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Seismic Performance Assessment of Confined Masonry Building

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Abstract

Confined Masonry (CM) structures are composed of masonry walls that are surrounded by RC confining elements, which are called tie beams and tie columns, respectively, in both the vertical and horizontal directions. In this study, different numerical techniques for the analysis of confined masonry and un-reinforced masonry structures are investigated on a full-scale masonry specimen. The model is developed with the program ETABS which offers wide possibilities in finite element method models. Using specific modeling tools of ETABS is intended to simulate the nonlinear behavior of masonry and the global response of the structure. Using the method of modeling layered shell element in a masonry wall, a pushover analysis and the results of both confined and URM building is done. The pushover curve of the CM building exhibits higher seismic performance as compared to the URM building. Due to its greater strength, stiffness, and ductility, the CM building is more resilient to seismic stresses and better able to sustain larger deformations without collapsing. Pushover analyses of CM, URM, and RCC buildings were compared, and it was found that RCC constructions appeared to perform better in resisting lateral loads than CM and URM structures. RCC construction is more ductile and capable of withstanding larger displacements while still maintaining a reasonably high base shear. Confined masonry buildings generally outperform unreinforced masonry buildings across all damaged states and exhibit higher base shear value limits, which implies better seismic resilience and greater capacity to withstand lateral forces during an earthquake.

Keywords

confined masonry, unreinforced masonry, tie elements

1. Introduction

Confined Masonry (CM) structures are made of masonry walls that are surrounded by RC confining elements in both the vertical and horizontal directions (tie-column and tie-beam, respectively), causing all materials to work together to resist action effects. The fundamental distinction between CM buildings and RC infilled frames is that in CM the concrete frame elements are only cast after the masonry walls are constructed, resulting in effective contact between the masonry and the surrounding RC elements due to adhesion and confinement effects. Tie columns are often positioned at all corners and wall intersections, along the vertical margins of door and window openings, and along the vertical edges of wall openings. The reinforcement of these tie columns should be closely coupled to the reinforcement of the horizontal bonding elements. Traditionally, the load-bearing portion of a structure is not represented by reinforced concrete confining elements. Most of the time, experience and the height and size of the building define how much reinforcement they will need. The study of confinement in masonry construction is important as they are effective in improving the stability and integrity of masonry walls for in-plane and out-of-plane earthquake loads (confining members can effectively contain damaged masonry walls), improving the strength (resistance) of masonry walls under lateral earthquake loads, and improving the earthquake performance of masonry walls. Confined Masonry (CM), which is used in numerous seismically sensitive regions all over the world, has developed as a practical and affordable building method. Despite having performed well during previous earthquakes, many seismically active regions, such as Nepal, are hesitant to use confined masonry as a dependable method of building construction due to the lack of studies comparing the behavior of entire building models to RC frame structures, which are more widely used in construction.

2. Objectives

The general objective of carrying out this study is to find out the performance of MRT RCC building, URM building and confined masonry building in terms of storey displacement, drift ratio and base shear by performing pushover analysis.

- To compare the results of confined masonry buildings with URM buildings and MRT RCC buildings.

3. Methodology

Methodology carried out is listed below to meet the objective discussed above:

1. Detail study of various literature available related to this work.
2. Selection of the RCC building constructed using MRT for comparison with CM and URM building.
3. Finite Element Modeling of the sample buildings using ETABS version 18.0.0 software.
4. Seismic coefficient, Response Spectrum and Non-linear static push over analysis was performed in the model.
5. Analysis and interpretation of the results based on different parameters.

3.1 Finite Element Modeling of Building

3.1.1 Configuration of Building

For carrying out this project, the RCC building constructed using MRT is used. A building considered is two-story having a floor height of 3.2m. It contains 3 bays in the Y-direction and 2 bays in the X-direction and also contains a staircase in the 1st bay of the Y-direction. Similar dimension for size of room, height of building, loading, arrangement of window, position of staircase is selected to model both CM and URM buildings.

Table 1: Configuration of MRT building

S.N	Criteria	Dimension/Size
1.	Beam Size	230×350 mm
2.	Column Size	300×300 mm
3.	Slab Thickness	127 mm

Table 2: Configuration of CM building

S.N	Criteria	Dimension/Size
1.	Tie Beam	230×230 mm
2.	Tie Column	230×230 mm
3.	Band	230×110 mm
4.	Slab	100 mm
5.	Masonry Wall	230 mm

