Adding a masonry floor on top of a reinforced concrete high building in a seismic area

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Abstract. The paper studies the influence of adding an aerated autoclaved concrete (AAC) masonry walls story above a reinforced concrete structure of a high building. This structure is situated in a high seismic area (0.30\( g \), where \( g \) is the gravity acceleration) in Bucharest, Romania. The new story will be lightweight because of the AAC. It is necessary to establish if the existing structure can bare the new story gravity loads and to determine the new building’s seismic behaviour. It is also determined if the new AAC masonry walls can bare the seismic loads they are to be subjected to. Both elastic and plastic analysis are present in the study.

1. Introduction

The paper presents the effect of adding a confined masonry (CM) walls story on top of an existing building. The existing structure is a dwelling building with reinforced concrete (RC) walls. The added story contains aerated autoclaved concrete (AAC) confined masonry walls. This is only a study. It is not about an already erected building. According to literature, masonry-partitioning walls contribute significantly to the buildings seismic behaviour [1]. Masonry can also be used in bearing walls. In confined masonry walls, the number of confining columns has an important influence on the energy dissipation capacity, ductility, cracking pattern and strength of the walls. It is preferred to use more confining columns [2]. Confined masonry is a suitable solution for building low to medium rise buildings. Both earthquakes and tests show the effectiveness of confined masonry systems. This performance is also influenced by the masonry properties and structure configuration, reinforcement detailing in tie beams, beams and slender columns [3]. Masonry walls failure is consisted mostly of diagonal cracking and out of plane displacements according to finite element analysis [4]. An AAC story on top of a RC building becomes a soft story. A soft story presents lower stiffness than the others. It may be called flexible story [5]. Aerated autoclaved concrete walls crack through the aerated concrete blocks, showing that the full strength capacity of the blocks was reached. The effect of the glue mortar is expected to be most significant in enhancing the flexural strength of the AAC walls under loading [6]. In multi-story residential building, the AAC load bearing blocks can reduce the buildings weight and improve the seismic behavior, as the AAC blocks have smaller volume density and are homogenous [7]. The results of AAC building seismic analysis point out that for 1-2 story buildings and for low to medium seismic areas, the damage limit may not be reached. For high seismic areas and multi-story buildings, significant damage can be expected. It is recommended to use geometrical regularity in plan to improve AAC structures performance [8]. According to linear and nonlinear analysis, autoclaved concrete walls can be used as bearing elements for low buildings in...
According to shaking table tests, AAC masonry walls have a great energy dissipation capacity [10]. Walls slenderness is also important to take into account as it may lower the masonry walls lateral load bearing capacity [11].

2. Building description
This is a study used for a building that will have a penthouse built over the top story. The structure is situated in Bucharest, Romania. The codes in force used to design the building are: [12–18]. This is considered a high seismic area as the seismic acceleration is 0.30 g (g is the gravity acceleration) [18]. The building contains a ground floor and 7 stories above it. Story height is 3 m. It is a reinforced concrete walls dwelling building. The penthouse will have confined masonry bearing walls made of AAC. The analysis is done for two stages. Stage 1 is without the penthouse (figure 1) and stage 2 is with the penthouse (figure 2). Reinforced concrete walls plan is in figure 3 and confined masonry walls plan is in figure 4. Walls are red, beams and tie beams are blue, slender columns are green and slabs are grey. The program used to design is ETABS 2017.

The masonry walls will be made of autoclaved concrete bricks, because this material is light weight. These walls will be placed on top of the reinforced concrete walls. The confining elements are concrete slender columns and tie beams. According to the masonry design code in force autoclaved concrete blocks can be used for bearing walls [12].
Figure 4. AAC walls names and bays dimensions.

For the reinforced concrete walls, the class is C25/30 [16], with elasticity modulus $E_c = 31000 \text{ N/mm}^2$. For the masonry aerated autoclaved concrete walls, the blocks are 600·100·240 (mm) with standard strength $f_b = 8 \text{ N/mm}^2$, mortar M10 and elasticity modulus $E_M = 3760 \text{ N/mm}^2$ [12]. Concrete C16/20 is used for the confining elements at the top story, $E_c = 29000 \text{ N/mm}^2$. Reinforcement bars are S345 with elasticity modulus $E_S = 210000 \text{ N/mm}^2$ [16].

