stress design value [19]. The masonry strengths insurance factor is $\gamma_M=1.9$ [19]. The concrete compression design strength $f_{cd}$ is determined using the characteristic strength $f_{ck}$ (8). For steel, $f_{od}$ is the design strength and $f_{ok}$ is the characteristic value (9).

$$f_{d,h} = f_{kh} / \gamma_M \ [N/mm^2] \quad (2)$$

$$f_d = f_k / \gamma_M \ [N/mm^2] \quad (3)$$

$$f_{vd,0} = f_{vk,0} / \gamma_M \ [N/mm^2] \quad (4)$$

$$f_{vd,i} = f_{vk,i} / \gamma_M \ [N/mm^2] \quad (5)$$

$$f_{ok,i} = 0.22 \cdot f_{bt} \cdot (1 + 5 \cdot \sigma_{od} / f_{bh})^{0.5} \ [N/mm^2] \quad (6)$$

$$f_{vk,i} = 0.1 \cdot f_{bh} \cdot (1 + 16 \cdot \sigma_{od} / f_{bt})^{0.5} \ [N/mm^2] \quad (7)$$

$$f_{cd} = f_{ck} / \gamma_C = 25/1.5 = 16.67 \ N/mm^2 \quad (8)$$

The volume weight for hollow clay bricks is $\gamma_{CB}=18\text{kN/m}^3$ and $\gamma_{AAC}=10\text{kN/m}^3$ for the autoclaved concrete blocks [19]. The load combinations SCX and SCY used to design the structure are 1.0-permanent loads+0.4-variable loads+1.0-seismic loads. The fundamental combination FC is 1.35-permanent loads +1.5-variable loads [14]. The non-bearing walls mechanical properties are taken from [19].
f_{yd} = f_y / \gamma_S = 345 / 1.15 = 300 \text{ N/mm}^2 \quad (9)

### Table 1. Masonry strengths

| case | f_k | f_d | f_{th} | f_{hm} | f_{vk,0} | f_{vd,0} | f_{vk,i} | f_{vd,i} |
|------|-----|-----|--------|--------|----------|----------|----------|----------|
| 1    | 6.00 | 3.15 | 1.38   | 0.72   | 0.15     | 0.173    | 0.09     |
| 2    | 5.25 | 2.76 | 1.18   | 0.62   | 0.15     | 0.154    | 0.08     |
| 3    | 3.70 | 1.95 | 0.98   | 0.51   | 0.15     | 0.115    | 0.06     |
| 4    | 1.80 | 0.95 | 0.59   | 0.31   | 0.13     | 0.096    | 0.05     |
| 5    | 4.70 | 2.47 | 2.09   | 1.10   | 0.13     | 0.11     | 0.06     |
| 6    | 2.35 | 1.23 | 1.44   | 0.75   | 0.13     | 0.078    | 0.04     |

### 3.1. Reinforced concrete beams design theory

Reinforcement in beams is designed according to $M_{Ed}$ according to [16]. $\lambda_x$ is the beam section compressed area height [16]. As is the minimum horizontal reinforcement area for beams. $f_{ctm} = 2.6 \text{ N/mm}^2$ is the medium value of the concrete tensile strength. As resulted as 3Φ 25, as seen in figure 5.

$$M_{Ed} = b \cdot \lambda_x \cdot f_{cd} \cdot (d - \lambda_x/2) = A_s \cdot f_{yd} \cdot z \ [kNm] \quad (10)$$

$$m = M_{Ed} / (b \cdot d^2 \cdot f_{cd}) \quad (11)$$

$$z = d - \lambda_x/2 = d - d \cdot (1 - (1 - 2 \cdot m)^{0.5})/2 \ [\text{mm}] \quad (12)$$

$$A_{s, min} = \min \{0.26 \cdot f_{ctm} / f_{yk} \cdot b \cdot d; 0.0013 \cdot b \cdot d\} \quad (13)$$

### Figure 5. Reinforced concrete beam section

### 3.2. Masonry walls design theory

$M_{Rd}$ is the wall’s bearing bending moment [12]. $M_{Rd(M)}$ is the bearing bending moment from the masonry area [12]. $A_c$ is the wall’s compressed area [12]. $y_c$ is the distance between the compressed masonry area ($A_c$) weight center $G_c$ and the wall’s weight center $G$ [12].