Table 3: Configuration of URM building

S.N	Criteria	Dimension/Size
1.	Slab	100 mm
2.	Masonry Wall	230 mm

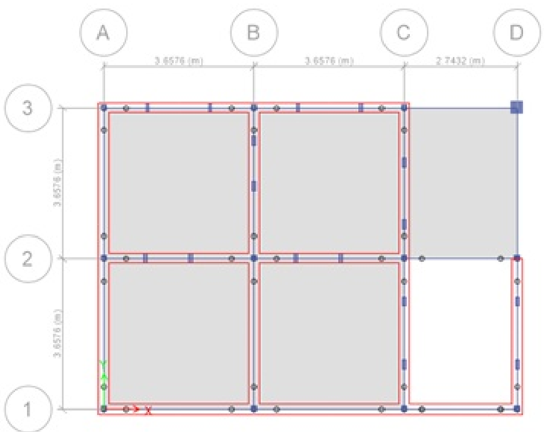

Figure 1: Plan of Building

3.1.2 Lateral Loads

The horizontal seismic base shear is assigned as per NBC105:2020 [1] which is the product of seismic weight and horizontal base shear coefficient. The base shear is then distributed along the height of the building to obtain design lateral force.

The elastic site spectra is given as

$$C(T) = Ch(T) \cdot Z \cdot I \tag{1}$$

where,

Ch(T) = Spectral Shape factor

Z = Zone factor

I = Importance factor

a. Equivalent static method

The horizontal base shear coefficient is given as

For the Ultimate Limit State:

$$Cd(T1) = \frac{C(T1)}{R\mu\Omega_u} \tag{2}$$

For the Serviceability Limit State:

$$Cd(T1) = \frac{Cs(T1)}{\Omega_s} \tag{3}$$

b. Modal Response Spectrum Method

The design base shear coefficient for each mode is given by

$$Cd(Ti) = \frac{C(Ti)}{R\mu \cdot \Omega_u} \tag{4}$$

c. Load Combinations

All building model are designed for load combinations as per NBC105:2020.

$$1.2DL + 1.5LL \tag{5}$$

$$DL + \lambda LL + E \tag{6}$$

$$DL + \lambda LL - E \tag{7}$$

Where, $\lambda = 0.6$ for storage facilities

=0.3 for other usage

3.1.3 Modelling and Analysis of the Building Model

3-D modeling to three models of building is modeled in ETABS V18.0.0 is used for 3D modeling of the buildings. MRT building is modeled as a bare frame where loading is assigned in the frame structure. CM building is modeled with masonry wall as a layered shell element with tie beam, tie column and RCC band with opening as per original building model. In CM building as tie beam and column are modeled as a frame element, so to make their connection pin jointed rather than fixed joint, releasing of moment is done at the joint of tie beam and column. URM building is modeled with masonry wall where only masonry wall is modeled as a layered shell element. The buildings are assumed to be fixed at base and floor act as semi rigid diaphragm. Three model are analyzed with both the equivalent static method (linear static method) and response spectrum method (linear dynamic method). The evaluation focused on parameters such as base shear, time period, top story displacement, and inter-story drift. Additionally, nonlinear pushover analysis was conducted to create a pushover curve.