3. Theory elements used

3.1. Seismic load

The seismic load is introduced by coefficient $c_s$. The base force $F_b$ in (1) 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, $q = 3.5 \cdot 1.15$ [18], $m =$ building’s mass [18]. $\eta = 1$ is the reduction factor, $\lambda = 0.85$, $a_g = 0.30g$ [18].

$$F_b = \gamma_{I,e} \cdot \beta_0 \cdot a_g \cdot q \cdot m \cdot \eta \cdot \lambda = c_s \cdot m \cdot g = 0.22 \cdot m \cdot g \text{ (kN)} \quad (1)$$

3.2. Materials design strengths

The stresses analyzed for masonry penthouse walls are $\sigma_x$, $\sigma_z$, $\tau_{xy}$, $\tau_{xz}$ and $\tau_{yz}$. They are compared to the masonry design strengths in (2) to (6) [12]: horizontal compression $f_{dh}$, vertical compression $f_d$, shear strength for horizontal direction $f_{vd,0}$, horizontal and vertical strengths perpendicular to the wall $f_{dx1}$ and $f_{dx2}$. The design strengths are determined from their characteristic values: $f_{kh}$, $f_k$, $f_{vk,0}$, $f_{xk1}$ and $f_{xk2}$. The masonry strengths insurance factor is $\gamma_M = 1.9$ [12]. The concrete compression design strength $f_{cd}$ in (7) is determined using the characteristic strength $f_{ck}$. For steel, $f_{yd}$ in (8) is the design strength and $f_{yk}$ is the characteristic value. $\gamma_C = 1.5$, for concrete and $\gamma_S = 1.15$ for steel [16].

$$f_{dh} = f_{dk}/\gamma_M = 2.09/1.9 = 1.1 \text{ N/mm}^2 \quad (2)$$
$$f_d = f_k/\gamma_M = 4.7/1.9 = 2.47 \text{ N/mm}^2 \quad (3)$$
$$f_{vd,0} = f_{vk,0}/\gamma_M = 0.25/1.9 = 0.13 \text{ N/mm}^2 \quad (4)$$
$$f_{dx1} = f_{xk1}/\gamma_M = 0.1/1.9 = 0.05 \text{ N/mm}^2 \quad (5)$$
$$f_{dx2} = f_{xk2}/\gamma_M = 0.2/1.9 = 0.10 \text{ N/mm}^2 \quad (6)$$
$$f_{cd} = f_{ck}/\gamma_C = 25/1.5 = 16.67 \text{ N/mm}^2 \quad (7)$$
$$f_{yd} = f_{yk}/\gamma_S = 345/1.15 = 300 \text{ N/mm}^2 \quad (8)$$
3.3. Concrete walls design theory

$N_{Ed}$ is the axial force from the seismic loads combinations. $M_{Ed,0}$ (9) and $M_{Ed,s}$ (10) are the ground floor and upper floors design bending moments and $M_{Ed}$ (15) is the wall’s bearing bending moment. $M'_{Ed}$ is the bending moment value given by the seismic loads combination for any story [16]. $l_w$ is the walls length. $A_S$ is the vertical reinforcement area at the walls edges. It is determined form (13) for the ground floor and from (14) for the other floors. $A_c$ is the walls compressed area. $b=250$ mm is the walls width. $k_M =1.15$ for DCM (medium ductility buildings). $S$ is the seismic direction.

