SEISMIC ANALYSIS OF MASONRY STRUCTURE

A REPORT SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE

OF

MASTER OF TECHNOLOGY

IN

STRUCTURAL ENGINEERING

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August-2021

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I, Vidit Deol , Roll No. 2K19/STE/13 student of M. Tech (Structural Engineering), hereby declare that the project dissertation titled "SEISMIC ANALYSIS OF MASONRY STRUCTURE" which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associate ship, Fellowship or other similar title or recognition.

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CERTIFICATE

This is to certify that the project report entitled "**Seismic Analysis of Masonry structure**", is the Bonafede work of **Vidit Deol** for the award of Master of Technology in Structural Engineering from Department of Civil Engineering, Delhi Technological University, Rohini, Delhi. The work has been carried out fully under my supervision. The content and results of this report, in full or in parts has not been submitted to any other institute or university for the award of a degree.

hdubey 03/09/21

Place: Delhi
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ACKNOWLEDGEMENT

I sincerely take this opportunity to acknowledge the debt of gratitude that I owe to my supervisor Mr. Hrishikesh Dubey for allotting this interesting and prestigious topic to me. He helped me by means of his vast knowledge and expertise.

He also provided the highly anticipated motivation when things were not going well which really boosted my morale to a high altitude to confess. I consider myself as one of the fortunate students of the institute as I had an opportunity to work under his guidance. Thanks for always being there with a smile and words of encouragement.

I am also thankful to all my dear friends for their kind co-operation during the completion of my project. Without their constant help and treasurable company, things would not have been as smooth.

I am indebted a lot to my parents for their blessings and constant moral support during the tenure of M-Tech degree and throughout my life.

VIDIT DEOL

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CHAPTER 1

INTRODUCTION

From the beginning of construction history, masonry have been most widely used material Because of its light weight, great durability, fire resistance, thermal insulating characteristics, relatively simple method of construction and execution, and low cost, masonry has been the most extensively used material in construction history. Apart from possessing such features, masonry is thought to have several disadvantages, such as distinct directional (orthotropic) qualities. weak strength, mortar connections without proper thickness, low interface binding strength between brick and mortar, poor treatment and improper methods of installation, etc., Chourasia, A. et. al. (2017) [6]. Besides, having weak strength and weak seismic behaviour masonry is still used heavily in construction for structures all around the world notably for low-rise buildings up to three floors, as well as antique and medieval structures.

Masonry can be used in construction with or without reinforcement; however, providing reinforcement improves masonry's seismic performance. Confined masonry is a relatively new type of masonry that has been presented, where light-reinforced frame elements (tie beams and columns) are utilized to enclose a masonry wall, the system acts as a single unit under load, providing ductility and strong seismic properties. Confined masonry has been found to be superior to reinforced masonry in some cases.

In this report a residentials G+3 building in, Delhi has been studied, building in its natural form is constructed of masonry single brick thick with 250 mm concrete slab. The building is further modelled in ETABS (2018) [12] software with masonry as thin shell element of 230 mm thick and concrete slab of 250 mm thickness and dead, live and seismic load is applied on this building. Building is further modified and modelled in software with a 1) lintel band and 2) with confining elements (confined masonry) and change is seismic behaviour and capacity is studied.

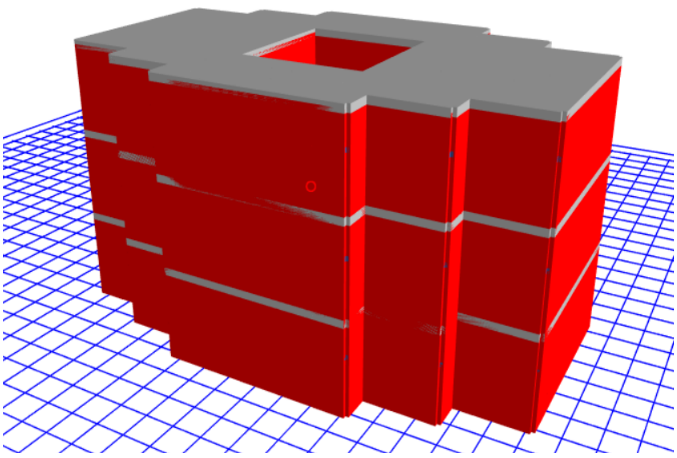


Figure 1: Type 3 residential building

CHAPTER 2

Masonry constructions And Type

2.1 Introduction

Since beginning of civilization, masonry has been most commonly used material for construction because brick masonry has high durability, great fire resistance, acoustic and thermal insulation features, and is relatively simple to realize and inexpensive, it has been the most often used material for construction from the dawn of civilization, primarily for low-rise buildings. Buildings made of masonry, on the other hand, have a few disadvantages. Distinction is one among them. Masonry-built structures, on the other hand, have a number of disadvantages also. Some of these characteristics include different directional (orthotropic) features; poor masonry unit and mortar strength; mortar connections with non-uniform thickness, low interfacial binding strength between brick and mortar, curing, and workmanship, and so on, Chourasia, A. et. al. (2017) [6]. Masonry, like most construction materials, has flaws that cause it to function poorly during earthquakes. Unreinforced masonry construction is widely found in the country and not termed as safe during earthquakes, reinforced and confined masonry was hence developed so as to improve seismic behaviour of masonry structures; See seismic design guidelines [4]. During an earthquake, unreinforced masonry constructions are the most vulnerable. They are intended to handle vertical loads and because the compressive strength of masonry is sufficient, the loads remain vertical. In earthquakes, shear and bending stresses arise when these structures are exposed to lateral inertial loads. Under these circumstances, the relationship between brick and mortar (or stone and mortar), affecting the strength of the masonry, Horizontal bands are important to hold the walls at joints together to avoid vertical splits and in-plane splits. “However, they might not be enough to protect against out-of-plane flexure, notably for horizontal flexure cracks”, Jagadish, Raghunath K.S et.al [14]. In order to overcome the inherent defects, it is essential to improve the seismic properties of the masonry system using appropriate ways. Different approaches were used to improve the seismic properties of URM systems, which finally led to the creation of reinforced masonry (RM) and confined masonry (CM) systems. Confined masonry or CM is a load bearing wall surrounded by cast-in-place tiny reinforced cement pillars and beams, respectively known as the Tie Column and the Bond Beam. The system handles vertical and side stresses on walls, Alcocer, S.M. et.al (2004) [7] For buildings with up to four stories, which is the most common housing typology in developing countries around the world. Functional, structural, societal, economic, and environmental criteria must all be met in order for a building to be considered sustainable. Actually, URM is the structural solution that best meets these needs. Recent earthquakes in India have demonstrated the repercussions of poorly constructed masonry structures, which account for around 85 percent of all extant buildings in the country Chourasia, A., et. al. (2015) [8]. The seismic activity must be addressed in terms of sustainability, earthquake resistance, and cost, and constrained masonry (CM) may be the best option for low-medium rise buildings on these parameters

2.2 Unreinforced masonry

Unreinforced masonry is defined as masonry that is built without the use of reinforcement or restricting devices. These structures/buildings are extremely vulnerable to earthquakes. Due to the enormous dimension of the wall in the plane of loading, masonry walls perform well under in-plane loading, but when lateral loads are applied horizontally in a direction perpendicular to the plane of the wall, the wall topples down quickly, leading to failure. Unreinforced masonry's seismic capacity is primarily determined by stability and energy considerations rather than stress levels.

2.3 Reinforced masonry

Reinforced masonry is built to withstand both vertical and lateral out-of-plane stress. These vertically spanning walls transfer load to the roof and foundation. The reinforcements in these walls reaches between the supports and is suitably linked to the lintel band. Under seismic loads, the inclusion of reinforcement improves out of plane properties and prevents early cross cracking. Reinforcements, in general, cause a wall to act like a basic beam extending from one support to another. Furthermore, the usage of reinforcements protects the structure from slipping and collapsing in the direction of the weak point. Reinforced masonry can fail by 1) flexural failure: out of plane bending, brickwork under high axial stress and flexure may not necessarily fail demonstrating ductile behaviour, and damage is also highly severe in this form of failure. 2) Shear failure: masonry with apertures for windows and doors frequently fails in this mode; this failure is most common in masonry with a small height-length ratio; shear failure is brittle in nature and dissipates very little energy.

2.4 Masonry infill

It is essentially a framed structure in which the frame is infilled with a rigid masonry construction to provide residents with safety and separation. The infill brickwork serves only as a safety barrier and a partition, while the frame sections handle nearly all of the weight. There is structural contact between the frame and the filler. This sort of structure is extremely resistant to seismic forces. According to clause 7.9.2.2 of IS: 1893 (2016) [18], these types of structures can be studied using the comparable diagonal strut technique.

