Masonry strength influence on the buildings’ performance

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Abstract. This paper studied the behaviour of a medium height, confined masonry walls building, in a medium seismic area (0.25g, where g is the gravity acceleration), in Iasi, Romania. The study focuses on different masonry strengths impact on the building’s elastic and plastic behaviour. It is important to establish if the structure can bare the gravitational and seismic loads if weaker bricks are used and also how the plastic mechanism is formed if the walls are made of stronger and stiffer masonry. It is also highlighted how the beams stiffness affects the structure ductility and how it can be improved. The building does not have the same stiffness on both X and Y directions, so this will also be taken into account.

1. Introduction

The paper compares the behaviour of medium height buildings, with different masonry strengths, in medium seismic areas. Load bearing walls can be made of other materials than clay bricks. Rammed earth walls are traditional and easy to make in some countries [1]. Bricks made of mud, water and straws are also used for masonry walls. There is no accurate design code for these materials to follow. The experimental results show that the adding of high cement ratios, can lead to optimum compressive strengths [2]. Concrete blocks can be used for masonry walls just like clay bricks. The compressive strengths of concrete and clay bricks are similar. The concrete bricks require more mortar, and they have a more brittle failing mechanism [3]. Clay brick walls have higher shear strength than concrete blocks walls, but lower deformation capacity [4]. Both clay and concrete of masonry walls subjected to eccentric loads crack at 40% to 60% of the maximum load at the time of the collapse [5]. According to tests on masonry prisms made using different bricks and mortars, the bricks elasticity modulus and mortar strength have an important influence on the masonry walls [6]. Confined masonry behaves well in moderate earthquakes. Masonry walls are thought to fail due to diagonal shear or sliding shear [7]. Well defined continuous load paths help the masonry buildings bare the earthquake loads. They can be achieved with regular structural layout with symmetry and uniformity both in plan and elevation [8]. Masonry walls reinforcement may be useful in medium seismic areas. The flexural strengths of reinforced masonry walls are 5-16 times higher than for the non-reinforced walls [9]. Horizontal walls reinforcement or connecting bars improve the seismic performance of confined masonry walls, by enhancing the ductile behavior to some extent and lateral load carrying capacity [10]. Masonry walls continuous horizontal reinforcement helps achieve a ductile behavior [11].
2. Building description

The study is used for a new building that will be situated in Iasi, Romania. The building 3D image can be seen in figure 1. The codes in force used to design the building are according with [12–18].

![3D building image](image1)

**Figure 1.** 3D building image.

This is considered a medium seismic area as the seismic acceleration is 0.25g (where $g$ is the gravity acceleration). The building contains a ground floor and 3 stories above it. Story height is 3m. The structure has masonry walls confined with tie beams and reinforced concrete tie-columns designed according to [18]. Hollowed bricks are used for the masonry walls. According to the masonry code, it is possible to use bricks of higher or lower strengths. The paper will highlight the buildings behavior according to the bricks strength. Two cases (A and B) of bricks strengths are considered here. In both cases some beams are slender and some are stiff. Case A uses stronger bricks. Case C highlights the buildings plastic stage behavior with the weaker bricks from case B and slender beams.

![Bays dimensions](image2)

**Figure 2.** Bays dimensions (8 bays on $X$ and 3 on $Y$).

![Bearing masonry walls](image3)

**Figure 3.** Bearing masonry walls on direction $X$. 
The floor plan with spans dimensions is in figure 2. The walls names on directions \( X \) and \( Y \) are in figures 3 and 4. Walls are red, beams and tie beams are blue, slabs are grey and reinforced concrete tie-columns are green. The program used to design is ETABS 2017.

![Figure 4. Bearing masonry walls on direction \( Y \).]

2.1. Materials used
Hollowed bricks 250x115x88 mm (\( L \times l \times h \)) are used for the masonry walls. For the first case studied, case A, the bricks standard strength is \( f_b = 15 \text{ N/mm}^2 \), mortar M10 and elasticity modulus \( E_m = 6000 \text{ N/mm}^2 \). For the second case B the bricks standard strength is \( f_b = 7.5 \text{ N/mm}^2 \), mortar M7.5 and elasticity modulus \( E_m = 3350 \text{ N/mm}^2 \) [12]. For the reinforced concrete tie beams and reinforced concrete tie-columns, the concrete class is C16/20 [16], with elasticity modulus \( E_C = 29000 \text{ N/mm}^2 \). Tie beams are beams placed in the masonry wall at each floor. Beams are placed between the masonry walls at each floor. Both beams and tie beams are attached to the floor slabs. They may have the same transversal sections and reinforcements.