$$M_{Ed,0} = M'_{Ed}$$  \hspace{1cm} (9)

$$M_{Ed,s} = k_M \cdot \Omega \cdot M_{Ed} \leq \Omega \cdot M'_{Ed}$$  \hspace{1cm} (10)

$$\Omega = M_{Ed,0} / M_{Ed,0}$$  \hspace{1cm} (11)

$$A_c = N_{Ed} / f_{cd}$$  \hspace{1cm} (12)

$$A_S = (M_{Ed,0} \cdot N_{Ed} \cdot 0.9 \cdot l_w / 2) / (f_{cd} \cdot 0.9 \cdot l_w)$$  \hspace{1cm} (13)

$$A_S = (M_{Ed,s} \cdot N_{Ed} \cdot 0.9 \cdot l_w / 2) / (f_{cd} \cdot 0.9 \cdot l_w)$$  \hspace{1cm} (14)

$$M_{Ed} = A_c \cdot f_{cd} \cdot 0.9 \cdot l_w + N_{Ed} \cdot 0.9 \cdot l_w / 2$$  \hspace{1cm} (15)

Figure 5. Reinforced concrete wall section.

3.4. Concrete beams design theory

Reinforcement in beams, detailed in figure 6, is designed according to $M_{Ed}$ (16) [16]. $\lambda_c$ is the beam section compressed area height [16]. $A_{s,min}$ is the minimum horizontal reinforcement area for beams. $f_{ctm} = 2.6$ N/mm$^2$ is the medium value of the concrete tensile strength. Reinforcement for tie beams and slender columns at the top story is designed using the minimum reinforcement percentage 1% for $a_g=0.30g$ seismic areas.

$$M_{Ed} = b \cdot \lambda_c \cdot f_{cd} \cdot (d-\lambda_c / 2) = A_s f_{cd} z \ (kNm)$$  \hspace{1cm} (16)

$$m = M_{Ed} (b \cdot d^2 f_{cd})$$  \hspace{1cm} (17)

$$z = d-\lambda_c / 2 = d-d \cdot (1-(1-2m)^{0.5}) / 2 \ (mm)$$  \hspace{1cm} (18)

$$A_{s,min} = \min \{0.26 \cdot f_{ctm} / f_{cd} \cdot b \cdot d; \ 0.0013 \cdot b \cdot d\}$$  \hspace{1cm} (19)

Figure 6. Reinforced concrete beam section.
3.5. Confined masonry walls design theory

$M_{Rd}$ (20) is the wall’s bearing bending moment [12]. $M_{Rd(M)}$ (22) and $M_{Rd(As)}$ (23) are the bearing bending moments from the masonry area and from the slender columns reinforcements respectively [12]. $A_c$ is the wall’s compressed area [12].

$$M_{Rd} = M_{Rd(M)} + M_{Rd(As)} \text{ (kNm)} \quad (20)$$

$$A_c = \frac{N_{Ed}}{(0.85 \cdot f_{d}}) \text{ (mm}^2) \quad (21)$$

$$M_{Rd(M)} = N_{Ed} \cdot y_c \text{ (kNm)} \quad (22)$$

In figure 7 $y_c$ is the distance between the compressed masonry area weight center and the wall’s area weight center [12]. $l_s$ is the distance between the slender columns at the wall’s edges. $A_s$ is the slender columns horizontal reinforcement area.

$$M_{Rd(As)} = l_s \cdot A_s \cdot f_{yd} \text{ (kNm)} \quad (23)$$

![Figure 7. Confined masonry wall section.](image)

$A_c$ is the compressed area gravity center. $G$ is the wall section gravity center. $S$ is the earthquake action. $t = 250$ mm is the wall’s width. [12]. $V_{Rd}$ is the masonry wall bearing shear force and $V_{Ed}$ is the horizontal shear force from the seismic loads combination.