2.5 Confined Masonry

“Confined masonry, or CM, is made up of load-bearing walls surrounded by small cast-in-place reinforced concrete tie-columns and beams, referred to as Tie Columns and bond-beams, respectively”. The system is set up in such a way that the walls withstand both vertical and lateral loads, Alcocer, S.M. et.al (2004) [7]. Plain masonry wall panels are constrained(confined) in this style of construction by using frame elements all around, which increases ductility and enhances masonry out of plane loading properties. Tie beams and tie columns are lateral and vertical restricting elements, respectively; these elements are not meant to act as frame elements. Masonry units carry the load, while concrete confining components have a little role in sharing vertical loads, although they do offer restraint to masonry walls and protect them from damage and failure during seismic events. This kind of construction combines the advantages of reinforced masonry and masonry infill.

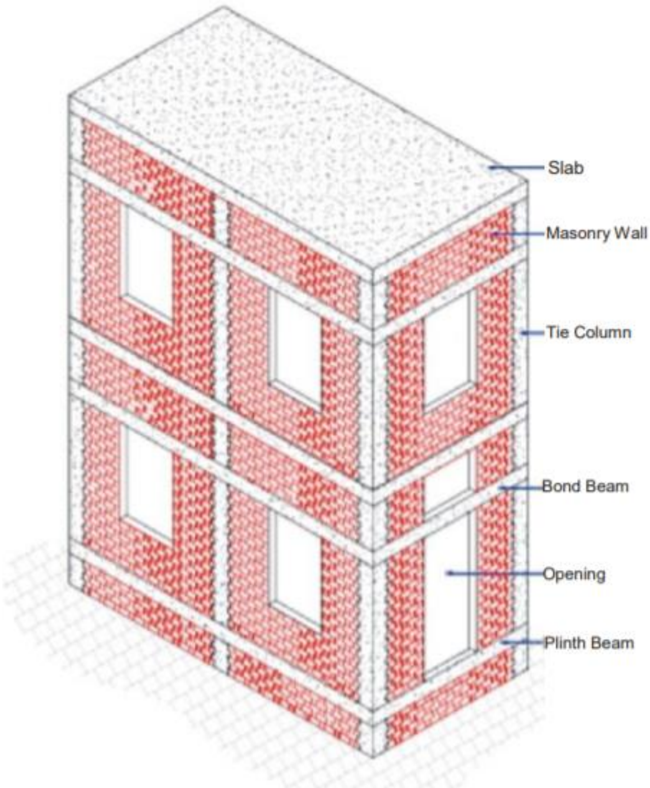


Figure 2: Typical section of confined masonry structure

2.6 Confined Masonry Vs Masonry Infill

Figure 3:Masonry with infill wall

Figure 4: Confined masonry

Although, confined masonry and masonry infill looks similar after constructed but they differ form each other in ways as follows:

- In confined masonry, the walls are constructed first, followed by the tie columns and beams, whereas in masonry infill, the frame structure is constructed first, followed by the masonry infill.
- In confined masonry construction, the masonry unit absorbs practically all of the gravitational and vertical load, as well as lateral loads during earthquakes, but in masonry infill, Due to the lack of any bond between the masonry unit and the frame element, the weight is carried by the frame elements.

Both confined masonry and infill masonry walls behaves differently under seismic load due to reasons are summarized below:

- Frame action is not provided by smaller cross-section tie pieces. In contrast to the moment connections offered in RC frames there is pinned type connection between columns and beams in CM, when compared to tie elements in a constrained masonry building, RC frame beams and columns have a greater cross section and are stiffer.

- There is no frame action in confined masonry tie columns and beams connection are pinned as compared to stiff connection between the RC frame structure.

- Sharp toothed surfaces and doweled edges are used to bond tie-columns and beams to masonry units. In masonry infill construction, there is no link between the infill and the RC frame component.

- When lateral seismic loads are applied to confined masonry structures, the walls operate as shear walls, similar to unreinforced or reinforced masonry walls or RC shear walls. Infill wall panels in RC frame buildings, on the other hand, serve as diagonal struts rather than shear walls. “There are gaps between the masonry infill and the RC frame due to a lack of connection, which significantly reduces the ability of infill walls to resist lateral forces in a seismic event, as seen in Figure 4. Due to building tolerances, these gaps may already exist prior to an earthquake”: Seismic Design guidelines for low-rise confined masonry buildings (2011) [9]

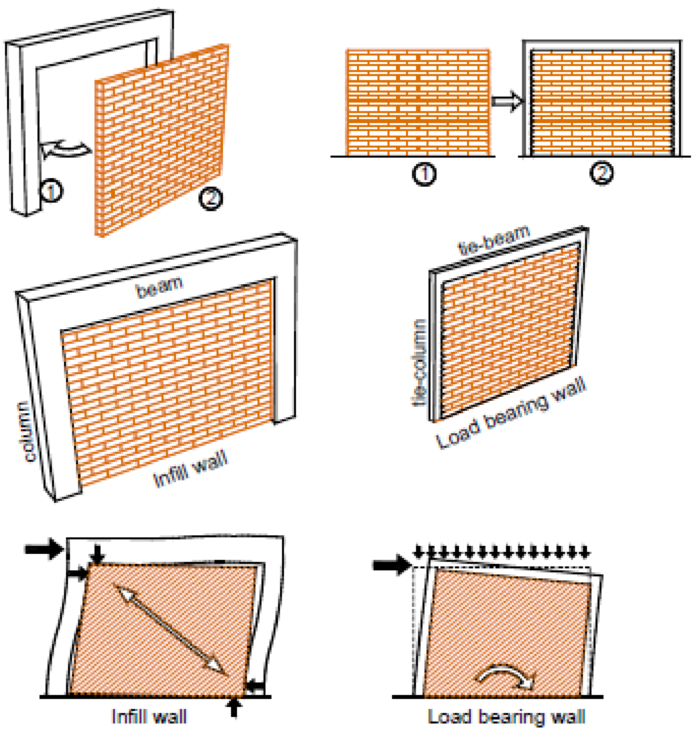


Figure 5:Seismic Behaviour of infill masonry and confined masonry.

2.7 Past Performance of confined masonry

During a prior earthquake, the performance of a confined masonry structure was found to be appropriate in the context of seismic design philosophy (Photo 1.1), which emphasises life safety and collapse prevention. CM constructions built using both good and terrible construction practises have performed well in several of the world's most powerful earthquakes. The information gathered from these occurrences revealed that the common damage patterns are as follows:

1. Shear failure of walls
2. Shear and bending failure at ends of tie-column
3. Separation of tie column from walls
4. Inadequate wall densities in two orthogonal directions
5. Development of first story mechanisms.

“In some of the cases, damage occurred at upper storeys of the building, with associated out-of- plane damage, mostly due to absence of integral box behaviour of the storey. Large spacing between tie elements, lack of anchorage in reinforcement of tie beam and column, excessive spacing between tie elements, high height to depth ratio, unsymmetrical walls in plan, poor wall density, cheap craftsmanship, poor material selection and minor constructional defects are the most common reasons for failure in CM buildings”. There have been no known cases of CM building foundation failure. Nonetheless, confined masonry architecture has generally shown strong seismic resistance when built appropriately, and no serious damage has occurred during previous earthquakes. Seismic Design guidelines for low- rise confined masonry buildings (2011) [9]

Figure 6:Good performance of Confined Masonry construction during Earthquake

Design Recommendations for Confined Masonry Structures Confining features such as RC tie columns and bond beams confine masonry walls, resulting in increased strength, integrity, and stability in both in-plane and out-of-plane, even during major earth movements. Tie columns are less in size because they do not support considerable gravity loads.

Figure 7: Damage to confined masonry buildings during various earthquakes.

2.8 Mechanism of failure in CM

Masonry is a composite material made up of two distinct components: brick and mortar. Masonry, in its most basic form, is brittle, with a high unit weight, low tensile strength, and a poor response to seismic stress. Flexural failure, diagonal failure, and sliding failure are the three types of masonry failure. In confined masonry during failure the emergence of the first crack is due to the compressive strength of masonry and the tensile strength of reinforcing bars, and fractures form first at joints under tension, followed by crushing of compressed masonry toe, because it is ductile and good at absorbing earthquake energy, this form of failure is always favoured in Confined masonry. Shear failure, on the other hand, occurs when horizontal and vertical joints crack prior to the flexural limits of the walls, resulting in diagonal X-shaped cracks. Chourasia, A. et. al. (2017) [6]

Up until the first crack appears, confined masonry exhibits elastic behaviour. After then, the masonry wall behaves as two triangles restricted between columns due to the creation of diagonal fissures. When these triangles move or slide around the restricting elements, the wall is compressed, and another load transmission pathway is developed. Shear deformations owing to frictional effect, interlocking of bricks , and resistance offered by tie-column ends frequently cause such cracks to zigzag through mortar joints, generating a plastic hinge at the compressed column's base. As a result, confinement changes the masonry's failure mode and enhances the CM building's post-cracking performance.

Figure 8: cracking Behaviour of confined masonry

CHAPTER 3

LITERATURE REVIEW

3.1 Brief

1. “Study of confined masonry buildings in seismic areas”, Sorina Constantinescu (2016)

To demonstrate ductile behaviour is obtained in confined masonry structures, pushover analysis was performed on a model of a recently built building in Romania, and a separate model were used to analyse the ductility behaviour of each wall. The failure mechanism was discovered in tie elements, and the brickwork was fractured before reaching the plastic mechanism, yet the structure did not collapse. Masonry buildings were discovered to have stiff behaviour, and limiting features offer adequate ductility through a forming mechanism to keep the structure from collapsing.

