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SEISMIC DESIGN OF MASONRY BUILDINGS

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Abstract

This study examines the seismic design of masonry buildings, focusing on the limitations of traditional linear analysis methods and the need for more accurate modeling. Linear static and dynamic analyses, commonly used for design, often overestimate safety checks when ground accelerations exceed 0.05g, as the behavior factor (q-factor) tends to underrepresent the seismic capacity of masonry structures. To address this, the inclusion of the Overstrength Ratio (OSR) is proposed. The OSR accounts for additional strength due to force redistribution after initial failure, providing a more realistic assessment of a building's ultimate capacity. Nonlinear static analyses, such as pushover analysis, offer better alignment with real-world observations, showing improved results compared to linear methods. By integrating the OSR, this revised approach enhances the accuracy and safety of seismic design for masonry buildings, minimizing discrepancies in previous design methods and offering a more reliable framework for assessing seismic performance.

Keywords: *Seismic performance, dynamic analysis, building safety, force redistribution, earthquake engineering.*

1. Introduction

The seismic design of masonry buildings plays a crucial role in ensuring their safety and stability during earthquakes [1]. Traditional analytical methods, such as linear static and dynamic analyses, are widely used in practice for evaluating the seismic performance of these structures. However, these linear approaches often encounter limitations, particularly when ground accelerations exceed 0.05g. In such cases, the safety checks can be overestimated, leading to potentially unsafe design assumptions. One key issue is the behavior factor (q-factor), which tends to underestimate the seismic capacity of masonry buildings, failing to account for the complex behavior of these structures under seismic loads. To improve the accuracy of seismic design, this study proposes the inclusion of the Overstrength Ratio (OSR), which incorporates the additional strength derived from force redistribution after initial failure. The OSR helps provide a more realistic assessment of a building's ultimate capacity [2]. Moreover, nonlinear static analyses, such as pushover analysis, offer results that more closely align with real-world behavior, offering a more reliable and accurate framework for seismic design.

2. Literature Review

The seismic design of masonry buildings is a critical aspect of structural engineering, focusing on ensuring their stability and safety during seismic events. Traditional methods, such as linear static and dynamic analyses, are commonly used to evaluate seismic performance. However, these approaches often fall short in accurately capturing the complex behavior of masonry structures under strong ground motions. This literature review examines the limitations of conventional design methods, explores advanced techniques such as nonlinear static

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analyses, and highlights the role of factors like the Overstrength Ratio (OSR) in improving seismic safety and performance.

Summary of Literature Review

Author's	Work Done	Findings
Shah, A. S. (2024)	Seismic response of masonry structures considering overstrength ratio.	Emphasized the importance of incorporating the overstrength ratio (OSR) for a more accurate seismic response of masonry buildings.
Verma, A. (2023)	Application of nonlinear static analysis in seismic design of masonry buildings.	Highlighted the advantages of using nonlinear static analysis for more accurate modeling of masonry building behavior during seismic events.
Gupta, R. (2022)	Influence of overstrength ratio in seismic design codes for masonry buildings.	Discussed how OSR can lead to safer design by improving the prediction of building response to seismic loads.
Sharma, A. (2021)	Assessment of masonry building performance under seismic loads using pushover analysis.	Found that pushover analysis aligns more closely with observed seismic performance compared to linear methods.
Khurana, M. (2020)	Comparative study of linear and nonlinear seismic analysis of masonry buildings.	Concluded that nonlinear analyses provide more reliable results and better reflect real-world seismic behavior.
Banerjee, S. (2020)	Overstrength ratio and its impact on seismic safety of masonry structures.	Identified that the inclusion of OSR improves the safety assessment by accounting for force redistribution post-failure.
Jain, V. K. (2019)	Seismic evaluation of masonry buildings using dynamic analysis methods.	Found dynamic methods to be more accurate in predicting seismic performance compared to static approaches.
Desai, M. (2018)	Limitations of linear analysis methods in seismic design of masonry buildings.	Demonstrated that linear methods overestimate safety when ground accelerations exceed 0.05g.
Gupta, S. (2018)	Seismic response of masonry structures: A comparative study between linear and nonlinear models.	Found that nonlinear models better replicate the actual seismic response of masonry buildings.
Singh, P. (2017)	Performance of masonry buildings under seismic loading: Linear vs nonlinear analysis.	Showed that nonlinear analysis offers better insight into the behavior of masonry structures under seismic stress.
Agarwal, S. (2017)	The importance of behavior factors in seismic analysis of masonry buildings.	Highlighted how the behavior factor (q-factor) often underestimates the seismic capacity of masonry buildings.