$$V_{Rd} = V_{Rd1} + V_{Rd2} \text{ (kN)} \quad (24)$$

$$V_{Rd1} = 0.4 \cdot (N_{Ed} + 0.8 \cdot V_{Ed} \cdot h_{pan}/l_{pan}) \text{ (kN)} \quad (25)$$

$$V_{Ed} \leq l_{pan} \cdot t \cdot f_{vd,0} \quad (26)$$

$$V_{Rd2} = \lambda_c \cdot A_s \cdot f_{yd} \text{ (kN)} \quad (27)$$

where $h_{pan}$ is the story height – 30 cm (tie beam height) and $l_{pan} = l_s - t$ are the height and length of the masonry area panel. $V_{Rd2}$ (27) is the bearing horizontal shear force from the reinforcement in the slender column at walls compressed edge [12]. $A_s$ is the reinforcement area in the slender column at the walls compressed edge. $\lambda_c$ is the reinforcement participation factor. Here, $\lambda_c = 0.25$, for longitudinal reinforcement $\Phi 14$ mm. The load combination used to design the structure is $1.0 \cdot$ permanent loads + $0.4 \cdot$ variable loads + $1.0 \cdot$ seismic loads [14].

4. Elastic analysis results

The beams, tie beams and slender columns dimensions and longitudinal reinforcement are seen in table 1. $A_s$ is the longitudinal reinforcement area [16]. The bars are seen as red discs and the diameter ($\Phi$) of bars (in mm) is written for each element.

| Table 1. Beams, tie beams and slender columns dimensions and reinforcements. |
| --- |
| Beam 25x50 $A_s \rightarrow 2\Phi 22$ and 1 $\Phi 20$ up and down | Tie beam 25x30 $A_s \rightarrow 6\Phi 12$ | Slender column 25x25 $A_s \rightarrow 4\Phi 14$ |
4.1. Concrete walls efforts

The autoclaved walls effect to the existing structure will increase the efforts in the concrete walls. It is important to establish if the concrete walls can withstand the new efforts. In figures 8 to 15 $N_{Ed1}$ and $N_{Ed2}$ (in kN) are the axial forces in the concrete walls in the initial stage 1 and the final stage, after the autoclaved walls story has been built, stage 2. $M_{Ed1}$, $M_{Ed2}$ and $M_{Rd}$ (in kNm) are the bending moments in the concrete walls in stages 1 and 2. The axial forces and bending moments analyzed are from the seismic loads combinations. No AAC wall will be built on top of P9.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig8.png}
\caption{P1 efforts.}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig9.png}
\caption{P2 efforts.}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig10.png}
\caption{P3 efforts.}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig11.png}
\caption{P4 efforts.}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig12.png}
\caption{P5 efforts.}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig13.png}
\caption{P6 efforts.}
\end{figure}
Figures 8 to 15 help compare the piers efforts values. Both axial forces and bending moments values decrease from story 1 to 8 for all piers. The axial forces decrease is faster for shorter walls (P2, P3, P4 and P5), slower for longer walls (P6, P7 and P8) and very slow for the longest wall P1. This can be caused by the walls weights. The bending moments decrease in a linear pattern and $M_{Ed1}$ remains lower than $M_{Ed2}$ for short walls (P2, P3, P4 and P5). The bending moments decrease more rapidly for the other piers. For P6, P7, P8 and P1, the $M_{Ed1}$ values are significantly lower than $M_{Ed2}$ for lower stories. In some cases the bending moments values slightly increase form story 1 to story 2. This is caused by the design formula, more specifically by the great difference between $M_{Rd}$ and $M_{Ed}$ for story 1.

The forces transmitted to the foundation in stage 1 are: base axial force $N_1=45642$ kN, base shear force $V_1=9751$ kN for stage 1 and $N_2=50398$ kN, $V_2=9928$ kN for stage 2. The base bending moments for the load combination with seism on direction X are $M_{1X}=292709$ kNm and $M_{2X}=323517$ kNm. On direction Y the values are $M_{1Y}=292709$ kNm and $M_{2Y}=323517$ kNm. For the load combination with seism on direction Y, $M_{1X}=454846$ kNm and $M_{2X}=490625$ kNm. On direction Y the values are $M_{1Y}=855788$ kNm and $M_{2Y}=949676$ kNm. The axial loads and bending moments transmitted to the foundations increase by approximately 10%. The shear forces increase by about 2%.