2. “Seismic performance evaluation of full-scale confined masonry building using light weight cellular panels”, Ajay Chourasia, Shubham Singhal , Jalaj Parashar

A confined masonry structure was retrofitted with a light-weight cellular panel to reduce overall weight and make the construction more cost-effective was investigated under seismic loads. The results were validated by modelling the structure in ABAQUS software. A half-scale model of the building was produced and tested on a shake table. The outcomes of both physical and computer model testing were compared.

3. “Experimental study of seismic behaviour of renovated masonry structures after removing walls and seismic retrofitting”, Fang - fang Wei a, You - hua Zhu a, Jun Yu a, Yong - quan Wang b

The masonry structure was restored and retrofitted using three methods. Two bearing walls on the ground floor were removed, and two short-width shear walls, as well as a wider beam section, were installed in their place. By eliminating one ground-floor wall, expanding the sections of constricted beams and columns to form a single-bay frame, and retrofitting the neighbouring structure, the second structure was rehabilitated. According to the findings, all three species displayed frantic behaviour accompanied by pinching. The energy dissipation capacity of the two renovated structures, particularly the first renovation plan, was greater than that of the original structure prior to 1% drift. In comparison, the second remodelling scheme enhanced structural resistance more visibly., However, because the stiffness of the restored ground story was significantly greater than that of the second story, the ultimate deformation capacity was reduced compared to the original structure. In comparison, the second remodelling scheme enhanced structural resistance more visibly., However, because the stiffness of the restored ground story was significantly greater than that of the second story, the ultimate deformation capacity was reduced compared to the original structure.

4. “Stress-Strain Characteristics of Clay Brick Masonry under Uniaxial Compression”, Hemant B. Kaushik, Durgesh C. Rai, and Sudhir K. Jain

Studies the stress strain relationship in masonry panel in compression by conducting test on masonry prism using different type of bricks and mortar and also developed an empirical formula for the strength of masonry in compression. Nonlinear stress-strain curves for bricks, mortar, and masonry have been obtained based on the results and observations of the comprehensive experimental study, and control points on the stress-strain curves of masonry have been identified, which can also be used to define the performance limit states of the masonry material or member.

5. “Design guidelines for confined masonry”, Ajay Chourasia CRRI

A single-story, full-scale restricted brick structure was subjected to single directional reversed cyclic lateral deformations with increasing magnitudes at a very low frequency as part of an experimental study.

Figure 9: Single Storey model of Unreinforced, reinforced and confined masonry for experiment by Chourasisa.et.al

The occurrence of the first horizontal crack in a mortar joint, which modifies the building's initial stiffness and is dictated by lateral load (H_{cr}) and displacement, is known as the elastic crack limit state (d_{cr}). When the building attained maximum resistance, the maximum lateral load (H_{max}) and associated lateral displacement (d_{H-max}) were measured.

Table 1: Observation of experimental results by Chourasia.et.al

Building Typology	dcr (mm)	dH -max (mm)	d d-max (mm)	Hcr (kN)	H H-max (kN)	Hd -max (kN)
Unreinforced Masonry	2.85	3.30	3.70	41.00	44.50	43.01
Reinforced Masonry	4.77	6.70	23.70	47.86	57.85	38.75
Confined masonry	9.39	25.15	54.01	131.04	152.25	132.92

Building Typology	Maximum Drift %	Ductility

Unreinforced Masonry	0.123	1.298
Reinforced Masonry	0.790	4.968
Confined masonry	1.80	5.755

1. Kömürcü1 Sedat and Gedikli1 Abdullah (2019) “Macro and Micro Modelling of the Unreinforced Masonry Walls” European Journal of Engineering and Natural Sciences, EJENS(2019).

Micro and macro modelling are two different types of modelling. These two modelling methodologies were numerically investigated on a solid unreinforced masonry shear wall in this paper. The models were tested using the ANSYS programme to simulate structural behaviour. The fracture propagations on the walls are numerically studied. The micro and macro modelling results are congruent with the experimental investigation published in the literature. However, in terms of material identification and crack propagation, macro and micro models exhibit completely different behaviours.

2. Chourasia Ajay *, Singhal Shubham, Parashar Jalaj (2015), “Seismic performance evaluation of full-scale confined masonry building using light weight cellular panels”, Elsevier.

Confined masonry was further retrofitted with light weight cellular panels to make overall construction less bulky so as to increase seismic behaviour of structure. The above model was tested to various vibration produced by actuator till collapse point and cracking analysis was observed.

CHAPTER 4

Material and Modelling

4.1 Material

The materials used in masonry building vary depending on the kind of masonry, but typically include mortar, bricks, reinforcements, and concrete. Masonry is a non-homogeneous composite object, formed by repetitive units of bricks and mortar that both exhibit nonlinear behaviour. Bricks are often stiffer than mortar, and masonry is generally, weak in tension because it is made up of two nonlinear materials bonded together by a weak link. Because masonry structures are non-homogenous material obtained by repetitive units of brick and mortar placed alternatively, it is supposed that the strength of masonry will be somewhere be between that individual strength of bricks and mortar. "This assertion is only valid when one material, such as brick and mortar, is significantly stiffer than the other", Hemant. B Kaushik et.al [1].

4.1.2 Bricks

Bricks are made of clay and come in a variety of shapes and sizes, depending on the manufacturing procedure and the type of material employed. A range of tests, including colour, texture, and size, as well as water absorption, compressive strength, and chemical composition, are used to choose bricks for brickwork. Brick tests are carried out in accordance with IS 3495. (1992) [20]. However, the most essential stress in brick analysis is compressive stress, which is symbolized by f_b: strength of brick under compression. Hemant. B Kaushik et.al [1], performed several tests on 40 samples of bricks made up of four different types of bricks, demonstrating that bricks have non-linear behaviour as follows:

Table 2: Stress- Strain behaviour of different type of bricks

Brick type	fb (N/mm^2)	Failure strain	Modulus of Elasticity (N/mm^2)
M (10 samples)	17.7	0.0072	5300
B (10 samples)	16.1	0.0060	5030
O (10 samples)	28.9	0.0070	7516
S (10 samples)	20.6	0.0057	6534
Average of 40	20.8	0.0065	6095

4.1.3 Mortar

Mortar is a non-homogeneous material made up of two components: cement and fine aggregate combined with water. Mortar acts as a binder, joining the bricks together. Different quantities of cement and particles are mixed together to make different types of mortar. Mortar contributes to the masonry wall by allowing the clay bricks to make touch with each other. Mortar contributes to the masonry wall by providing contact friction for the clay bricks. As a result, the mortar composition has a significant impact on compressive and shear strength. The composition also ensures the material brick wall's bonding and workability, Hemant. B Kaushik et.al [1], conducted compressive test on three types of mortar that 1:0:6 ratio weak mortar, 1:0:3 strong mortar, 1:0.5:4.5 intermediate mortar as:

Table 3: Stress- Strain behaviour of different type of mortar.

Mortar type	f_j (N/mm^2)	Failure strain	modulus of elasticity E_j (N/mm^2)
weak	3.1	0.0087	545
strong	20.6	0.0185	3750
intermediate	15.2	0.0270	3300

4.1.4 Masonry prism

A masonry prism test is used to analyse and determine the strength and properties of masonry as a single unit. The strength of this prism is modelled as the strength of masonry unit in our model, where masonry is represented by thin shell parts. Prism strength is given by empirical formula, $f_{m:}$ compressive strength of masonry prism:

$f_m = 0.433\, f_b^{0.64}\, f_j^{0.36}$
$E_m = 550 f_m$

Stress strain behaviour of masonry is taken as:

Table 4: Stress Strain behaviour of Masonry taken for analysis.