$$M_{Rd} = M_{Rd(M)} = N_{Ed} \cdot y_c \ [kNm] \quad (14)$$

$$A_c = N_{Ed} / (0.85 \cdot f_d) \ [\text{mm}^2] \quad (15)$$

### Figure 6. Masonry wall section

$S$ is the earthquake action. $t = 250 \text{mm}$ is the wall’s width. [12]. $V_{Rd1}^*$ is the masonry wall’s bearing shear force and $V_{Ed}$ is the horizontal shear force from the seismic loads combination. $h_{pan}$ and $l_{pan}$ are the height and length of the masonry area panel. $V_{Rd}$ is the minimum value among $V_{Rd1}^*$ and $V_{Rd,i}$. In equation (18), $b$ is a correction coefficient connected to the masonry walls dimensions (height $h=h_w$ and length $l_w=l_{pan}$) [19]. $b=1.5$ if $h/l_w \geq 1.5$, $b=1.0$ if $h/l_w < 1.0$ and $b=h/l_w$ if $1.0 \leq h/l_w < 1.5$. $\sigma_{td} = N_{Ed} / (t \cdot l_{pan})$.

$$V_{Rd1}^* = 0.4 \cdot (N_{Ed} + 0.8 \cdot V_{Ed} \cdot h_{pan} / l_{pan}) \ [kN] \quad (16)$$

$$V_{Ed} \leq l_{pan} \cdot t \cdot f_{vd,0} \quad (17)$$

$$V_{Rd,i} = A_w \cdot f_{vd,i} / b \quad (18)$$
4. Elastic analysis results

4.1. Natural periods of vibration

Table 2. Natural periods of vibration

|       | case 1 | case 2 | case 3 | case 4 | case 5 | case 6 |
|-------|--------|--------|--------|--------|--------|--------|
| T1    | 0.399s | 0.404s | 0.419s | 0.449s | 0.404s | 0.429s |
| T2    | 0.350s | 0.355s | 0.368s | 0.396s | 0.353s | 0.374s |
| T3    | 0.342s | 0.346s | 0.356s | 0.378s | 0.345s | 0.361s |

The seismic action is applied by the coefficient in formula (1). The non-bearing walls stiffness influences the structure’s natural periods of vibration. The values increase from case 1 to 4 and from 5 to 6 as the partitioning walls elasticity modulus decreases.

4.2. Efforts in masonry partitioning walls

Efforts in masonry walls are evaluated at all stories, as tables 3 and 4 show. They are designed for the efforts at story 8, because the highest effective efforts are there. Effective and bearing bending moments $M_{Ed}$ and $M_{Rd}$ and effective axial force $N_{Ed}$ are in table 5. Effective and bearing shear forces $V_{Ed}$ and $V_{Rd}$ are in table 6. $M_{Ed}$ and $V_{Ed}$ values are greater for the higher stories. This is something seen in theory, as the seismic force distribution shows higher values at the top stories. There are also higher displacements at the top stories. These values also have an impact on the nonbearing walls. The partitioning walls have the same $N_{Ed}$ values at all stories, because they only carry their own loads, unlike the bearing walls. The nonbearing walls have the upper side set not to get loaded from the slabs above them. In tables 3 and 4 it is seen that all axial forces are the same for identical nonbearing walls situated at different stories.