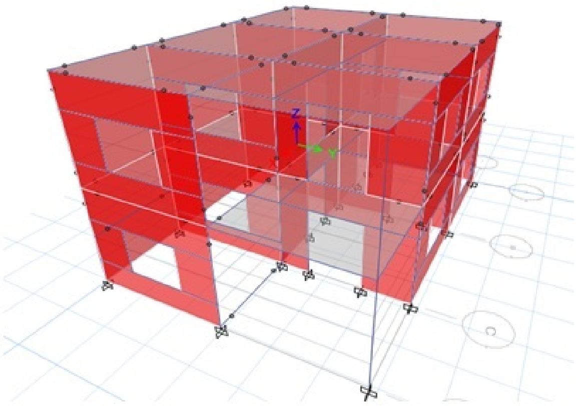

Figure 2: 3D Model of CM Building

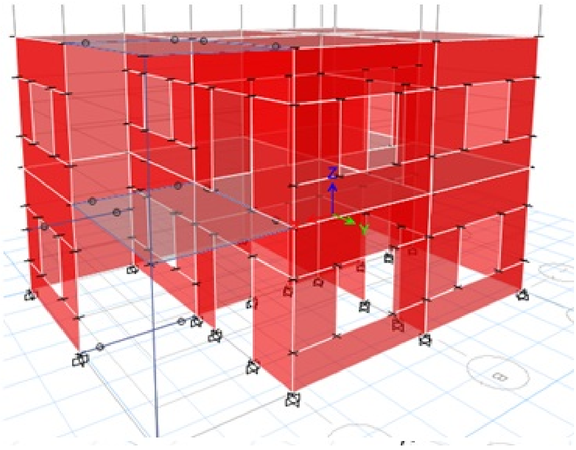

Figure 3: 3D Model of URM Building

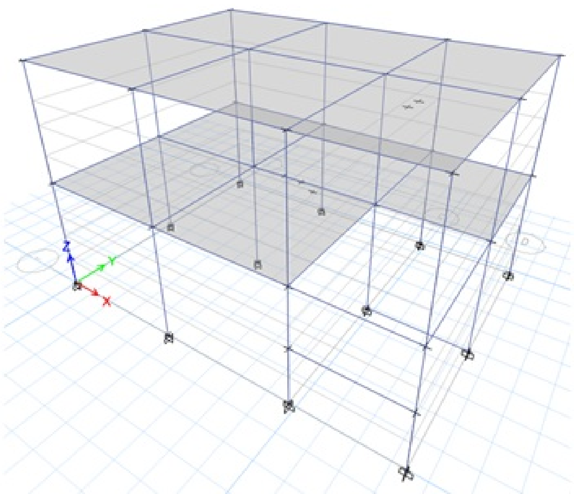

Figure 4: 3D Model of RCC Building

Masonry walls are modelled using a layered technique with a thickness of 230 mm in ETABS, and other parameter defined is shown in the table 4.

In confined masonry, modeling is done for masonry, tie-beam, tie-column, and RC band for openings in building. Tie beam and Tie column are defined as frame property in ETABS. Frame release is carried out in ETABS to release moment, both major and minor moment which is shown in Figure 2 . RC Band in confined masonry building is modeled in ETABS using section modifier.

Table 4: Masonry wall layered property definition

Property	Assigned Parameter
Thickness	230mm
Modeling Type	Shell
Material Behavior	Directional
Material S11	Non-Linear
Material S22	Non-Linear
Material S12	Non-Linear

3.1.4 Pushover Analysis of Masonry Building

Pushover analysis is a nonlinear static analysis method that simulates the inelastic behavior of a structure subjected to increasing lateral loads. It is commonly used to evaluate the seismic performance of buildings, including masonry buildings. To perform a pushover analysis of a masonry building, the building is first modeled using a finite element analysis software. Then, a series of lateral loads are applied to the model in a step-by-step manner, and the behavior of the structure is analyzed at each load level. The results of the pushover analysis are typically presented in the form of a pushover curve, which shows the relationship between the applied lateral load and the resulting deformation, or displacement, of the structure. The pushover curve can be used to evaluate the seismic performance of the building, including its strength and stiffness, as well as the locations where the structure is likely to experience the greatest damage during an earthquake. In the context of masonry buildings, pushover analysis can help identify potential vulnerabilities and suggest possible retrofit strategies to improve the building's seismic performance. It can also be used to compare the seismic performance of different building configurations or retrofit options and help guide decision-making in the design process.

a. Material Properties

The material properties used by [2] for confined masonry are used to carry out FEM for detailed analysis. The material properties used for this model are tabulated below from the calculation. Compressive strength of masonry is calculated using $f_m = 0.422 f_b^{0.69} f_{mo}^{0.252}$

Table 5: Masonry Properties for FEM Modeling

Masonry Parameter	Value
Brick Strength (f_b)	6 MPa
Mortar Strength (f_{mo})	3 MPa
Compressive Strength of Masonry Prism (f_m)	1.91633 MPa
Modulus of Elasticity of Masonry (E_m)	2107.963 MPa
Poisson's Ratio	0.2

b. Non-linear material properties in Compression

The compressive stress-strain plot of the masonry wall was plotted using the data obtained from [3] based on the compressive strength and strain at the maximum stress. This stress-strain value for masonry is given input in FEM software to define it's non-linear material data.

Table 6: Control points on stress–strain curves of masonry [3]

Stress in terms of f_m	Stress (MPa)	Strain
0	0.000	0
0.33	0.632	0.0009
0.75	1.437	0.0021
0.9	1.725	0.0029
1	1.916	0.0036
0.6	1.150	0.0059
0.5	0.958	-
0.2	0.383	-

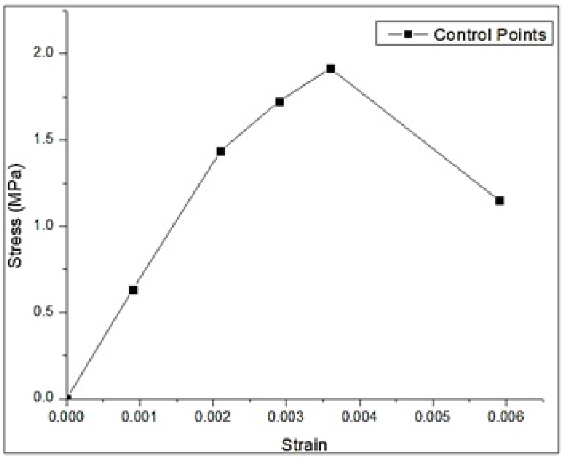

Figure 5: Stress–strain curves of masonry for 1:0:6 mortar strength [3]

4. Result and Discussion

4.1 Modal Analysis

From the comparison, we can observe that the time periods (or natural periods of vibration) vary for each mode and for each type of building. In general, the time periods for confined masonry buildings are longer than those for unreinforced masonry buildings, and both are shorter than the time periods for RC buildings. This suggests that RC buildings tend to have a stiffer and more rigid response to vibrations compared to confined masonry and unreinforced masonry buildings.