S.No	Strain	Stress (N/mm^2)	Point ID
1	0	0	
2	0.0005	2.18	A
3	0.0015	4.95	B
4	0.0021	5.94	C
5	0.003	6.6	D
6	0.0062	3.3	E

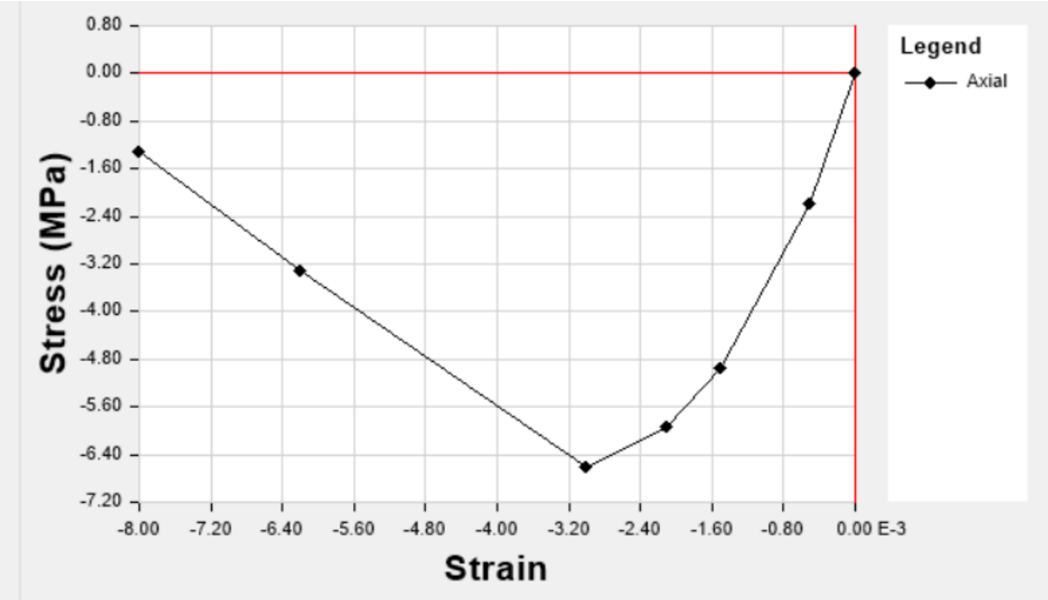


Figure 10: Stress strain plot of Masonry for analysis.

4.2 Modelling

The analytical models for the building incorporate all components that determine mass, strength, and stiffness. Non-structural parts and components that have little impact on the building's behaviour were left out of the model. All the load has been taken as per IS: 875 Part 1, Part 2[15] [16] and seismic details and details have been taken and done as per IS: 1893 (2016) [18]. Model has been provided with lintel bands in model 2(Masonry with lintel band) as according to IS 1905 (1987) [19]. Provision of tie columns and beams in confined masonry models has been done as stated in Eurocode 6(2006) [9] and Seismic Design guidelines for low-rise confined masonry buildings (2011) [9]

4.2.1 Macro Modelling

In macro modelling or homogeneous modelling, masonry is considered one unit, consisting of repeating bricks and mortar units, both of which exhibit nonlinear behaviour, and values derived by various tests on masonry prism are considered masonry wall properties. In this present work macro modelling is used with masonry properties obtained by the prism test concluded by that of previous work. Masonry walls or panels has been macro modelled as thin shell element, ETABS (2018) [12], software is used for modelling, Sedat Kömürcü.et.al (2019) [3]

4.2.2 Micro Modelling

Micro model or Heterogenous model the mortar and the units are treated independently. This method works well with little models. The analysis cannot be completed in a reasonable amount of time due to the intricacy of the models. Large-scale models can benefit from homogeneous modelling. Smeared masonry units and mortar elements are labelled as isotropic or anisotropic materials. For this modelling, it's critical to get test results of a large masonry component with a sufficient number of units and mortar combinations., Sedat Kömürcü.et.al (2019) [3]

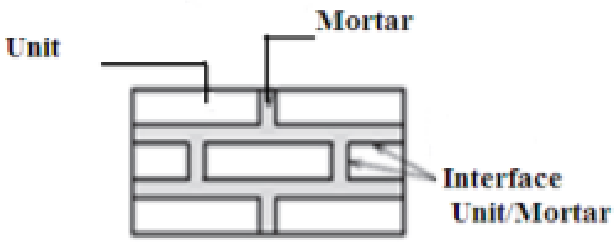


Figure 11: Micro Modelling of Masonry

4.2.3 Description of current model

In present work a G+3 residential building in Delhi has been studied, building in its natural form is constructed of masonry single brick thick with 250 mm concrete slab. The building is further modelled in ETABS 2018 software with masonry as thin shell element of 230 mm thick and concrete slab of 250 mm thickness and dead, live and seismic load is applied on this building. Building is further modified and modelled in software with a 1) lintel band and 2) with confining elements (confined masonry) and change is seismic behaviour and capacity is studied. The model's dimensions and other specifications are listed in the table and figure below:

Table 5:Details of Model Unreinforced Masonry

S.N.	Parameter	value
1	storey height (c/c)	3m
2	Unit wt of concrete	25kN/m^3
3	unit wt of masonry wall	21 kN/m^3
4	modulus of elasticity of masonry wall	3800 MPa
5	modulus of elasticity of concrete (Ec)	22360.67 MPa
6	Thickness of slab	200mm
7	masonry wall thickness	230mm
8	slab thickness	230mm
9	concrete poison’s ration	0.2
10	Seismic zone	IV
11	Zone factor	0.24
12	Importance factor (I)	1
13	Response reduction$ factor (R)	1.5
14	Type of soil	Medium (Type-II)
15	Damping ratio	10%
16	Type of frame	Unreinforced masonry

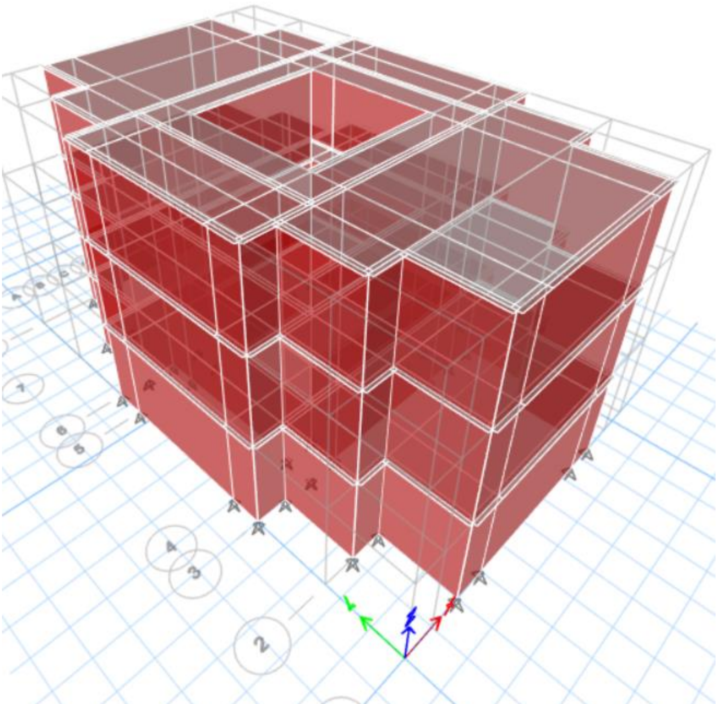


Figure 12: Unreinforced Masonry model of type building

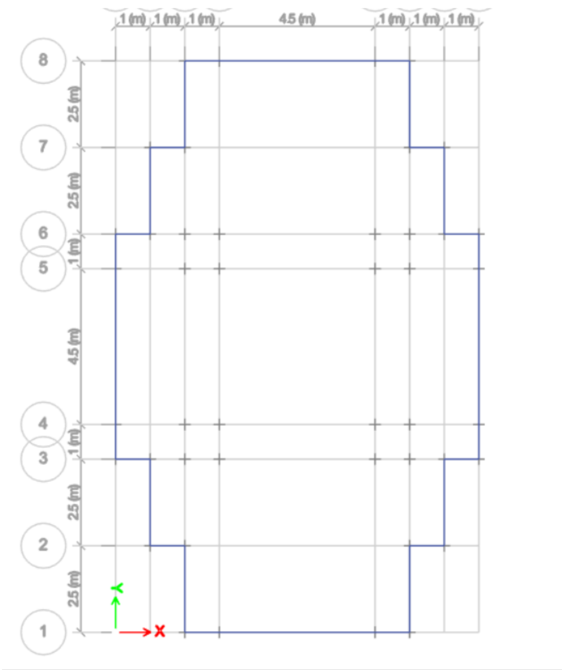


Figure 13: Plan of T Building,

Table 6: Details of Masonry with lintel band model

S.N.	Parameter	value
1	storey height (c/c)	3m
2	Unit wt of concrete	25kN/m^3
3	Lintel band	230mm*150mm
4	unit wt of masonry wall	21 kN/m^3
5	modulus of elasticity of masonry wall	3800 MPa
6	modulus of elasticity of concrete (Ec)	22360.67 Mpa
7	Thickness of slab	200mm
8	masonry wall thickness	230mm
9	slab thickness	230mm
10	concrete poisons ration	0.2
11	Seismic zone	IV
12	Zone factor	0.24
13	Importance factor (I)	1
14	Response reduction$ factor (R)	2
15	Type of soil	Medium (Type-II)
16	Damping ratio	10%
17	Type of frame	Masonry reinforced with horizontal lintel bands.