Research Gap

A significant research gap in the seismic design of masonry buildings lies in the limitations of traditional linear analysis methods, particularly when ground accelerations exceed 0.05g. These methods tend to overestimate safety checks and fail to accurately represent the seismic capacity of masonry structures, primarily due to the underestimation of the behavior factor (q-factor). Additionally, the need for incorporating more realistic design parameters, such as the Overstrength Ratio (OSR), and utilizing nonlinear static analyses like pushover analysis, remains underexplored in current design practices.

3. Methodology

The seismic design of masonry buildings is primarily based on linear static, linear dynamic, nonlinear static (pushover), and nonlinear dynamic analyses [3]. Common approaches include elastic linear analysis, which involves two performance levels: the Ultimate Limit State (ULS) for strength verification and the Damage Limit State (DLS) for deformation demands. While the ULS ensures that structural elements do not exceed strength limits, the DLS focuses on controlling drift. Nonlinear static analysis (pushover) evaluates the building's capacity by applying vertical loads and horizontal forces, resulting in a capacity curve that plots displacement versus base shear. Despite being widely used, linear analysis methods often show inconsistencies, especially when peak ground accelerations exceed 0.05g, as they overestimate safety checks. The behavior factor (q-factor), used in linear analyses, often underestimates the building’s actual seismic performance, necessitating the introduction of the overstrength ratio (OSR) for a more accurate representation of the building's seismic capacity [4]. The overstrength ratio is integral in determining the true ultimate capacity of masonry buildings.

4. Result & Discussion

Current Approach to Seismic Design of Masonry Buildings

Modern seismic design codes identify four primary structural analysis methods: linear static (or simplified modal), linear dynamic (typically multimodal with response spectrum), nonlinear static (“pushover”), and nonlinear dynamic analysis [5]. For contemporary building designs, structural elements—such as wall slenderness limits and connections—are essential in preventing out-of-plane collapse. The in-plane response of walls is assessed using global analysis methods. Commonly employed methods in practice include elastic linear (static or dynamic) analysis and nonlinear static approaches, which rely on storey mechanisms or equivalent frame and macro-element idealizations. In linear elastic analysis, the safety verification generally involves two performance levels: no collapse (Ultimate Limit State, ULS) and damage control (Damage Limit State, DLS). At the ULS, the design check focuses on strength verification, while at the DLS, the check is based on deformation (drift) demands [6]. The seismic action is determined from an elastic acceleration response spectrum, scaled by a seismic force reduction factor (or behavior factor, q-factor), which approximates the inelastic response at the ultimate state. This action is applied to a linear elastic model of the structure, with resulting internal forces and displacements computed. For masonry structures, the ULS check generally governs over the DLS check. The ULS verification ensures that the design resistance of each structural element does not exceed the strength limits defined by the relevant codes. If the shear strength or flexural strength of even a single element is exceeded, the ULS requirement is not met. The nonlinear static (or "pushover") analysis involves applying vertical gravitational loads and a horizontal force distribution to the building's structural model. This force distribution, which maintains a constant ratio between acting forces, is scaled to increase the horizontal displacement of a control point (e.g., the center of mass of the roof) until ultimate conditions are reached. The result of this analysis is typically presented as a "capacity curve," with the control point displacement plotted on the x-axis and the base shear on the y-axis [7]. Masonry buildings are often modeled as three-dimensional equivalent frames, with walls, ring beams, and masonry spandrels treated as beam-column elements located at the centroids of the structural components. The walls and horizontal elements are assumed to exhibit elastic-plastic behavior with limited deformation, typically expressed in terms of chord rotation or drift. These elements follow linear elastic behavior until one of the failure criteria—flexure or shear—is met. This approach approximates the experimental resistance envelope under cyclic loading conditions.