Sometimes beams may have greater sections and reinforcement percentages. Reinforcement bars are S345 with elasticity modulus \( E_S = 210000 \text{ N/mm}^2 \) [16].

3. Theory elements used in the paper

3.1. Seismic force design
The seismic action is introduced in the model by coefficient \( \gamma_I \). The base force \( F_B (1) \) is calculated using [17, 18]. \( \gamma_I = 1.2 \) 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, \( q = 2.25 \cdot 1.25 \) [18], \( m \) is building’s mass [18]. \( \eta = 0.88 \) is the reduction factor, \( \lambda = 0.85 \), \( \alpha_g = 0.25g \) [18].

\[
F_B = \gamma_I \cdot \beta_0 \cdot \alpha_g \cdot q \cdot m \cdot \eta \cdot \lambda = \gamma_I \cdot m \cdot g \cdot \eta \cdot \lambda = cs \cdot m \cdot g = 0.17 \cdot m \cdot g \quad \text{[kN]} (1)
\]

3.2. Materials strengths
The stresses analyzed for masonry walls are \( \sigma_x \), \( \sigma_z \) and \( \tau_{xz} \). They are compared to the masonry design strengths [12]: horizontal compression \( f_{dh} \), vertical compression \( f_d \) and shear strength for horizontal direction \( f_{sdh} \). The design strengths (2) to (4) are determined from their characteristic values: \( f_{sk} \), \( f_k \) and \( f_{skh} \). The masonry strengths insurance factor is \( \gamma_M = 1.9 \) [12]. The concrete compression, tension and shear design strengths \( f_{cd}, f_{ctd} \) and \( f_{cvd} \) (7) to (10) are determined using the characteristic strengths \( f_{ck}, f_{ctk} \) with (9) if \( \sigma_{cp} \leq \sigma_{c:lim} \) and with (10) if \( \sigma_{cp} > \sigma_{c:lim} \). For steel, \( f_{yd} \) is the design strength and \( f_{yk} \) is the characteristic value. \( \gamma_M = 1.5 \), for concrete and 1.15 for steel [16]. \( N_{fd} \) is the axial force and \( Ac \) is the section area of the reinforced concrete tie-columns for which \( f_{cvd} \) is calculated. The masonry design bending strengths perpendicular to the wall \( f_{sid} \) and \( f_{sid2} \) (5) and (6) are calculated using their characteristic values \( f_{skd} \) and \( f_{skd2} \) [12]. (A) and (B) mean cases A and B respectively.

\[
f_{sk} = f_{skh}/\gamma_M = 1.18/1.9 = 0.62 \text{ N/mm}^2 \quad \text{(A)}; \ 0.86/1.9=0.45 \text{ N/mm}^2 \quad \text{(B)} (2)
\]

\[
f_{skd} = f_{skd2}/\gamma_M = 6.0/1.9= 3.15 \text{ N/mm}^2 \quad \text{(A)}; \ 3.35/1.9 = 1.76 \text{ N/mm}^2 \quad \text{(B)} (3)
\]
\( f_{vd,0} = f_{vk,0}/\gamma_M = 0.3/1.9 = 0.158 \text{ N/mm}^2 \) (A); 0.25/1.9 = 0.13 (N/mm²) (B) (4)

\( f_{vd1} = f_{vk1}/\gamma_M = 0.24/1.9 = 0.126 \text{ N/mm}^2 \) (5)

\( f_{vd2} = f_{vk2}/\gamma_M = 0.48/1.9 = 0.25 \text{ N/mm}^2 \) (6)

\( f_{cd} = f_{ck}/\gamma_M = 16/1.5 = 10.67 \text{ N/mm}^2 \) (7)

\( f_{cd} = f_{ck}/\gamma_M = 1.3/1.5 = 0.87 \text{ N/mm}^2 \) (8)

\( f_{cvd} = (f_{ck}^2 + \sigma_c f_{ck})^{0.5} \) (N/mm²) (9)