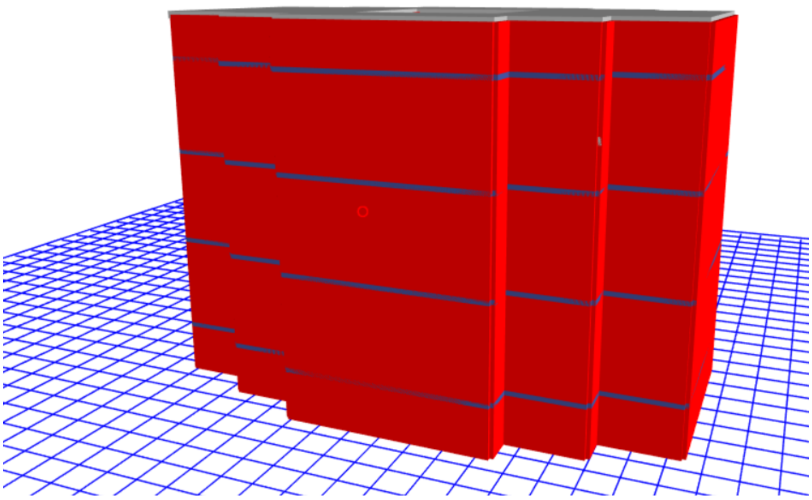
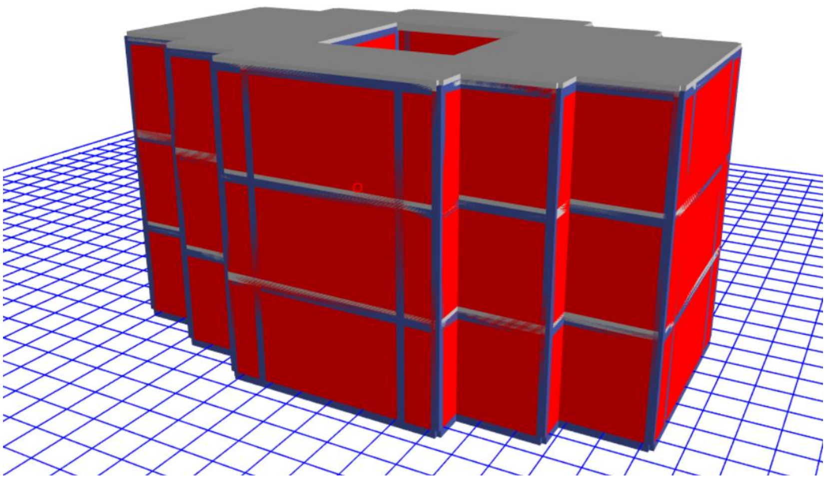


Figure 14: Masonry with lintel band model of building,

Table 7:Details of confined masonry model

S.N.	Parameter	value
1	storey height (c/c)	3m
2	Unit wt of concrete	25kN/m^3
3	Tie beam size	230mm*230mm
4	Tie column size	230mm*230mm
5	unit wt of masonry wall	21 kN/m^3
6	modulus of elasticity of masonry wall	3800 MPa
7	modulus of elasticity of concrete (Ec)	22360.67 Mpa
8	Thickness of slab	200mm
9	masonry wall thickness	230mm
10	slab thickness	230mm
11	concrete Poisson ration	0.2
12	Seismic zone	IV
13	Zone factor	0.24
14	Importance factor (I)	1
15	Response reduction$ factor (R)	3
16	Type of soil	Medium (Type-II)
17	Damping ratio	10%
18	Type of frame	confined masonry



CHAPTER 5

Results And Observations

5.1 Introduction

Depending upon nature of the variables, material and structure different type of method can be used. Based on the sort of force or external action applied and how the structure behaves as a result of this applied action, methods can be characterised as linear static, linear dynamic, nonlinear static, or nonlinear dynamic.

For symmetric constructions up to 2-3 stories with a simple plan such as a square or rectangle, linear static analysis is typically utilised. The methods for performing linear static analysis are response spectrum analysis and elastic time history analysis.

Because it allows for inelastic structural behaviour, non-linear static analysis is superior to linear static analysis. This method is simple and offers useful information on structure strength, deformation, and ductility, but it is based on certain assumptions that ignore higher order vibration nodes. FEMA 356 (2000) [13]

Linear dynamic analysis and linear static analysis are nearly identical, with the exception that linear dynamic analysis and linear static analysis have different load distributions along the narrative and different degrees of forces.

Nonlinear dynamic analysis is the only way for describing the real behaviour of a structure during an earthquake. This method allows for the numerical integration of numerous motion equations in a continuous manner.

5.2 Method of analysis used

5.2.1 Modal analysis

Modal analysis in structural engineering is a technique for determining the various periods at which a structure will naturally resonate by analysing its overall mass and stiffness. These time periods are basic to see in tremor designing since it is important that the normal recurrence of a structure doesn't coordinate with the recurrence of extended quakes in the district where the structure is to be developed. Modular examination is the most essential, since everything it does is mention to you what your calculation's "fundamental frequencies" are. Now, it has little to do with stacking and everything to do with math. Just the state of your model and how it is restricted and how it is impacted by the seismic frequencies.

Table 8:Modal results of unreinforced Masonry

Analysis Type	Mode Number	Time Period	Cycles per second	Circular frequency	Eigenvalue
		sec	cyc/sec	rad/sec	rad^2/sec^2

Modal	1	0.199	5.024	31.5698	996.6491
Modal	2	0.196	5.11	32.1042	1030.6797
Modal	3	0.078	12.87	80.8617	6538.6189
Modal	4	0.074	13.521	84.957	7217.6872
Modal	5	0.052	19.145	120.2889	14469.4308
Modal	6	0.045	22.249	139.7923	19541.879

Table 9: Modal results of Masonry with lintel band

Analysis Type	Mode Number	Time Period	Cycles per second	Circular frequency	Eigenvalue
		sec	cyc/sec	rad/sec	rad²/sec²
Modal	1	0.202	4.952	31.1165	968.2389
Modal	2	0.198	5.038	31.6542	1001.9852
Modal	3	0.155	6.451	40.5343	1643.0297
Modal	4	0.155	6.453	40.5439	1643.8051
Modal	5	0.155	6.456	40.5632	1645.3741
Modal	6	0.155	6.456	40.5633	1645.3818

Table 10: Modal results of Confined Masonry

Analysis Type	Mode Number	Time Period	Cycles per second	Circular frequency	Eigenvalue
		sec	cyc/sec	rad/sec	rad²/sec²
Modal	1	0.2	4.994	31.3789	984.6375
Modal	2	0.186	5.387	33.8499	1145.8168
Modal	3	0.073	13.782	86.5946	7498.6266
Modal	4	0.069	14.552	91.4301	8359.4554
Modal	5	0.047	21.332	134.0339	17965.0809
Modal	6	0.045	22.011	138.3007	19127.0921

5.2.2 Response spectrum analysis

Response spectra are graphs that show the relationship between the maximum response of an SDOF system and the time period during which it was subjected to a particular specified earthquake ground motion or acceleration. The maximum response of an SDOF system for a particular dampening fraction is defined as the range of responses. Response spectra help determine peak structural responses within a linear range, which may subsequently be used to quantify lateral forces produced in structures as a consequence of earthquakes, making earthquake-resistant structure design easier.

5.2.2.1 Storey Displacements

The movement of the storey relative to the ground during an earthquake is referred to as storey displacement. Extreme displacements can cause cracks, and excessive deflection is not psychologically acceptable.

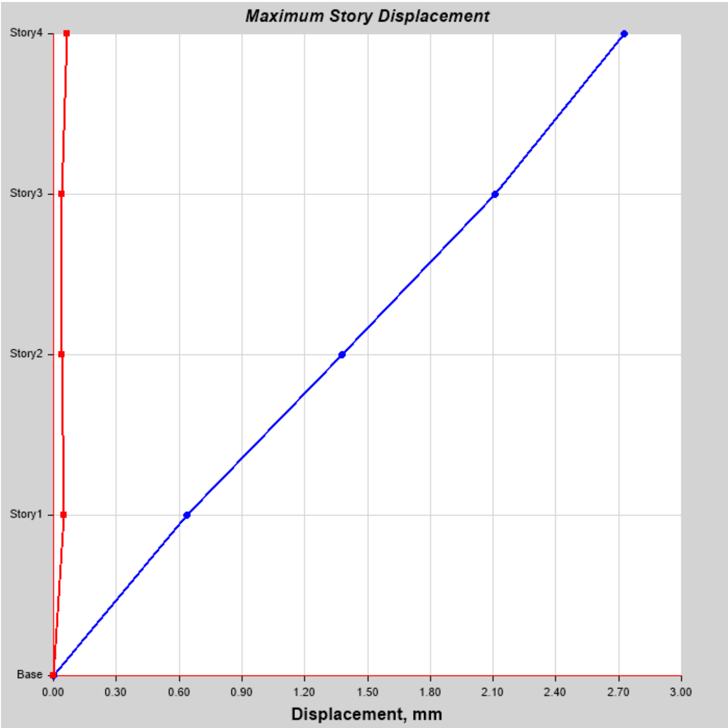


Table 11:plot of storey displacements in Unreinforced masonry.

Table 12:Data of Maximum Displacements in different stories in URM.