Inconsistencies in Linear Analyses of Masonry Buildings

Several inconsistencies were observed in the results of linear analysis methods applied to typical masonry building configurations when following standard force-based procedures [8]. For instance, using a q-factor of 1.5-2.0, as recommended by some seismic codes, it was nearly impossible to meet strength safety checks for unreinforced two- or three-story masonry buildings when peak ground acceleration (agS) exceeded 0.1g. In many cases, safety checks failed even at lower levels of agS (greater than 0.05g). The results of safety checks from elastic analyses were found to be overly conservative and contradicted experimental evidence and nonlinear analyses. In contrast, nonlinear static analyses provided results that were more consistent with real-world observations. Additionally, elastic analysis results often conflicted with provisions for "simple buildings" (regular buildings with basic geometric and construction features that do not require detailed safety verification). Nonlinear procedures, however, provided results more in line with these provisions [9]. It was concluded that the inconsistencies were not caused by the definition of seismic action or the need for deformation or dissipation capacity reserves in the nonlinear range, but rather by the design seismic action used in elastic analyses, specifically the behavior factor, q.

The Role of the Overstrength Ratio in the Definition of the q-Factor

Given these inconsistencies, it became necessary to reconsider the criteria for defining the behavior factor q. The main observation was that, according to linear elastic analyses, the "ultimate" limit state is reached when a single wall of the building reaches its flexural or shear strength. This state, referred to as Fel, does not represent the true ultimate capacity of the building. Unreinforced masonry elements provide limited deformation capacity in the nonlinear range, allowing the building to withstand increasing seismic loads beyond the "elastic" limit (Fel), with forces being redistributed to other structural elements [10]. The actual ultimate strength capacity is reached at higher values of base shear than Fel. Therefore, for masonry buildings, it is evident that the behavior factor q should include an overstrength ratio (OSR), as is done for other structural systems like reinforced concrete and steel. The behavior factor q should be redefined to account for this overstrength ratio.

$$q = \frac{\bar{F}_{el,max}}{F_{el}} = \frac{F_{el,max}}{F_y} \cdot \frac{F_y}{F_{el}} = q_0 \cdot \frac{F_y}{F_{el}} = q_0 \cdot OSR$$

Where q0 is the basic value that takes into account the dissipative capacity of the structure multiplied by the overstrength ratio OSR=Fy/Fel .

The ultimate capacity of a masonry building is reached when the structural system attains its displacement capacity. The ultimate base shear associated with this point can be significantly higher than the base shear corresponding to the attainment of the strength capacity in the first wall of the building [11]. When the first failure occurs, forces can be redistributed through the other structural elements, allowing the building to maintain its global resistance. In practice, the ultimate capacity of a masonry building is usually reached when any wall attains its ultimate displacement capacity. The overstrength ratio (OSR) can be evaluated through nonlinear numerical simulations and experimental testing on building models. The OSR is expected to depend on various factors, some of which are related to modeling assumptions. Therefore, a reliable evaluation of the OSR cannot solely rely on experimental results. Numerical evaluations of the OSR were carried out for different unreinforced masonry building configurations, and the results were used as a reference for the introduction of new q-factor values in the latest Italian seismic codes. The OSR was found to vary widely (from 1.2 to 3.8) for two- and three-story unreinforced masonry buildings with reinforced concrete (r.c.) ring beams.