Table 3. Masonry walls efforts

| STORY | P1 | P2 | P3 | P4 |
|-------|----|----|----|----|
|       | $N_{Ed}$ | $V_{Ed}$ | $M_{Ed}$ | $N_{Ed}$ | $V_{Ed}$ | $M_{Ed}$ | $N_{Ed}$ | $V_{Ed}$ | $M_{Ed}$ |
| [kN]  | [kN] | [kNm] | [kN] | [kN] | [kNm] | [kN] | [kN] | [kNm] |
| 1     | 26  | 1   | 2   | 105 | 2   | 7   | 39  | 1    | 3    | 66   | 2    | 4    |
| 2     | 26  | 1   | 3   | 105 | 5   | 14  | 39  | 2    | 5    | 66   | 3    | 9    |
| 3     | 26  | 2   | 5   | 105 | 7   | 21  | 39  | 3    | 8    | 66   | 5    | 13   |
| 4     | 26  | 2   | 7   | 105 | 10  | 28  | 39  | 4    | 10   | 66   | 6    | 18   |
| 5     | 26  | 3   | 9   | 105 | 12  | 35  | 39  | 4    | 13   | 66   | 8    | 22   |
| 6     | 26  | 4   | 10  | 105 | 14  | 42  | 39  | 5    | 16   | 66   | 9    | 27   |
| 7     | 26  | 4   | 12  | 105 | 17  | 49  | 39  | 6    | 18   | 66   | 11   | 31   |
| 8     | 26  | 5   | 14  | 105 | 19  | 56  | 39  | 7    | 21   | 66   | 12   | 36   |

Table 4. AAC walls efforts

| STORY | P1 | P2 | P3 | P4 |
|-------|----|----|----|----|
|       | $N_{Ed}$ | $V_{Ed}$ | $M_{Ed}$ | $N_{Ed}$ | $V_{Ed}$ | $M_{Ed}$ | $N_{Ed}$ | $V_{Ed}$ | $M_{Ed}$ |
| [kN]  | [kN] | [kNm] | [kN] | [kN] | [kNm] | [kN] | [kN] | [kNm] |
### Table 5. Infill walls bending moments at story 8

| wall | N<sub>Ed</sub> [kN] | M<sub>Ed</sub> [kNm] | M<sub>Rd</sub> [kNm] | M<sub>Rd</sub> [kNm] | M<sub>Rd</sub> [kNm] | N<sub>Ed</sub> [kN] | M<sub>Ed</sub> [kNm] | M<sub>Rd</sub> [kNm] | M<sub>Rd</sub> [kNm] |
|------|-------------------|-----------------|-----------------|-----------------|-----------------|----------------|----------------|----------------|----------------|
| case 1 | case 2 | case 3 | case 4 | case 5 | case 6 | case 5 | case 6 |
| P1  | 26 | 14 | 25 | 25 | 24 | 24 | 14 | 8 | 13 | 13 |
| P2  | 105 | 56 | 408 | 408 | 403 | 391 | 58 | 31 | 226 | 224 |
| P3  | 39 | 21 | 56 | 56 | 56 | 54 | 22 | 12 | 32 | 32 |
| P4  | 66 | 36 | 162 | 162 | 160 | 156 | 37 | 20 | 91 | 90 |

### Table 6. Infill walls shear forces at story 8

| V<sub>Ed</sub> [kN] | V<sub>Rd</sub> [kN] | V<sub>Ed</sub> [kN] | V<sub>Rd</sub> [kN] |
|-----------------|-----------------|-----------------|-----------------|
| wall | case 1, 2, 3, 4 | case 5, 6 | case 5, 6 | case 5, 6 |
| P1  | 5 | 12 | 3 | 7 |
| P2  | 19 | 44 | 11 | 24 |
| P3  | 7 | 18 | 4 | 10 |
| P4  | 12 | 28 | 7 | 16 |

### 4.3. Seismic force perpendicular to the masonry walls

\[ F_{NBW}(z) = \gamma_{Le} \cdot a_g \cdot \beta_{NBW} \cdot k_x \cdot m_{NBW}/q_{NBW} = 1.86 \text{ kN/m}^2 \]  

This is the seismic force to the non-bearing walls [18]. \( \beta_{NBW} = 1 \) is the non-bearing walls amplification factor, \( k_x \) is a coefficient according to the non-bearing wall’s level (\( z \) = the distance from the partitioning wall’s top and bottom to the building’s base) and \( H \) is the building’s height [18].

\[ k_x = (k_{x1} + k_{x2})/2 = (1 + 2 \cdot z_1/H + 1 + 2 \cdot z_2/H)/2 \]  