Story Number	Storey Height	Displacement(X)	Displacement(Y)	Allowable
	m	mm	mm	
4	12	4.149347	0.095649	2.53
3	9	3.217665	0.079127	2.53
2	6	2.125773	0.096329	2.53
1	3	1.028477	0.131107	2.53
Base	0	0	0	

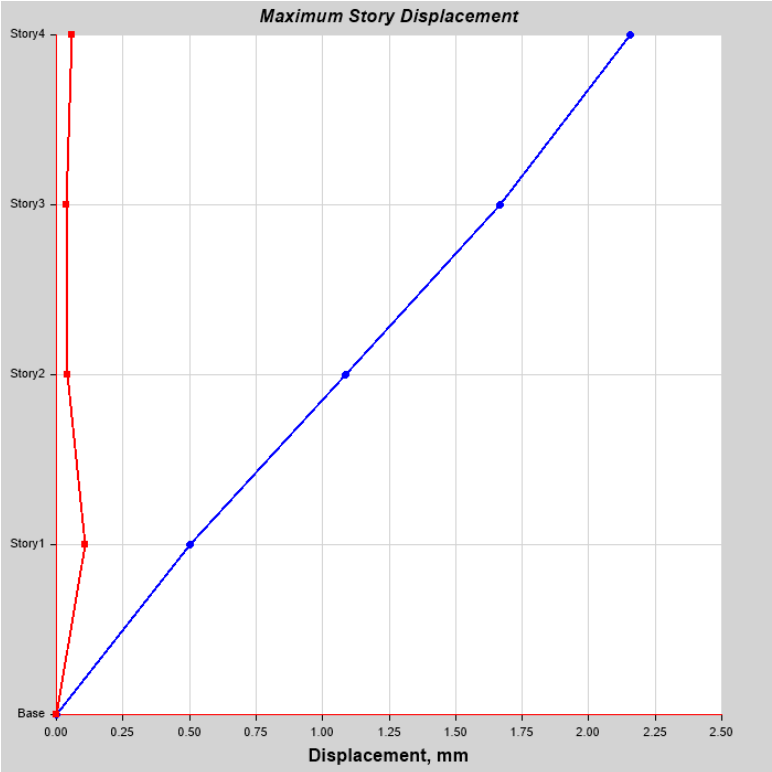


Figure 15:Storey Response in masonry with lintels

Figure 16: Data of Maximum Displacements in different stories in masonry with lintel band.

Story Number	Storey Height	Displacement(X)	Displacement(Y)	Allowable
	m	mm	mm	
4	12	3.35	0.134	4.77
3	9	2.594	0.079	4.77
2	6	1.708	0.095	4.77
1	3	0.833	0.171	4.77
Base	0	0	0	

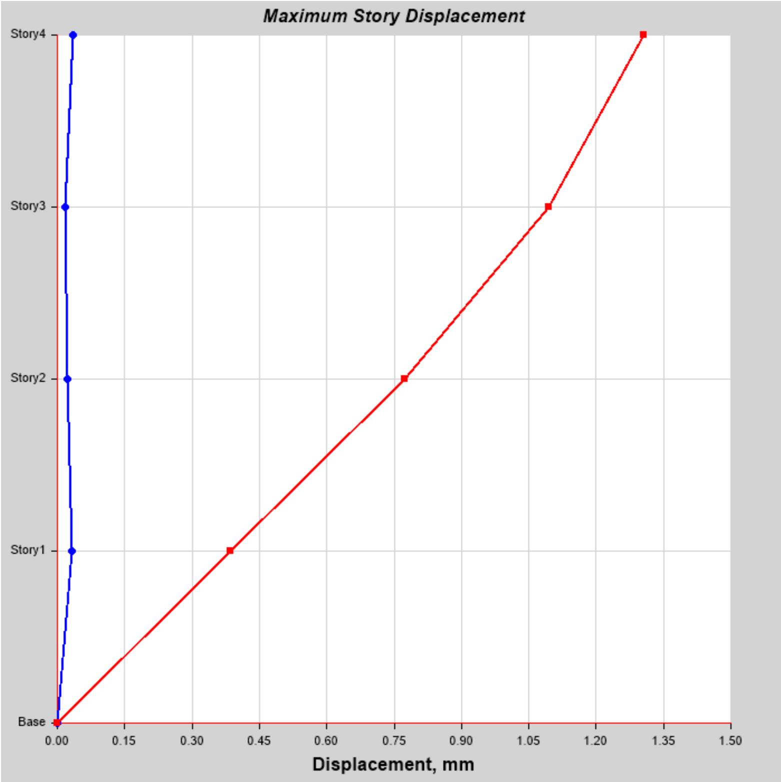


Figure 17:Storey Response in CM

Table 13: Data of Maximum Displacements in different stories in CM.

Story Number	Storey Height	Displacement(X)	Displacement(Y)	Allowable
	m	mm	mm	
4	12	1.801107	0.039954	9.44
3	9	1.431766	0.023304	9.44
2	6	0.953605	0.017716	9.44
1	3	0.434103	0.015224	9.44
Base	0	0	0	

Comparison of max. story displacement in confined masonry, masonry with lintel and unreinforced masonry is as shown in fig below:

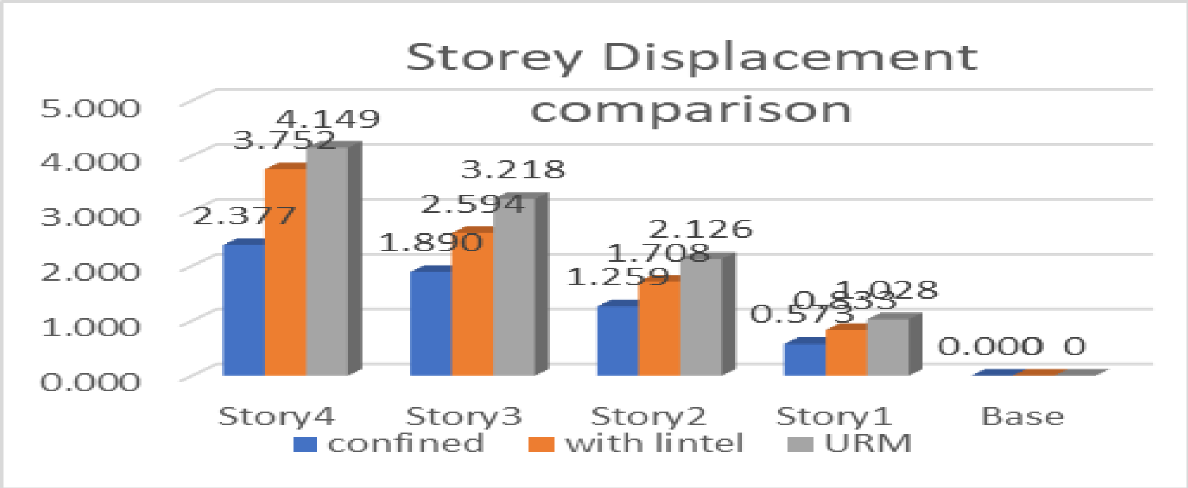


Figure 18: Comparison of storey displacements of URM, CM and masonry with lintel band

For the same given condition's maximum displacement under earthquake according to response spectrum analysis in URM is nearly 109% than that of CM and 26% more in masonry with lintel. Displacements in masonry provided with lintel band is found to be around 65% more than in case of confined masonry.

5.2.2.2 Storey Drifts

The storey drift ratio is the storey drift divided by the storey height. Story drift is the lateral movement of one level relative to the level above or below.

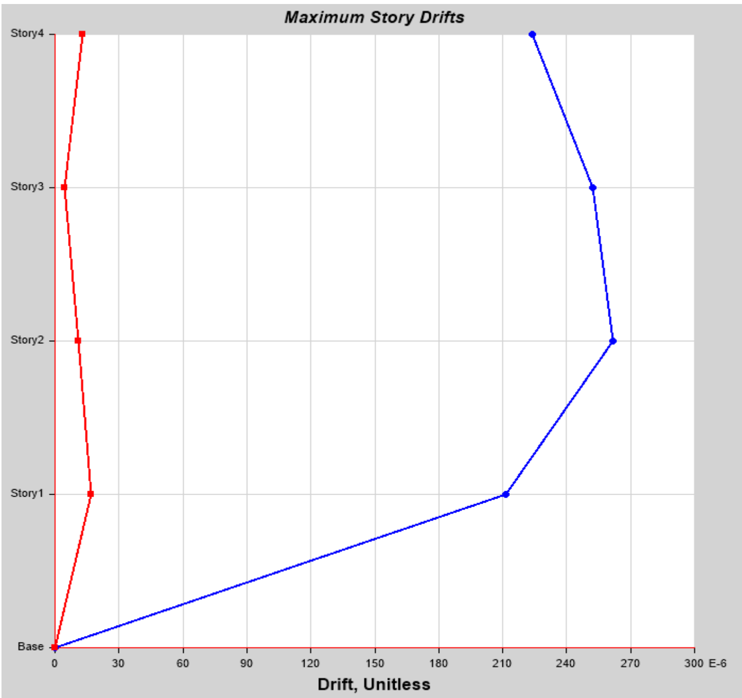


Figure 19:plot of storey drifts in Unreinforced masonry.