Drawbacks of Linear Methods Based on the q-Factor

The recognition of the need for the OSR in masonry design, introduced in the revised version of seismic codes, was an important step toward resolving the inconsistencies observed in the application of previous code versions. However, choosing a specific value for the OSR, even for a homogeneous typology of masonry buildings, does not fully address the inherent issues of linear analysis methods. For example, for two- and three-story buildings with OSRs as shown earlier, selecting a single conservative value—whether the minimum or a "sufficiently conservative" percentile (such as 1.8, as suggested in certain codes)—results in a design seismic action that may be too high for the majority of cases where the OSR is much higher (e.g., 2.5 or 3). This conservative approach may lead to unnecessarily high seismic forces, making it impossible to meet strength safety checks, even when the building's materials, configuration, and details indicate that the design should be safe [12]. One potential solution to these challenges is applying a redistribution of internal forces after the linear analysis. This approach is particularly effective when only a few walls fail to meet the design criteria. However, it is essential to define the limits of force redistribution and understand how the overstrength ratio relates to this redistribution. To better address these issues, a detailed comparison between the distribution of internal forces at the elastic limit and the ultimate limit state was conducted.

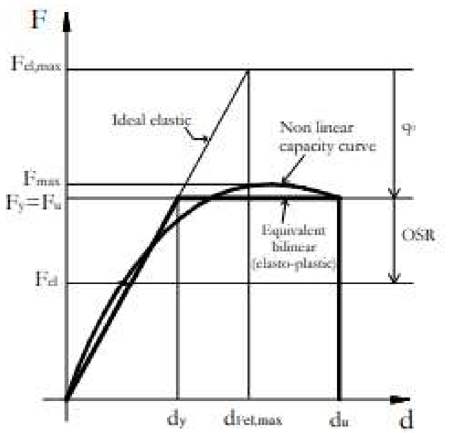

Figure 1 Definition of the q-factor and of the over strength ratio (OSR)

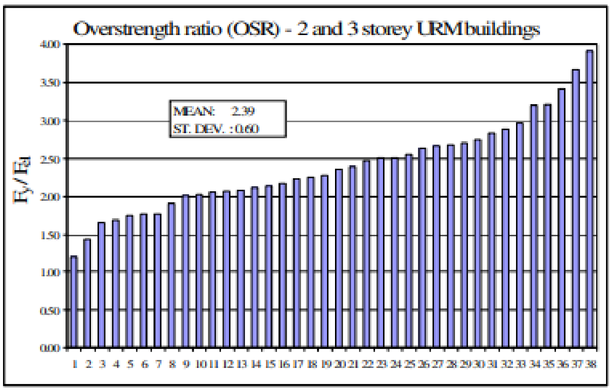

Figure 2 Calculated OSR values for 2 and 3 storey URM buildings with flexurally coupled model

Comparison Of Internal Forces At The Elastic Limit And Ultimate Limit State

This section focuses on the results obtained from analyzing plane structural walls and simple buildings modeled using either the cantilever (pier) method or the equivalent frame method. The comparison is made in terms of internal forces for each wall of the structure, specifically examining shear force, axial load, and moment distributions. The comparison is conducted using a linear elastic model at the elastic limit state (Fel) and at the ultimate base shear (Fy), as well as a nonlinear static model at the ultimate limit state [13]. It is important to note that the elastic limit state occurs when the first wall of the building, in the direction of the seismic action, fails due to shear or flexure. On the other hand, the ultimate limit state is reached when the structure's capacity curve shows a 20% force degradation from the maximum. This often occurs near the point where the first wall of the building reaches its ultimate displacement capacity in shear or flexure. The ultimate base shear is the shear value corresponding to the ultimate displacement on the capacity curve. These limit states are illustrated in Figure 4, where the seismic response curve (or capacity curve) of the structure is idealized as a bi-linear elastic-perfectly plastic envelope.