Table 7: CM Building Mode Data

Mode	Time Period	Ux	Uy	Rz
1	0.06	0.3236	0.4095	0.1543
2	0.053	0.8207	0.8106	0.1563
3	0.037	0.9005	0.8965	0.9173
4	0.023	0.935	0.929	0.9283
5	0.021	0.9735	0.9717	0.9283
6	0.016	0.9738	0.9729	0.934
7	0.015	0.9797	0.9795	0.9756
8	0.013	0.9799	0.9796	0.9756
9	0.01	0.9799	0.9796	0.9759
10	0.01	0.9799	0.9796	0.9759
11	0.01	0.98	0.9796	0.976
12	0.009	0.98	0.9799	0.976

Table 8: URM Building Mode Data

Mode	Time Period	Ux	Uy	Rz
1	0.011	0.3092	0.271	0.2073
2	0.009	0.6487	0.6825	0.2105
3	0.007	0.6972	0.6996	0.2402
4	0.006	0.7439	0.8309	0.3344
5	0.006	0.7458	0.8331	0.6274
6	0.005	0.8591	0.8615	0.7434
7	0.005	0.8596	0.8619	0.7478
8	0.005	0.8830	0.8967	0.7532
9	0.005	0.8833	0.9073	0.7593
10	0.005	0.8984	0.9080	0.7672
11	0.005	0.8998	0.9258	0.8027
12	0.004	0.9100	0.9263	0.8039

Table 9: RC Building Mode Data

Mode	Time Period	Ux	Uy	Rz
1	0.484	0.3236	0.4095	0.1543
2	0.454	0.8207	0.8106	0.1563
3	0.377	0.9005	0.8965	0.9173
4	0.180	0.935	0.929	0.9283
5	0.173	0.9735	0.9717	0.9283
6	0.142	0.9738	0.9729	0.934
7	0.011	0.9797	0.9795	0.9756
8	0.010	0.9799	0.9796	0.9756
9	0.009	0.9799	0.9796	0.9759
10	0.008	0.9799	0.9796	0.9759
11	0.007	0.9800	0.9796	0.976
12	0.006	0.9800	0.9799	0.976

4.2 Maximum Storey Displacement

As shown in Table 10, the maximum storey displacements in CM are relatively consistent across both analysis methods. The maximum storey displacements in URM are significantly smaller compared to CM and RCC structures. In general, the maximum storey displacement values for RCC are significantly higher than those for CM and URM. RCC Building exhibits larger top storey displacements, indicating potentially higher structural deformations under the applied loads. Moreover, the maximum storey displacement values are generally higher in the Y direction than the X direction for all building types and analysis methods. These results indicate that the Response Spectrum Method predicts higher top storey displacement values as compared to the Equivalent Static Method for all three building types. This is because the Response Spectrum Method takes into account the dynamic characteristics of the building, while the Equivalent Static Method assumes that the building responds statically to the applied loads.

Table 10: Maximum Storey Displacement

Building Type	Method of Analysis	Top Storey Displacement	
		X	Y
CM	ESM	0.383	0.413
	RSM	0.4506	0.529
URM	ESM	0.0167	0.018
	RSM	0.0297	0.0352
RCC	ESM	9.594	10.99
	RSM	10.2465	12.1321

*ESM = *Equivalent Static Method*
*RSM = *Response Spectrum Method*

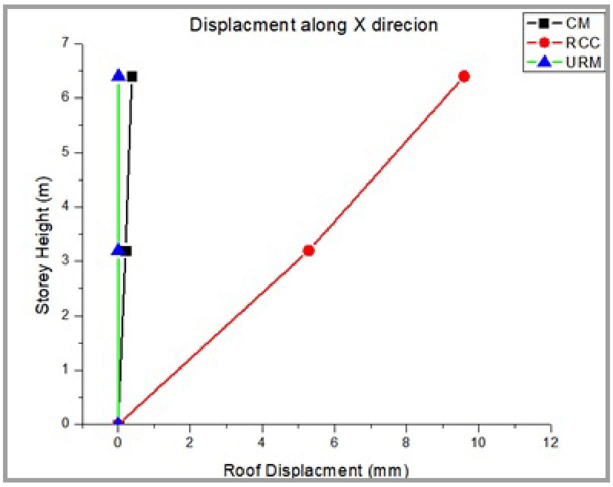

Figure 6: : Displacement along X direction (CM, RCC and URM building)