Table 14:Data Of Storey Drift in Unreinforced masonry Structure

Story Number	Storey Height	Drift(X)	Drift(Y)
	m		
4	12	0.000224	0.000013
3	9	0.000252	0.000004
2	6	0.000262	0.000011
1	3	0.000212	0.000017
Base	0	0	0

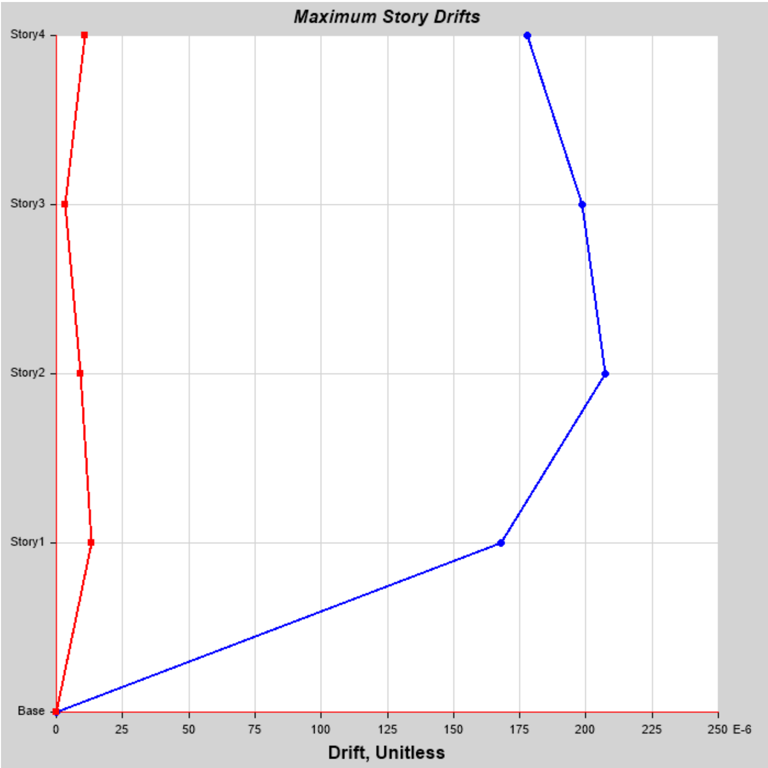


Figure 20:plot of storey drifts in masonry provided with lintel bands.

Table 15 Data Of Storey Drift in masonry Structure with lintel bands.

Story Number	Storey Height	Drift(X)	Drift(Y)
	m		
4	12	0.000178	0.000011
3	9	0.000199	0.000003
2	6	0.000207	0.000009
1	3	0.000168	0.000013
Base	0	0	0

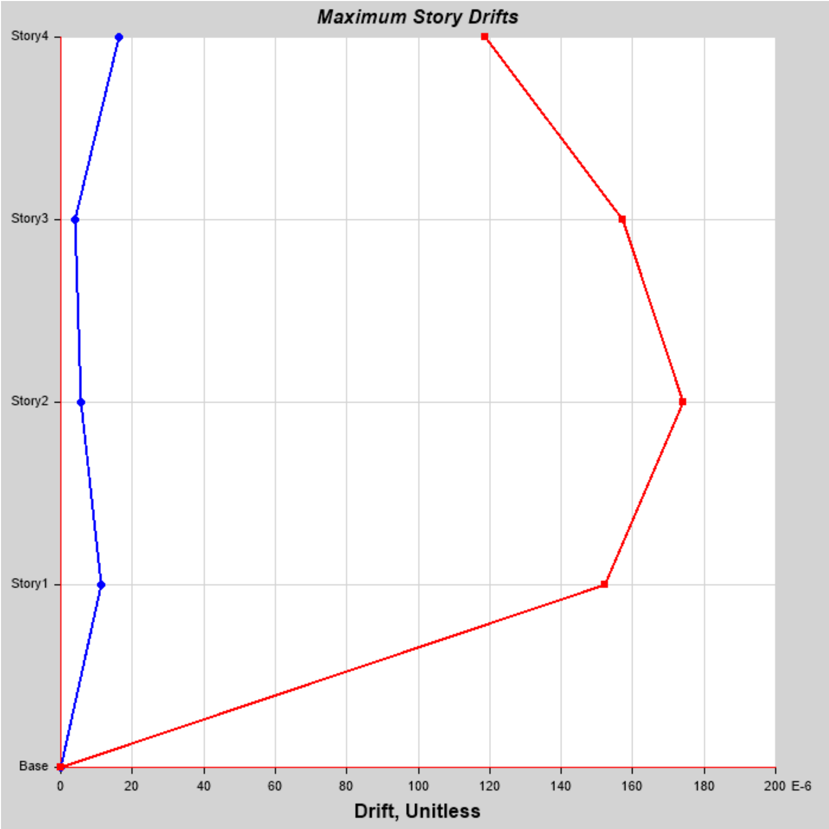


Figure 21:plot of storey drifts in confined masonry.

Table 16: Data Of Storey Drift in CM.

Story Number	Storey Height	Drift(X)	Drift(Y)
	m		
4	12	0.000016	0.000119
3	9	0.000004	0.000157
2	6	0.000006	0.000174
1	3	0.000011	0.000152
Base	0	0	0

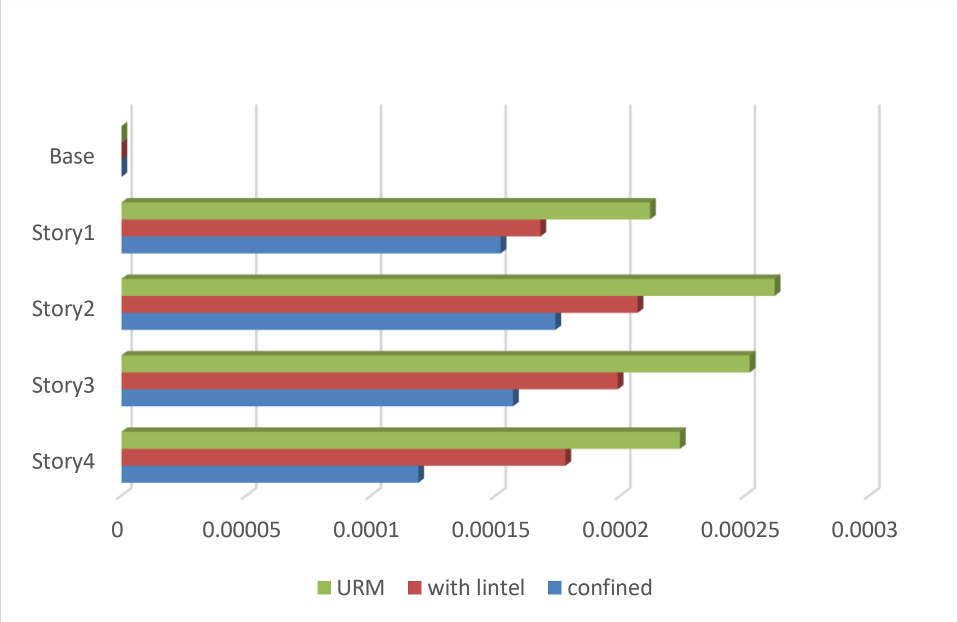


Figure 22: Relative comparison of storey drift in URM,CM and masonry with lintel band.

5.2.2.3 Storey stiffness

The lateral force causing unit translational lateral deformation in that storey is estimated as the storey stiffness, with the bottom of the storey prevented from moving laterally, i.e., only translational motion of the bottom of the storey is constrained while it is free to rotate. Stiffness can be useful as far as earthquake harm are considered since it diminishes the deformation load of construction.

Table 17: Storey stiffness of URM, CM and masonry with lintel band as per response spectrum analysis

storey	Height	storey stiffness in kN/m		
		URM	WITH LINTEL	CM
Story4	12	1268347.105	1332354.011	2097926.572
Story3	9	2243943.096	2265537.504	2842218.118
Story2	6	3014901.694	2967662.705	3170330.469
Story1	3	4377192.671	4395684.598	3873913.109
Base	0	1268347.105	1332354.011	2097926.572

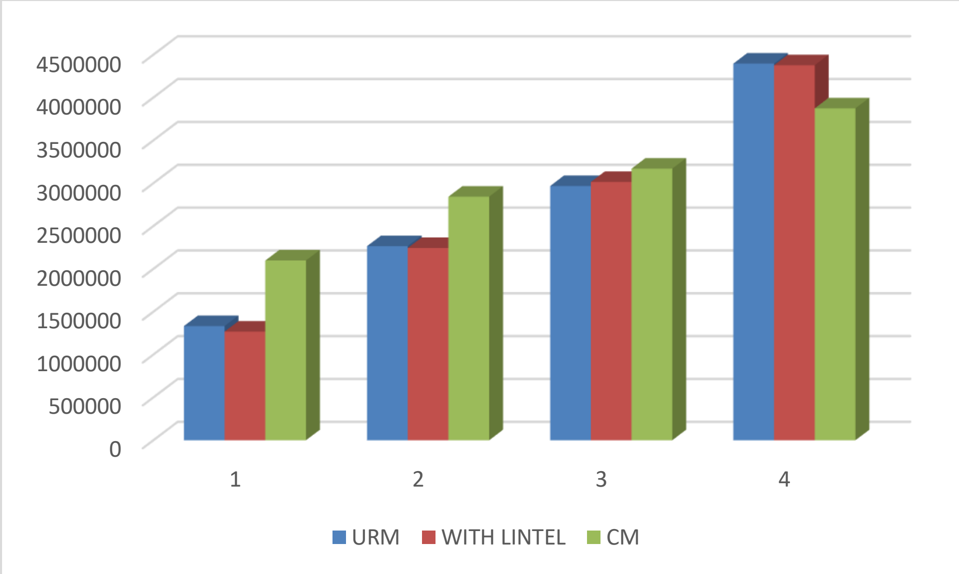


Figure 23: Comparison of storey stiffness of Models

Its can be seen that storey stiffness is comparatively in URM structure that is 1268347.105 KN/m for storey 4, while it increases when we provide lintel in masonry to 1332354.011 KN/m and it can still be increased by confining the walls with tie elements to 2097926.572 kN/m. Hence, CM can be preferred type of construction as it has least amount of deformation and possess high degree of stiffness for same given conditions.