\( f_{cvd} = \left[ f_{ck}^2 + \sigma_c f_{ck} - (\sigma_c - \sigma_{c,lim})^2/4 \right]^{0.5} \) (N/mm²) (10)

\( \sigma_c = N_{Ed}/A_c \) (N/mm²) (11)

\( \sigma_{c,lim} = f_{ck} - 2 \left[ f_{cd} (f_{cd} + f_{ck}) \right]^{0.5} \) (N/mm²) (12)

\( f_{yd} = f_{yd}/\gamma_M = 345/1.15 = 300 \text{ N/mm}^2 \) (13)

Here, the values for \( \sigma_c \) are 1.28 N/mm², 0.88 N/mm², 0.48 N/mm² and 0.112 N/mm² from stories 1 to story 4. \( f_{cvd} \) values have resulted as 1.37 N/mm², 1.23 N/mm², 1.08 N/mm² and 0.924 N/mm² from stories 1 to story 4. Reinforcement for tie beams and reinforced concrete tie-columns is designed using the minimum reinforcement percentage 0.8% for \( a_g = 0.25g \) seismic areas. The load combinations ECX and ECY used to design the structure is 1.0·permanent loads+0.4·variable loads+1.0·earthquake loads [14] on directions X and Y.

3.3. Masonry walls bearing efforts parallel to the walls plane

\( M_{Rd} \) is the wall’s bearing bending moment [12]. \( M_{Rd(M)} \) and \( M_{Rd(As)} \) are the bearing bending moments from the masonry area and from the reinforced concrete tie-columns reinforcements respectively [12]. \( A_c \) is the wall’s compressed area [12].

\( M_{Rd} = M_{Rd(M)} + M_{Rd(As)} \) (kNm) (14)

\( A_c = N_{Ed}/(0.85\cdot f_{cd}) \) (mm²) (15)

\( M_{Rd(M)} = N_{Ed} \cdot y_c \) (kNm) (16)

In figure 5, \( y_c \) is the distance between the compressed masonry area weight center CG and the wall’s weight center WG [12]. The wall in figure 5 is used to explain theory elements used in the paper. It is not one of the piers P1 to P10. \( l_s \) is the distance between the reinforced concrete tie-columns at the wall’s edges. \( A_s \) is the reinforced concrete tie-columns horizontal reinforcement area. \( b_w = 25 \text{ cm} \) is the walls width and \( E \) is the earthquake direction. \( V_{Rd} \) is the masonry wall bearing shear force and \( V_{Ed} \) is the horizontal shear force from the seismic loads combination [12].

\( M_{Rd(As)} = l_s \cdot A_s \cdot f_{yd} \) (kNm) (17)

\( V_{Rd} = V_{Rd1} + V_{Rd2} + V_{RSC} \) (kN) (18)

\( V_{Rd1} = 0.4 \cdot (N_{Ed} + 0.8 \cdot V_{Ed} h_{pan}/l_{pan}) \) (kN) (19)

\( V_{Ed} \leq l_{pan} \cdot t \cdot f_{vd,0} \) (kN) (20)
where $h_{pan}$ = story height – 30 cm (tie beam height) and $l_{pan}$=$l_c b_w$ are the height and length of the masonry area panel. $V_{RD2}$ is the bearing horizontal shear force from the reinforcement in the reinforced concrete tie-columns at the walls compressed edge [12]. $A_c$ is the reinforcement area in the reinforced concrete tie-columns at the walls compressed edge. $\lambda_c$ is the reinforcement participation factor. Here, $\lambda_c$=0.20, for longitudinal reinforcement $\Phi_{16}$. $V_{RSC}$ is the shearing capacity for reinforced concrete tie-columns at the walls compressed area.

$$V_{RD2} = \lambda_c \cdot A_c \cdot f_{yd} \text{ (kN)}$$  \hspace{1cm} (21)

$$V_{RSC} = A_c \cdot f_{cvd} \text{ (kN)}$$  \hspace{1cm} (22)

### 3.4. Masonry walls bearing efforts perpendicular to the walls plane

The bearing bending moments perpendicular to the masonry walls $M_{Rd1}$ (horizontal) and $M_{Rd2}$ (vertical) values will be compared to the design bending moments values, $M_{Ed1}$ and $M_{Ed2}$ [12] calculated from the model respectively.