The results of the comparison (as shown in Figure 5 for 1, 2, and 3-story configurations in the case of equivalent frame modeling) indicate that the shear force distribution in the walls derived from linear elastic models differs significantly from the shear force distribution at the ultimate limit state, which is obtained through nonlinear static analysis. Notably, the base shear forces in the walls are distributed based on the strength of the walls rather than their stiffness. The distribution of axial loads on the walls is strongly influenced by the boundary conditions, which are determined by the coupling provided by reinforced concrete (r.c.) ring beams, floor slabs, or spandrels. In equivalent frame models, the axial force distribution on the walls changes as the total lateral force applied to the structure increases due to the coupling effects of the horizontal elements. However, it has been found that the axial load distribution at the ultimate limit state closely resembles the distribution in a linear elastic analysis, provided the applied lateral force (Fh) remains between the elastic limit (Fel) and the ultimate limit (Fy).

The moment distribution in the walls at the elastic and ultimate limits differs considerably. However, by comparing the parameter θ (representing the height of the point of contra-flexure relative to the total height of the wall, $\theta = M/(Vh)$, where M is the moment, V is the shear, and h is the height of the wall), it is possible to conclude that the differences between the elastic and ultimate limits are minimal. Therefore, the point of contra-flexure remains relatively unchanged from the elastic to the ultimate limit state. Based on these observations, two simplified procedures for the seismic design of masonry buildings have been proposed, and these are discussed in the following sections.

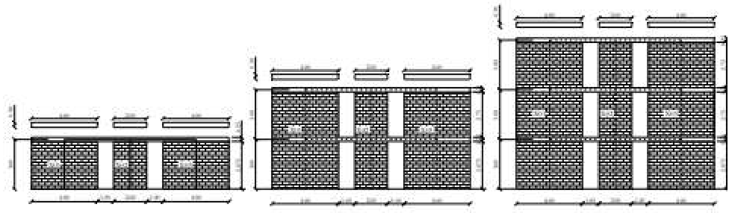

Figure 3 Some of the structural configurations analyzed: 1, 2 and 3 storey plane masonry walls

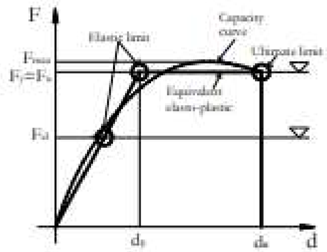

Figure 4 Limit states on the capacity curve

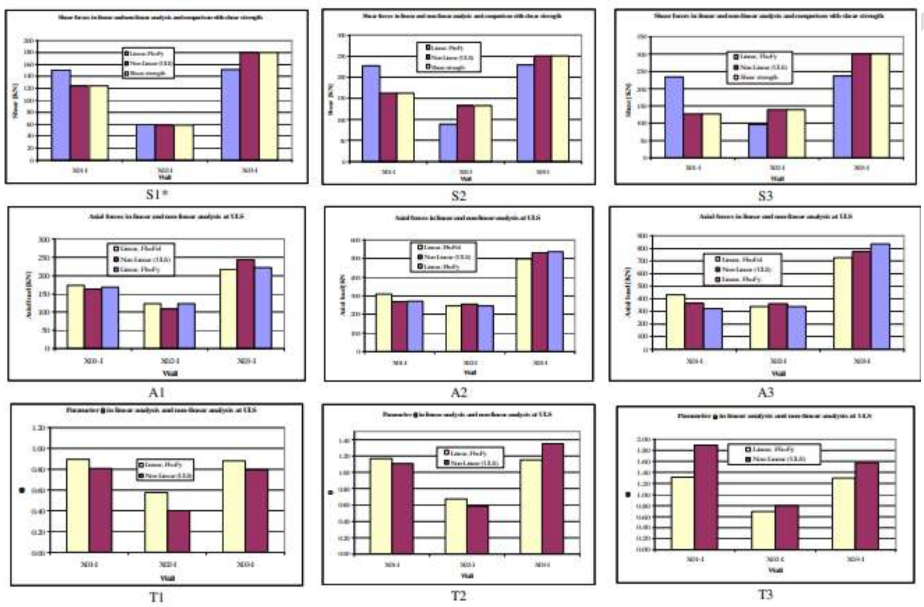

Figure 5 Histograms of the distribution of the ground storey of the shear forces in the walls (S1, S2, S3) Histograms of the distribution of the ground storey of the axial forces in the walls (A1, A2, A3) Histograms of the values of the parameter θ in the walls of the ground storey (T1, T2, T3)

* The number after S, A, T means that the results are related to the structural configuration with 1, 2 or 3 storeys.