\( k_{x1} \) and \( k_{x2} \) are the coefficients for the partitioning wall’s top and bottom. The greatest value for \( k_x \) is at the top story. \( q_{NBW} = 2.5 \) is the behavior factor for non-bearing walls. \( m_{NBW} = \gamma_{mas} \cdot g = 18 \cdot 0.25/g = 4.5/g \text{ kN/m}^2 \) is the wall mass/m². For clay brick walls (CB) \( F_{NBW} = 0.89 \text{ kN/m}^2, 1.05 \text{ kN/m}^2, 1.21 \text{ kN/m}^2, 1.38 \text{ kN/m}^2, 1.54 \text{ kN/m}^2, 1.70 \text{ kN/m}^2 \) and \( 1.86 \text{ kN/m}^2 \), for stories from 2 to 8. For autoclaved concrete walls (AAC) \( F_{NBW} = 0.49 \text{ kN/m}^2, 0.59 \text{ kN/m}^2, 0.67 \text{ kN/m}^2, 0.76 \text{ kN/m}^2, 0.86 \text{ kN/m}^2, 0.95 \text{ kN/m}^2 \) and \( 1.03 \text{ kN/m}^2 \), for stories from 2 to 8.
4.4. Slabs reinforcement design and shear stresses

The slabs stresses are not visibly influenced by walls stiffness, just by their weight. It is very important to mention the non-bearing walls (both CB and AAC) are all 25cm thick. They load the structure differently because they have different volume weights. This is why only one of the clay bricks walls (CB) cases and one of the aerated autoclaved concrete blocks walls (AAC) cases are considered here.

In figures 7 and 10, the walls placed on slabs are grey and those resting on beams are purple.

Figures 7.a and 10.a show the partitioning walls and the slab effective stress on the undistorted shape. Figure 7.b and 10.b contain the stresses on the deformed shape. The masonry walls are not seen here, so the stress values are clearer.

![Figure 7](image1)

**Figure 7.** Shear effective stress in slabs $\nu_{Ed}$ [N/mm$^2$] for CB

![Figure 8](image2)

**Figure 8.** Mx [kNm] in FC (CB)

![Figure 9](image3)

**Figure 9.** My [kNm] in FC (CB)

![Figure 10](image4)

**Figure 10.** Shear effective stress $\nu_{Ed}$ [N/mm$^2$] for AAC
The slabs reinforcement area $A_s$ is calculated using the effective bending moment $M_{Ed}$, the steel design strength $f_{yd}$ and the distance $d=h_{slab}(\text{slab thickness})-c(\text{reinforcement coverage})-\Phi(\text{reinforcement bars diameter})/2$. $M_{EdX}=20\text{kNm}$ and $M_{EdY}=13\text{kNm}$ for cases 1 to 4. For cases 5 and 6 $M_{EdX}=17\text{kNm}$ and $M_{EdY}=10\text{kNm}$ (figures 8, 9, 11 and 12). $A_s=10\Phi10$ (10 bars with 10mm diameter) on direction X and 10$\Phi8$ on Y. The same reinforcement area $A_s$ can be used for all 6 cases.

$$A_s = \frac{M_{Ed}}{(0.9 \cdot d \cdot f_{yd})} \quad (21)$$

Due to placing some partitioning walls (grey in figures 7 and 10) sitting directly on the slabs, there are some important shear stresses as it is seen in figures 7 and 10. The greatest stresses are not under the masonry walls that sit on slabs. They are near the partitioning walls resting on beams. These partitioning walls rest just on a certain length of the beams. The high stress values are seen at the walls edges. They are caused by shear force differences in these areas. The capable piercing/shear stress is calculated in equation (20).

$$\nu_{rd,c} = C_{rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} = 0.57 \text{N/mm}^2 \geq v_{min} = 0.49 \text{N/mm}^2 \quad (22)$$

$f_{ck}$ is introduced in N/mm$^2$, $k=1+(200/d)^{0.5}\leq2$ in mm, $\rho_1=(\rho_x \cdot \rho_y)^{0.5} \leq 0.02$. $\rho_x$ and $\rho_y$ are slab reinforcement coefficients on directions X and Y. $\nu_{min} = 0.035 \cdot k^{1/2} \cdot f_{ck}^{1/2}$. $C_{rd,c} = 0.18/\gamma_C$. $\nu_{rd,c}$ is greater than $\nu_{Ed}$ in figures 7 and 10, so the slab can withstand the piercing stress for all clay brick and autoclaved concrete non-bearing walls cases.