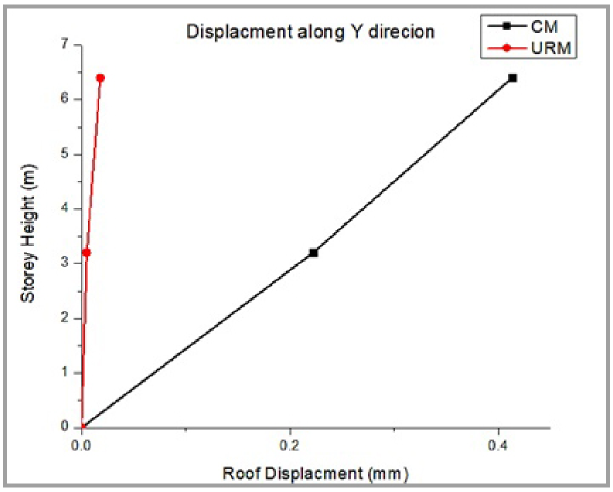

Figure 9: Displacement along Y direction (CM and URM building)

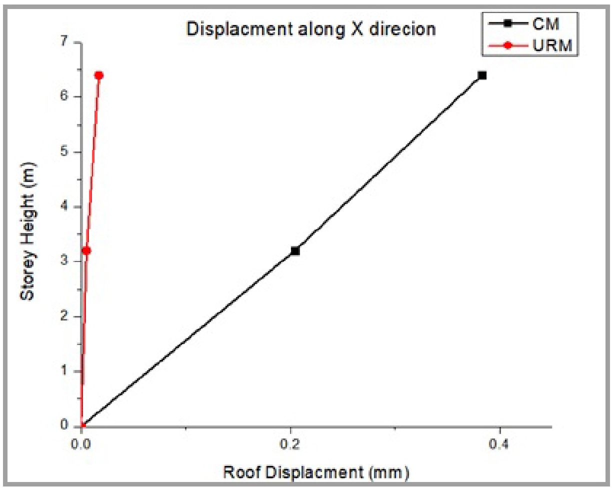

Figure 7: Displacement along X direction (CM and URM building)

In the case of confined masonry, the presence of confinement in the masonry walls of the structure contributes to its overall ductility and energy dissipation capacity. The gradual increase in displacements with height suggests a more controlled response under lateral loads. In the case of URM buildings, smaller displacements indicate a more brittle behavior. Without reinforcement or confinement, the structure is susceptible to rapid strength degradation and potential failure.

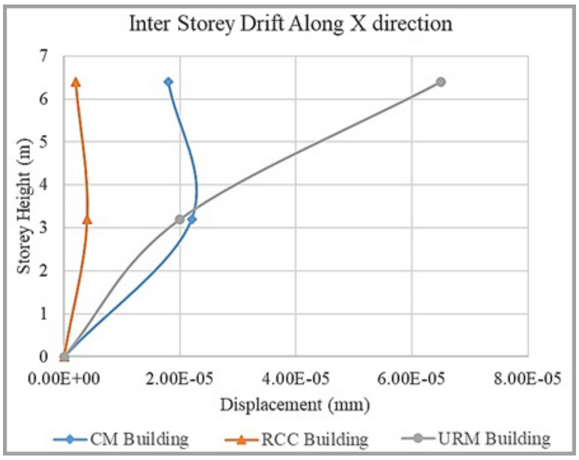

Figure 10: Inter Storey Drift along X-direction

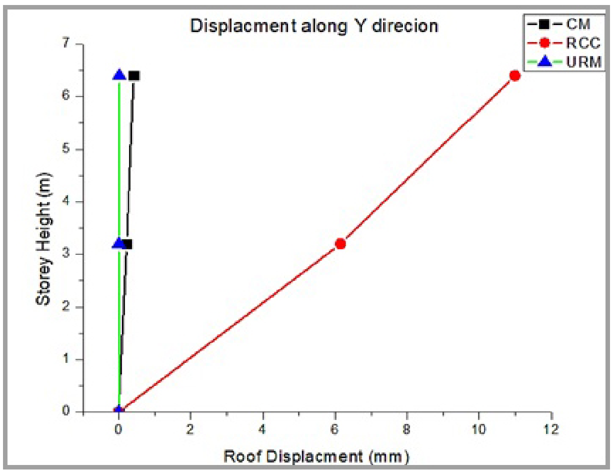

Figure 8: Displacement along Y direction (CM, RCC and URM building)