$$M_{Rd1} = W_w \cdot (f_{d1} + \sigma_{dw}) \text{ (kNm/m)}$$  \hspace{1cm} (23)

$$M_{Rd2} = W_w \cdot f_{d2} \text{ (kNm/m)}$$  \hspace{1cm} (24)

$$\sigma_{dw} = \gamma_{max} \cdot H_S \text{ (N/mm}^2)$$  \hspace{1cm} (25)

$$W_w = 1000 \cdot \frac{t^2}{6}=1000 \cdot 250^2/6=10416666 \text{ mm}^3/m$$  \hspace{1cm} (26)

where $W_w$ = 1000 · $t^2/6$ is the wall resistance modulus (in mm $^3$/m), $t$ is the wall thickness, $\sigma_{dw}$ is the compression stress at the wall’s middle height section [12], $\gamma_{max}$ =18 kN/m$^3$ is masonry weight per cubic meter and $H_S$ is masonry story walls height from the building bottom to the level where $\sigma_{dw}$ is calculated. The values for $\sigma_{dw}$ are 0.189 N/mm$^2$, 0.135 N/mm$^2$, 0.081 N/mm$^2$, 0.027 N/mm$^2$ for stories 1, 2, 3 and 4 respectively.

$M_{Rd1}$ is 3.28 kNm/m, 2.72 kNm/m, 2.15 kNm/m and 1.59 kNm/m for stories 1, 2, 3 and 4 respectively and $M_{Rd2}$= 2.60 kNm/m for all 4 stories.

### 4. Elastic analysis results

Dimensions and longitudinal reinforcement in beams, tie beams and reinforced concrete tie-columns can be found in table 1. As is the longitudinal reinforcement area. The bars are blue discs and the diameter ($\Phi$) of bars (in mm) is written for each element.

#### Table 1. Dimensions and longitudinal reinforcement in beams, tie beams and reinforced concrete tie-columns.

| Tie beam (beam) and lintel 25x30 | Reinforced concrete tie-columns 25x25 |
|----------------------------------|---------------------------------------|
| As → 4$\Phi_{14}$               | As → 4$\Phi_{16}$                     |

#### 4.1. Efforts and stresses in masonry walls parallel to the walls plane

Efforts parallel to the walls planes are presented as graphics to compare the values easier. The $M_{Ed}$ values are split to 100 and the $N_{Ed}$ and $V_{Ed}$ values are split to 10 to fit the numbers in the tables.
4.1.1. Masonry walls efforts on direction X.

Figure 6. $M_{Ed}$ for cases A and B.

Figure 7. $N_{Ed}$ for cases A and B.

Figure 8. $V_{Ed}$ for cases A and B.
4.1.2. Masonry walls efforts on direction Y.

Figure 9. $M_{Ed}$ for cases A and B.

Figure 10. $N_{Ed}$ for cases A and B.

Figure 11. $V_{Ed}$ for cases A and B.
Figures 8 to 11 show efforts in the masonry walls: $M_{Ed}$ (bending moment), $N_{Ed}$ (axial force) and $V_{Ed}$ (shear force) and the bearing efforts respectively $M_{Rd}$, $N_{Rd}$ and $V_{Rd}$. In the elastic stage, the effective efforts $N_{Ed}$, $M_{Ed}$ and $V_{Ed}$ in cases A and B are comparable for both directions $X$ and $Y$.

4.2. Masonry walls efforts perpendicular to the walls plane

$M_{Ed}$ values reach from -0.5 kNm/m to 1 kNm/m, below $M_{Rd}$ and $M_{Ed}$ values are from -2.1 to 2.4 kNm/m below $M_{Rd}$, for both cases A and B, on both directions $X$ and $Y$ (figures 12 and 13).

The highest positive values are blue, yellow means 0 and the lowest negative values are purple.

5. Analysis results in the plastic stage

The structure’s behavior in the plastic stage is analyzed using the pushover diagrams PX1 and PY1 for case A and PX2 and PY2 for case B for directions $X$ and $Y$ respectively.