Simplified Procedures

Procedure A: Linear Elastic Seismic Design with Redistribution: This procedure involves a traditional equivalent linear elastic static analysis, where lateral forces are applied at each storey based on a linear first-mode displacement shape. The lateral seismic force is derived from the acceleration design response spectrum, adjusted by the behavior factor qqq, which accounts only for the structure's dissipation capacity and should not include the overstrength ratio (OSR), as this can vary widely. After calculating the lateral seismic force, a structural analysis is performed using either a cantilever or equivalent frame model. The internal forces in all walls are computed, and safety checks are made by comparing these forces with the wall strengths. If some walls fail to meet safety requirements under moderate seismic actions, internal forces can be redistributed among the walls. This redistribution must maintain translational and rotational equilibrium, meaning the total force at each storey and its application point must remain unchanged [14]. The shear and bending moment can be adjusted within the strength capacity of the walls as long as equilibrium is maintained.

Procedure B: Capacity-Based Seismic Design: In this procedure, the shear distribution at the ultimate limit state is based on wall strength rather than elastic stiffness. At the ultimate limit, the shear is distributed according to strength, not stiffness, and this holds true for the critical storey walls. The distribution of axial force at ultimate depends on the coupling between the walls, influenced by the r.c. ring beams. If coupling is low, the walls can be modeled using the "cantilever" model, where axial forces remain constant from the elastic to the ultimate limit state. However, for stiffer r.c. beams, an equivalent frame model should be used, as axial loads vary during the analysis. The distribution of axial forces at ultimate closely mirrors the distribution from a linear elastic analysis when the lateral force lies between the elastic and ultimate limits.

$$F_h = \min(0.3 \cdot W_{TOT}; W_{TOT} \cdot \frac{S_e(T)}{q_0})$$

$$F_i = F_h \frac{s_i W_i}{\sum_{j=1}^{ns} s_j W_j}$$

where WTOT is the weight of the building, 0.3WTOT is a simplified estimate of the strength of the building, Se(T) is the ordinates of the elastic response spectrum, q0 is the behaviour factor without considering the overstrength, Fh is the total base shear, Fi is the force to apply to the i-th storey of the building (ns is the total number of the storeys), si is the displacement of the i-th floor in the first mode of vibration. Based on the above considerations, a linear elastic analysis can be used to estimate the axial force distribution and the point of contra-flexure of the moment diagrams for the walls of an unreinforced masonry building, assuming these values are close to those at the ultimate limit state. By knowing the axial force and the contra-flexure point for each wall, the shear and flexural strength can be calculated. The shear strength corresponding to flexural failure can be determined by dividing the flexural strength by the distance from the end of the structural element to the point of contra-flexure. The minimum of these calculated strengths represents the effective shear strength of the wall. This procedure can be repeated for each wall of the building to compute the resisting shear at every storey.

The critical storey is the first storey to experience a failure mechanism based on the distribution of the applied lateral forces. It is the storey that provides the minimum base shear resistance $V_{R,TOT}$ Base for the assumed lateral force distribution.

$$V_{R,TOT\_Base} = \min\left\{ V_{R,TOT\_1}; V_{R,TOT\_2} \cdot \left(\frac{\sum_{i=1}^{ns} s_i \cdot W_i}{\sum_{j=2}^{ns} s_j \cdot W_j} \right); \ldots; V_{R,TOT\_ns} \cdot \left(\frac{\sum_{i=1}^{ns} s_i \cdot W_i}{\sum_{j=ns}^{ns} s_j \cdot W_j} \right) \right\}$$

Where is $V_{R,TOTk}$ the shear resistance of the k-th storey.