### 4.5. Partitioning walls stresses for seismic loads combinations

The $\sigma_x$ and $\sigma_z$ stress values in figures 13 to 20 do not surpass the masonry strength $f_{dh}$ and $f_d$ values.
Figure 15. $\sigma_x$ [N/mm$^2$] SCX (AAC)

Figure 16. $\sigma_x$ [N/mm$^2$] SCY (AAC)

Figure 17. $\sigma_z$ [N/mm$^2$] SCX (CB)

Figure 18. $\sigma_z$ [N/mm$^2$] SCY (CB)

Figure 19. $\sigma_z$ [N/mm$^2$] SCX (AAC)

Figure 20. $\sigma_z$ [N/mm$^2$] SCY (AAC)
The masonry is not cracked by these stresses. It is noticed that stress values increase towards the higher stories and reach greater values in walls perpendicular to the seismic action. This is because $\sigma_x$ is horizontal, in the walls plane. The values for the seismic forces perpendicular to the walls increase towards the higher stories. So do the walls displacements. This causes $\sigma_z$ to increase. Stresses $\tau_{xz}$ seen in figure 21 to 24, reach the maximum strength $f_{d,\text{zero}}$ values on large areas, especially on the same direction as the seismic action. The masonry does get cracked especially at the walls corners. It is seen that the stress values are similar for stories 2 to 8. The highest stress values are seen in short masonry walls. This is explained as the non-bearing walls sit on beams or on slabs and these elements transmit seismic efforts easier to smaller walls. All 3 stresses are close to 0 in non-bearing walls at the ground floor (story 1). These walls sit on the slab above the basement. They are not seismically loaded because the structure model only contains the stories above the basement.

There is no out of plane verification here. Only the in plane stresses are analysed.

5. Plastic stage results

5.1. Plastic hinges development
The pushover analysis was performed with the non-bearing walls placed as elements. They are made of CB for the first 4 cases and AAC for the latter 2. The bearing structure is made of reinforced concrete. The material properties were defined differently for concrete and masonry elements from the beginning of the linear analysis.
Figures 25 to 36 show the last plastic analysis steps. The colour code is: green for the plastic hinges being formed (B), blue for hinges that have reached their limit capacity (C), pink means the load has been redistributed (D) and red is for collapse (E). The number of D and E plastic hinges increases from case 1 to 2 as the bricks stiffness decreases. For cases 3 and 4, the number of D and E hinges decreases compared to case 2. This can be explained as a lower elasticity modulus may provide a higher ductility and so it can delay the hinges from reaching the last stages. The same can be said
about cases 5 and 6 used for the AAC walls. The results are more visible on X because the bearing walls are shorter. On direction Y, the plastic hinges developed to high stages are concentrated in the right side because there are more stiff bearing walls in that area. The plastic hinges development for cases 5 and 6 is similar to cases 3 and 4. This means there is not much difference between the cases that use light or heavy partitioning walls in terms of structure collapse mechanism.

5.2. Pushover diagrams

For all cases, the diagrams reach the suggested displacement (250cm) as seen in figures 37 and 38. This is a value set to be high enough for the structure to become clearly unusable. The chosen value could have been lower. On both directions, the structure has a lower rigidity for lower elasticity modulus partitioning walls. The structure reaches the top displacement for all the non-bearing walls solutions. Stiffer partitioning walls are associated to a greater base force. As the elasticity modulus drops, the base force decreases. This happens regardless of the material the non-bearing walls are
made of. Some sudden drops in rigidity are noticed for diagrams on direction X. This is not noticed for direction Y, where the rigidity decreases gradually for some of the diagrams. It is however worth mentioning that AAC masonry walls do generate lower stiffness diagrams compared to CB of resembling elasticity modulus.

6. Conclusions
The autoclaved concrete walls in cases 5 and 6 generate lower stresses in slabs. The structure behaves well for all masonry walls solutions. The base force is higher for high elasticity modulus masonry.

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