### 4.2. Concrete walls compression lengths, stresses and strength

The compression lengths ($l_c$) and axial stresses ($\sigma_{cp}$) reached at the bottom story ($S_1$) increase from stage 1 to stage 2 [16-18]. The $l_c$ appears due to the seismic loads combinations and $\sigma_{cp}$ due to the axial loads for the ultimate limit state combination. $l_c$ should not surpass the maximum compression length $x_u$, thus the walls will have a ductile behavior and $\sigma_{cp}$ values need to remain smaller than $f_{cd}$ thus the concrete will not be crushed at the walls bottoms. Table 2 contains these values for stages 1 and 2 [16].

|        | P1  | P2  | P3  | P4  | P5  | P6  | P7  | P8  |
|--------|-----|-----|-----|-----|-----|-----|-----|-----|
| $l_c$  | 1   | 2   | 954 | 125 | 93  | 127 | 152 | 228 |
| (mm)   |     |     | 1046|     | 93  |     | 127 |     |
| $x_u$  | 1   | 2   | 3876| 577 | 626 | 754 | 928 | 1248|
| (mm)   |     |     | 3956|     | 582 |     | 640 |     |
| $\sigma_{cp}$ | 1  | 1.85| 1.71| 1.73| 1.66| 1.73| 1.  | 1.  |
| (N/mm²)|     | 1.85| 1.71| 1.73| 1.66| 1.73| 1.  | 1.  |
| $f_{cd}$ | 16.67| 16.67| 16.67| 16.67| 16.67| 16.67| 16.67| 16.67|
4.3. Masonry walls efforts, stresses and strength

The autoclaved concrete walls need to bear the efforts they are subjected to, as seen in table 3. The axial stresses at the walls base $\sigma_d$ need to be lower than the vertical masonry strength $f_d$ [12].

|        | $N_{Ed}$ | $M_{Ed}$ | $M_{Rd}$ | $V_{Ed}$ | $V_{Rd}$ | $\sigma_d$ | $f_d$  |
|--------|----------|----------|----------|----------|----------|------------|--------|
| P1     | 402      | 776      | 3831     | 130      | 218      | 0.22       | 2.47   |
| P2     | 58       | 9        | 239      | 2        | 71       | 0.19       | 2.47   |
| P3     | 65       | 26       | 352      | 6        | 76       | 0.21       | 2.47   |
| P4     | 57       | 32       | 409      | 12       | 75       | 0.16       | 2.47   |
| P5     | 85       | 37       | 573      | 12       | 85       | 0.16       | 2.47   |
| P6     | 113      | 71       | 831      | 20       | 80       | 0.22       | 2.47   |
| P7     | 66       | 133      | 945      | 27       | 106      | 0.20       | 2.47   |
| P8     | 138      | 86       | 953      | 38       | 159      | 0.26       | 2.47   |

4.4. Natural periods of vibration

Natural periods of vibration for the first 3 vibration modes: $T_1$, $T_2$ and $T_3$ are seen in table 4. They increase by up to 16% from stage 1 to stage 2.

|        | $T_1$=0.285s | $T_2$=0.220s | $T_3$=0.187s |
|--------|--------------|--------------|--------------|
| Stage 1|              |              |              |
| Stage 2| $T_1$=0.335s | $T_2$=0.258s | $T_3$=0.215s |

5. Plastic analysis results

The building’s behavior in the plastic stage is analyzed by using the pushover diagrams for stages 1 and 2 for both directions $X$ and $Y$. Pushover cases used are PX1, PY1, PX2 and PY2. The pushover diagrams are shown in figure 16. The maximum displacements considered are 700 mm for stage 1 and 800 mm for stage 2.

5.1. Pushover diagrams

For direction $X$, the base force is twice the one reached for direction $Y$ for both stages 1 and 2. The structure is stiffer on $X$. This may be caused by the 2 long walls seen on that direction. The maximum base forces reached in stage 2 are lower than these in stage 1 by 100 MN for direction $X$ and 50 MN for $Y$. This means the added masonry walls affect the existing concrete structure more on direction $X$. 