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The shear resistance calculated for each storey provides a good estimate of the storey shear resistance, especially in buildings modeled as "cantilever". The safety check involves comparing the base shear resistance with the seismic base shear, which is calculated as the product of the total seismic weight W_{TOT} and the spectral ordinates of the acceleration elastic spectrum corresponding to the first mode period $Se(T)$, divided by the behavior factor q0q_0q0 (i.e., $\frac{WTOT\ Se(T),}{q0}$). If the shear resistance is less than the seismic force, the building does not pass the safety check. If the shear resistance exceeds the seismic force in both main directions, torsional equilibrium must be checked as described in the next section.

Torsional Effects: Modern buildings are typically torsionally restrained systems, with at least one pair of walls not aligned along the same plane in both main directions to resist lateral seismic actions (Figure 6a). In such cases, one of the walls parallel to the seismic direction will fail first due to shear or flexure. The stiffness and location of the longitudinal walls perpendicular to the seismic direction ensure that lateral forces continue to increase until all walls fail. The overall building force-displacement response is shown in Figure 6b.

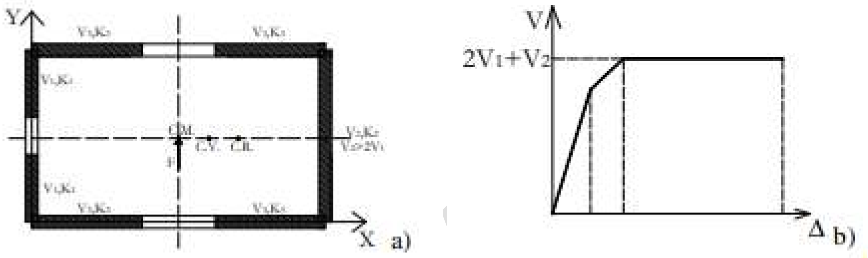

Figure 6 Response of a torsionally restrained building; a) plan view of building; b) force-displacement curve.

It is important to note that once all walls have reached their resistance, the effective stiffness in the considered direction becomes zero. This means that increasing displacement in the direction parallel to the seismic action no longer increases the total storey shear. Furthermore, when all walls have failed, the center of resistance shifts from the center of elastic stiffness (C.R.) to the center of shear strength (C.V.). The torsional moment M_T at the ultimate limit state, when all walls have attained their resistance, can be computed using the following expression, assuming the main seismic action direction is along the Y-axis (Figure 6a).

5. Conclusion

In conclusion, the seismic design of masonry buildings involves various analytical methods, including linear static, linear dynamic, nonlinear static (pushover), and nonlinear dynamic analyses. Linear elastic analysis, which utilizes two performance levels (Ultimate Limit State for strength verification and Damage Limit State for deformation control), is commonly employed. However, inconsistencies arise in linear methods when ground accelerations exceed 0.05g, often overestimating safety checks. The behavior factor (q-factor) typically underrepresents the seismic capacity of masonry buildings, necessitating the inclusion of the overstrength ratio (OSR) for more accurate modeling. The OSR accounts for the additional strength provided by the redistribution of forces after initial failure, thus ensuring a more realistic assessment of the building's ultimate capacity. Nonlinear static analyses, like pushover analysis, have been found to produce results that align better with real-world observations compared to linear methods. The revised approach, incorporating OSR, offers a more

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reliable representation of the seismic behavior of masonry buildings, reducing the discrepancies seen in previous linear design methods and enhancing safety and design accuracy.

Future Scope

- Further development of nonlinear dynamic analyses for more accurate predictions in extreme conditions.
- Research into fragility curves for tailored safety measures across various seismic intensities.
- Exploring combined linear and nonlinear methods to balance design efficiency and safety.

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