5.1. Plastic hinges development and pushover diagrams

The plastic hinges colors are: green when the hinge is formed, blue when the hinge reaches its limit, pink when the load is redistributed and red at collapse.
In case A, (figures 15 and 16) on direction $Y$, most beams connecting walls are thick. They are placed above doors and are accompanied by wall panels (figure 14 a), so they do not develop high stages in plastic hinges. In case B (figures 17 and 18) the structure reaches the displacement 700 mm on direction $X$ as many plastic hinges reach the collapse stage. For direction $Y$, the plastic hinges in advanced stages are seen in the thin beams (figure 14 b) at the edges and at the reinforced concrete tie-columns bottoms. There are both thick and thin beams for both directions. There are more thin beams on direction $X$ (the longitudinal side).

5.2. Pushover diagrams

The pushover diagrams are shown in figure 25. In case A, the maximum base force reached for direction $Y$ is twice greater than for $X$. The top displacement is also greater for $Y$. This is caused by the plastic hinges in the short beams connecting walls on $X$, giving in early. These beams, placed above windows, are thin. This is not valid for the internal walls. Thin beams are not accompanied by wall panels (figure 14). In case B, the maximum top displacement is smaller on direction $Y$ because plastic hinges reach high stages at the reinforced concrete tie-columns bottoms.

The structure’s ability to reach such a high displacement on direction $X$ is explained by the weaker masonry walls in this case. The beams work better with walls of lower bricks strength. This is also seen on direction $Y$, but it is less evident because the structure has a higher rigidity on $Y$. Rigidity is regarded as base force/displacement. For all 4 pushover cases the building’s rigidity is maintained the same throughout the analysis.
5.3. Walls stresses
Stresses $\sigma_x$ and $\sigma_z$ reach the highest values in the lower stories. They surpass the masonry strengths $f_{dh}$ and $f_d$ respectively from the second step of the analysis. At that moment, $\sigma_x$ values range from -0.7 N/mm² to 0.7 N/mm² for case A (PX1 and PY1) and from -0.5 N/mm² to 0.5 N/mm² for case B (PX2 and PY2). The stresses distribution is seen in figures 20 to 25. Stresses $\sigma_z$ values reach from -3.15 N/mm² to 3.15 N/mm² for case A and from -1.8 N/mm² to 1.8 N/mm² for case B.

The highest positive values are blue, yellow is for 0 and the lowest negative values are purple. Stress $\tau_{xz}$ values range from -0.16 N/mm² to 0.16 N/mm² for case A and from -0.13 N/mm² to 0.13 N/mm² for case B. These values are greater than the strength $f_{vd,0}$ from the first step of the analysis. $\tau_{xz}$ values are greater in walls parallel to the seismic action. In some walls, $\tau_{xz}$ values are greater than the masonry strength for all stories, so the all the wall area is cracked there. All stress values decrease from the building bottom to the top. For cases PX1 and PY1 all stresses reach higher values. The masonry is cracked before the plastic mechanism is formed.

![Figure 20. $\sigma_x$ distribution.](image1)

![Figure 21. $\sigma_z$ distribution.](image2)

![Figure 22. $\sigma_z$ distribution.](image3)

![Figure 23. $\sigma_z$ distribution.](image4)
6. Structure enhancement
It is interesting to further study the influence the thick beams (figure 14.a) have on the structure’s plastic behavior. If all beams are thin (figure 14.b) as in case C, then the structure becomes a mechanism at smaller top displacements and the rigidity value on direction $Y$ decreases. The plastic hinges development for case C is in figures 26 and 27 and the pushover diagrams for cases A, B and C are in figure 28. The structure’s rigidities are in table 2. For case C, the plastic mechanism is better than for cases A and B because there are less plastic hinges at the reinforced concrete tie-columns bottoms.

6.1. Plastic hinges development

6.2. Pushover diagrams for cases A, B and C

| Table 2. Structure rigidities |
|-----------------------------|
| Case | Rigidity | Direction |
|-----|----------|-----------|
| A   | 28.3     | X         |
|     | 34.4     | Y         |
| B   | 16.6     | X         |
|     | 19.5     | Y         |
| C   | 15.6     | X         |
|     | 17.0     | Y         |
7. Conclusions

- The structure can bare the efforts it is subjected to, for both cases A and B.
- In the plastic stage it behaves better in case B, so here lower strength bricks are better.
- Thick beams above doors assure larger displacements on direction $X$ in the plastic stage in case B.
- If only thin beams are used, as in case C, rigidities decrease for both directions $X$ and $Y$.
- The plastic mechanism is better in case C than in A and B, as there are less plastic hinges at the reinforced concrete tie-columns bottoms.

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