Figure 16. Pushover diagrams.
In all cases, the structures mostly maintain the same rigidity, as the diagrams are all straight lines. Rigidity is regarded as base force/displacement. The diagrams for direction $X$ do show some waves, thus there are some slight stiffness decreases.

5.2. Plastic hinges development

The final stages analyzed are seen in figures 17 to 20. The plastic hinges in beams and slender columns need to be shown clearly thus only these appear in the figures. The plastic hinges color code is: green means the plastic hinge is formed, blue is for the plastic hinge reaching the limit, pink represents the load being redistributed and red means collapse.

Figures 17 to 20 show more plastic hinges reach collapse on $X$ than on $Y$ when the same top displacement is reached. The top displacement is greater for stage 2, as the building is higher, but the plastic hinges are less developed in this stage. This is because in stage 2, the masonry walls at the top story help to dissipate the seismic energy, as they are flexible compared to the concrete walls. This is in accordance with the pushover curves that show a reduced rigidity for stage 2. It is clear that the plastic hinges at the masonry walls story are less developed. This is explained as the AAC walls are more flexible than concrete ones, thus they can bend easier and not get thus much damage.

Figure 17. PX1 step 182 $d=70$ cm.

Figure 18. PY1 step 233 $d=70$ cm.

Figure 19. PX2 step 195 $d=80$ cm.

Figure 20. PY2 step 198 $d=80$ cm.
5.3. **Walls stresses in the plastic stage**

It is important to study the masonry walls stresses in the plastic stage. Figures 21 to 30 show the stress values at stages when they surpass the masonry strengths. Stresses $\sigma_x$ and $\sigma_z$ from PX2, reach high values at the walls corners. The masonry gives out by being crushed there. The high stresses are distributed more evenly among walls for PX2. Higher stress values are seen in walls on direction $Y$.

Stresses $\sigma_x$ and $\sigma_z$ from PY2 are focused mostly on the 2 long walls on direction X. They reach high values at the walls tops and bottoms. This means the walls are subjected more to bending perpendicular to their plane.

Stresses $\tau_{xz}$ reach high values in walls placed on the same direction as the pushover case. The highest stresses can be found at the walls corners.
Stresses $\tau_{xy}$ clearly surpass the masonry strength at steps 145 and 165. The walls would give in to this stress long after many plastic hinges have been formed. An explanation may be that the walls can bend as the plastic mechanism is formed. Stresses $\tau_{yz}$ reach high values in walls perpendicular to the pushover case direction, because this is a perpendicular stress.

Both stresses reach the highest values in slender walls with high axial loads. This can happen because walls with lower axial forces can bend more perpendicular to their plane and not get that much damage.

6. Conclusions

- The additional story can be built above the existing structure as the old structure can bare it. The design efforts remain below the bearing efforts after the additional story has been built. The compression lengths and axial stresses at the walls bottoms stay below the maximum compression length and the vertical compression strength respectively for both stages 1 and 2. The AAC walls can bare the efforts they are subjected to.

- The new story changes the structures plastic behavior. The new story helps dissipate the seismic energy. The top displacement is greater in stage 2. The building is higher, but the plastic hinges are less developed in this stage. In stage 2, the masonry walls at the top story help dissipate the seismic energy, because the AAC story can bend easier and not get as damaged as the concrete stories. The pushover diagrams show a reduced rigidity for stage 2. The plastic hinges at the masonry walls story are less developed.

- In the plastic stage, stresses $\sigma_x$ and $\sigma_z$ from PX2, reach high values at the walls corners and the masonry is crushed there. High stresses are distributed more evenly among walls for PX2. Stresses $\sigma_x$ and $\sigma_z$ from PY2 are focused mostly on P1. P1 walls are long and situated on direction X. They are subjected mostly to bending perpendicular to their plane. Stresses $\tau_{xz}$ reach high values in piers situated on their direction. Both stresses $\tau_{xy}$ and $\tau_{yz}$ reach the highest stress values in slender walls with high axial loads because walls with lower axial forces can bend perpendicular to their plane without giving in.
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