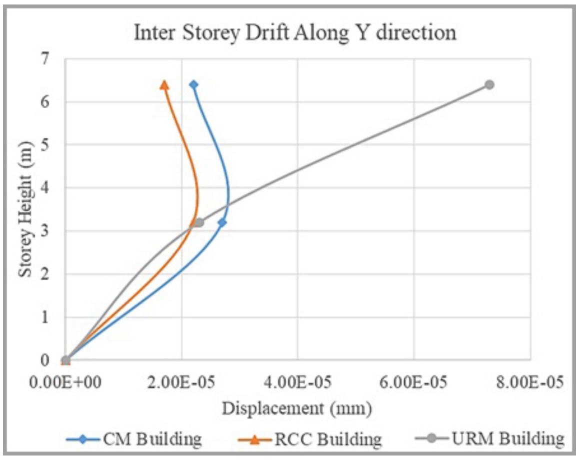

Figure 11: Inter Storey Drift along Y-direction

In the case of the RCC Building, the significantly larger displacements in the RCC building indicate its ability to undergo larger deformations while maintaining overall stability. The presence of reinforced concrete elements provides enhanced strength and ductility, allowing the structure to absorb and dissipate energy during lateral loading.

The inter-story drift for different buildings is shown in the figure above. Three model of RCC, CM and URM buildings, the interstorey drifts are less than the allowable value permitted by NBC 105:2020 [1] for ULS and SLS. The maximum interstorey drift is found in the second story. In both X and Y directions, inter storey drift in case of the URM building is higher than that of the CM and RCC building.

4.3 Pushover Curve

As shown in Figure 12, the maximum base shear for the CM building is 2592.7485 KN, whereas, for the URM building, it is 1278.727 kN. This indicates that the CM building is stiffer and more resistant to lateral forces than the URM building. The roof displacement at which the maximum base shear occurs is approximately 10.25 mm for the CM building and 4.777 mm for the URM building. This means that the CM building can undergo larger deformations before reaching its maximum resistance. Both buildings display a linear response up to a certain point, after which the response becomes nonlinear. This is indicated by the change in the slope of the curves. The URM building experiences a sudden drop in base shear after a displacement of 3.624 mm, indicating that it may be more susceptible to sudden failure than the CM building.

The pushover curve for the CM building indicates better seismic performance compared to the URM building. The higher strength, stiffness, and ductility of the CM building make it more resistant to seismic forces and more capable of sustaining larger deformations without collapsing.

On carrying out comparison of CM, URM and RC building from figure 14, and figure 15, looking at the results, it can be observed that the RCC structure has the highest base shear values compared to the other two structures for all the monitor displacement levels. This indicates that RCC structure is more stable and able to withstand higher lateral loads as compared to CM and URM.

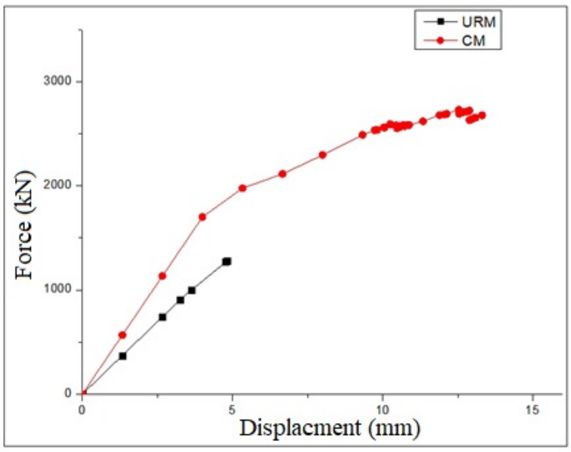

Figure 12: Pushover Curve along X-direction(CM and URM)

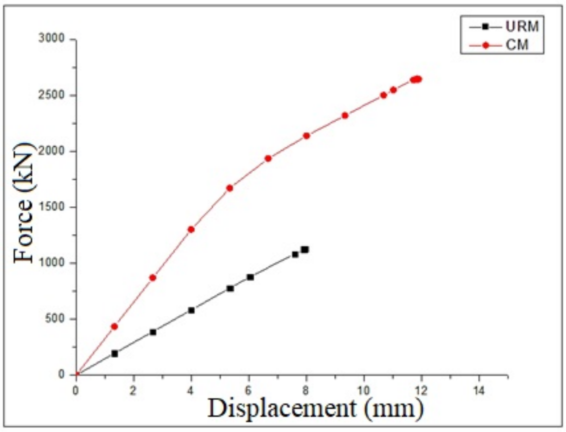

Figure 13: Pushover Curve along Y-direction(CM and URM)

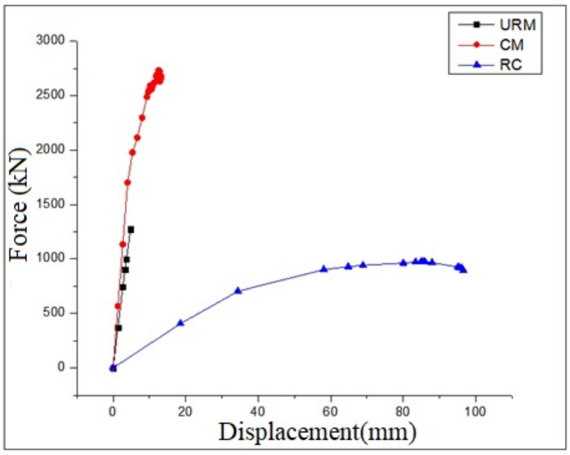

Figure 14: Pushover Curve along X-direction(CM, URM and RCC)

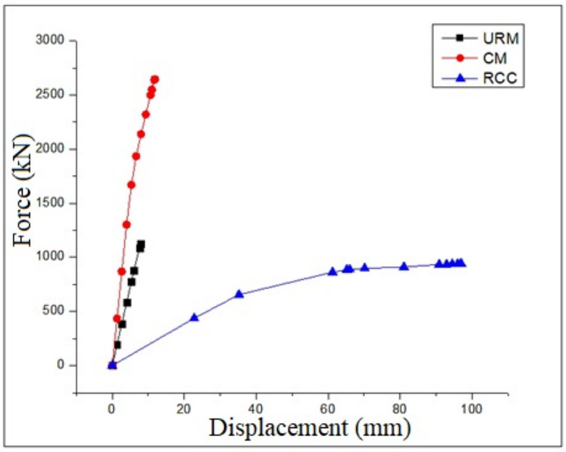

Figure 15: Pushover Curve along Y-direction (CM, URM and RCC)

On the other hand, the URM structure shows the lowest base shear values for all monitor displacement levels, indicating its poor performance in resisting lateral loads. CM structure lies between RCC and URM structures in terms of base shear values.

Therefore, these results cannot be generalized to all CM, URM, and RCC structures. However, based on the given data, RCC structures appear to perform better in resisting lateral loads compared to CM and URM structures.

Pushover Curve of Unreinforced Masonry Building

From figure 12 and figure 13 of the pushover curve of the URM building and from the HAZUS manual [4] , yield displacement (dy) = 4.593mm and ultimate displacement (du) = 5.09499mm.

Table 11: Damage Limit States for URM [5]

Damage States	Limit (mm)
Slight Damage	$0.7 \cdot dy$
Moderate Damage	$1.05 \cdot dy$
Extensive Damage	$0.5 \cdot (dy + du)$
Complete Damage	du

Table 12: Base Shear for various damage limit states [5]

Damage States	Limit for URM (mm)	Base Shear Value (kN)
Slight Damage	3.2151	875.968047
Moderate Damage	4.82265	1313.95207
Extensive Damage	4.843995	1319.76761
Complete Damage	5.09499	1388.12777

Pushover Curve of Confined Masonry Building

The pushover curve developed for confined masonry buildings compared with the backbone curve developed by [6] . From figure 14 and figure 15 , the yield point and ultimate point of CM building are near the result of the experiment. Experimental result of confined masonry with RC tie column and tie beam characterizes elastic limit, slight damage, moderate damage, serious damage, and collapse, respectively.

Table 13: Damage Limit States for CM (Lihong and Qiushen 2008 October)

Limit State	U/Umax
Elastic Limit	0.2
Slight Damage	0.35
Moderate Damage	0.6
Serious Damage	1
Collapse	1.3

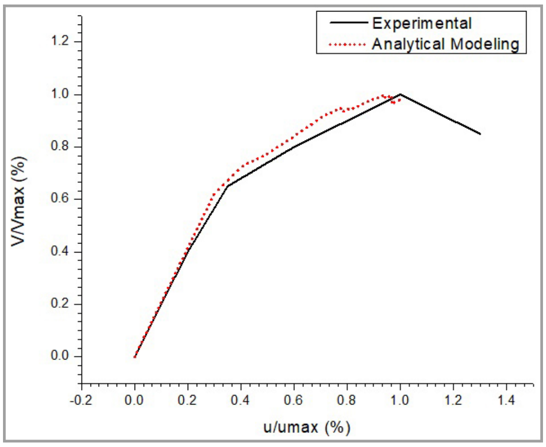

Figure 16: Comparison of base shear coefficient vs drift of model with the experimental result

From table 12 and figure 16 , the following results for the damage limit state level of the model.

Table 14: Base shear value of CM building at different damage limit state of model

Limit State	V/Vmax	Base Shear (kN)
Elastic Limit	0.41531	1133.81
Slight Damage	0.67294	1837.15
Moderate Damage	0.84098	2295.9
Serious Damage	0.98062	2677.13
Collapse	0.85	2320.52

Comparison of Pushover Curve of CM and URM building

1. Slight Damage:

Confined Masonry: Base Shear value limit = 1837.145715 kN
Unreinforced Masonry: Base Shear value limit = 875.968047 kN

- Discussion: The Base Shear value limit for confined masonry is significantly higher than that for unreinforced masonry at the slight damage state. This indicates that confined masonry buildings can withstand more lateral forces before experiencing slight damage compared to unreinforced masonry buildings.

2. Moderate Damage:

Confined Masonry: Base Shear value limit = 2295.902518 kN
Unreinforced Masonry: Base Shear value limit = 1313.95207 kN

• Discussion: Again, the Base Shear value limit for confined masonry is noticeably higher than that for unreinforced masonry at the moderate damage state. This suggests that confined masonry buildings have a better ability to resist lateral forces and maintain their structural integrity under moderate damage conditions compared to unreinforced masonry buildings.

3. Serious Damage:

Confined Masonry: Base Shear value limit = 2677.126498 kN
Unreinforced Masonry: Base Shear value limit = 1319.76761 kN

- Discussion: The Base Shear value limit for confined masonry is significantly higher than that for unreinforced masonry at the serious damage state as well. This demonstrates that confined masonry buildings can endure much larger lateral forces before reaching a state of serious damage compared to unreinforced masonry buildings.

4. Collapse:

Confined Masonry: Base Shear value limit = 2320.52465 kN
Unreinforced Masonry: Base Shear value limit = 1388.12777 kN

- Discussion: Even at the collapse state, the Base Shear value limit for confined masonry remains higher than that for unreinforced masonry. This indicates that confined masonry buildings have a better chance of resisting complete collapse under extreme lateral forces compared to unreinforced masonry buildings.

Performance Point

Table 15: Performance Points and Base Shear for CM (Along X and Y)

CM (Along X)		CM (Along Y)	
Performance Point (mm)	**Base Shear (kN)**	**Performance Point (mm)**	**Base Shear (kN)**
1.572	668.327	2.174	708.044

Table 16: Performance Points and Base Shear for URM (Along X and Y)

URM (Along X)		URM (Along Y)	
Performance Point (mm)	**Base Shear (kN)**	**Performance Point (mm)**	**Base Shear (kN)**
2.153	369.986	3.173	462.401

Table 17: Performance Points and Base Shear for RCC (Along X and Y)

RCC (Along X)		RCC (Along Y)	
Performance Point (mm)	**Base Shear (kN)**	**Performance Point (mm)**	**Base Shear (kN)**
24.896	525.0612	27.456	519.4505

The performance point was obtained for the given seismic demand of earthquake as per response spectra of soil type C and seismic zone factor 0.3 given in NBC 105:2020 [1] from ETABS based on FEMA 440 EL [7] method. The performance point and base shear for both models are shown in table. As per the data of pushover curve along the x- axis, following result is obtained.

1) CM Structure:

The CM structure exhibits a higher base shear value (708.044 kN) at a lower displacement (2.174 mm) compared to the URM structure. This indicates that the CM structure is stiffer and able to withstand higher lateral loads before reaching the same displacement as the URM structure.

2) URM Structure:

The URM structure has a lower base shear value (462.401 kN) at a higher displacement (3.173 mm) compared to the CM structure. This suggests that the URM structure is less stiff and exhibits lower resistance to lateral loads, resulting in a lower base shear.

3) RC Structure:

The RC structure has a significantly higher displacement (27.456 mm) compared to both the CM and URM structures. However, the base shear value (519.4505 kN) at this displacement is closer to the CM structure. This indicates that the RC structure is more ductile and capable of sustaining larger displacements while still maintaining a relatively high base shear.

The URM structure shows a higher performance point and a lower base shear, indicating its relatively lower stiffness and performance. The RC structure has the highest performance point among the three, indicating its ability to undergo larger deformations but with a moderate base shear value.

5. Conclusion

The seismic performance of low-rise CM buildings seems to be adequate even under high levels of seismic intensity. This is mainly due to the confined and modular arrangement of masonry walls, which are bounded by vertical tie-columns and horizontal bond beams. This confined structural arrangement prevents wall-to-wall propagation of seismic damage and also enhances the energy dissipation characteristics of the structure.

1. In case of story displacement, CM building is intermediate to RCC and URM building
2. In both X and Y directions, interstorey drift in the case of URM building is higher than that of the CM and RCC building.
3. When compared to the URM building, the CM building's pushover curve shows higher seismic performance. The CM building is more resistant to seismic pressures and better able to withstand bigger deformations without collapsing because to its higher strength, stiffness, and ductility.
4. On carrying out comparison between pushover analysis of CM, URM and RCC building, RCC structures appear to perform better in resisting lateral loads compared to CM and URM structures.
5. CM structure is stiffer and able to withstand higher lateral loads before reaching the same displacement as the URM structure and RCC structure is more ductile and capable of sustaining larger displacements while still maintaining a relatively high base shear.
6. Confined masonry buildings generally outperform unreinforced masonry buildings across all damaged states. Confined masonry exhibits higher base shear value limits, which implies better seismic resilience and greater capacity to withstand lateral forces during an earthquake.

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