5.2.2.4 Storey forces And Base shear

Table 18: Storey shear and Base shear value of models

storey	Height	Storey Shear in KN			
			URM	With lintel	CM
Story4	12	Top	-823.5854	-630.4069	-425.34381
		Bottom	823.5854	639.185	425.34381
Story3	9	Top	-1682.25	-1324.0955	-907.83447
		Bottom	1682.25	1330.4465	907.834474
Story2	6	Top	-2258.4867	-1769.5277	-1247.7571
		Bottom	2258.4867	1773.1445	1247.75709
Story1	3	Top	-2527.6448	-1961.0768	-1421.87
		Bottom	2527.6448	1962.0879	1421.86996

Base shear in URM was found to be maximum which is 2527.6448 kN, while in lintel it was 1962.0879 kN and in CM was least that is 1421.86996kN by response spectrum method of analysis. So, confining walls with the help of bond ties and columns redistributes load leading to lower base shear in CM.

5.2.3 Time History Method of analysis

Depending on the time function employed, time history analysis can examine dynamic structural reaction under stresses in a linear or nonlinear manner. It's a vital approach for structural seismic analysis, especially when the nonlinear structural reaction is being examined. To perform such an investigation, a representative earthquake time history for the structure being evaluated is required. Analysing the past entails, a thorough analysis of the dynamic behaviour of a structure intended to sustain a certain load that may vary over time. The data from the 2015 Nepal earthquake was utilised to generate time history data, the wave form of which is depicted in fig.:

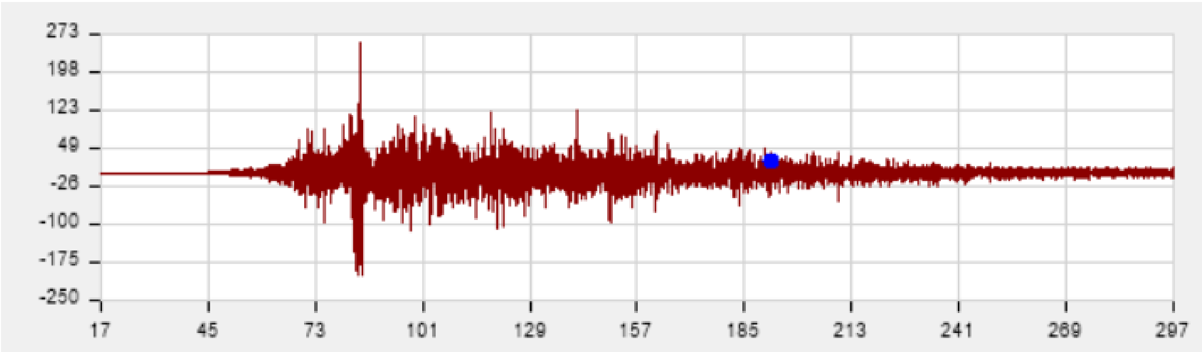


Figure 24:Acceleration time data of Nepal earthquake (2015) used for time history method of analysis.

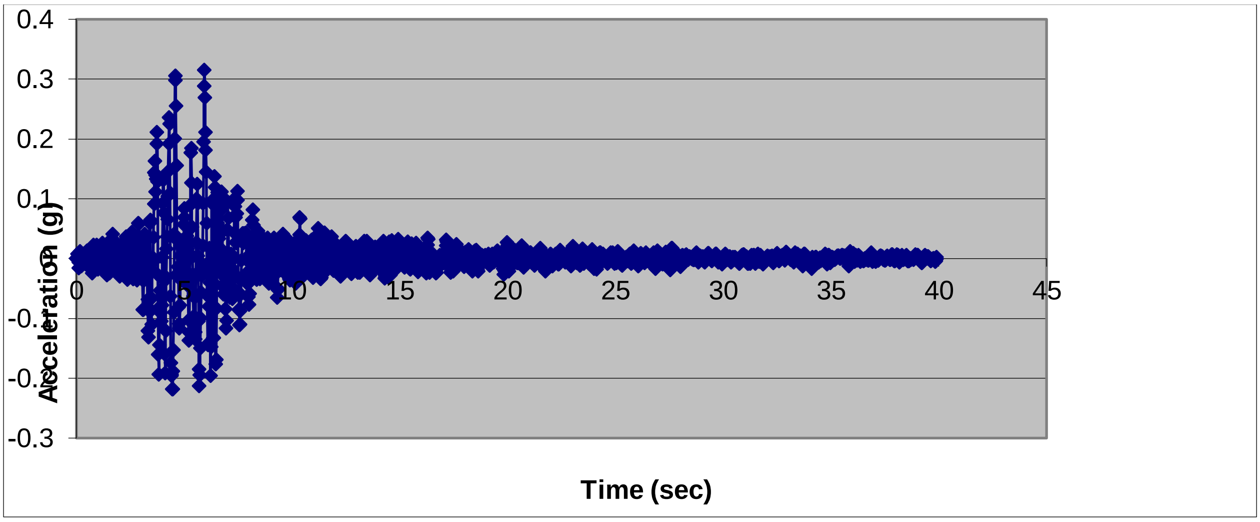


Figure 25:Acceleration time data of Uttarkashi earthquake used for time history method of analysis.

5.2.3.1 Storey Displacements

The movement of the storey relative to the ground during an earthquake is referred to as storey displacement. Extreme displacements can cause cracks, and excessive deflection is not psychologically acceptable

Table 19:Displacments by Nepal earthquake data

Story	URM	With Lintel Band	CM
	mm	mm	mm
Story4	4.60577517	3.9865	2.37746124
Story3	3.57160815	3.08686	1.88993112
Story2	2.35960803	2.03252	1.2587586
Story1	1.14160947	0.99127	0.57301596
Base	0	0	0

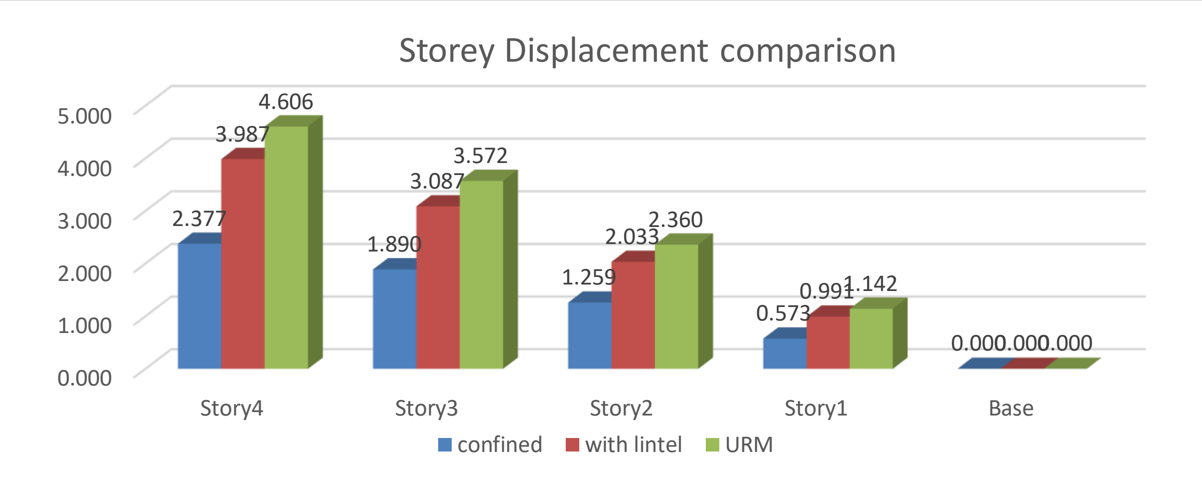


Figure 26: Comparison of maximum storey displacements of models by Nepal earthquake.

Table 20: Displacements by Uttarkashi earthquake data

Story	URM	With Lintel Band	CM
	mm	mm	mm
Story4	4.33	3.71	2.33
Story3	3.36	2.87	1.85
Story2	2.22	1.89	1.23
Story1	1.07	0.92	0.56
Base	0	0	0

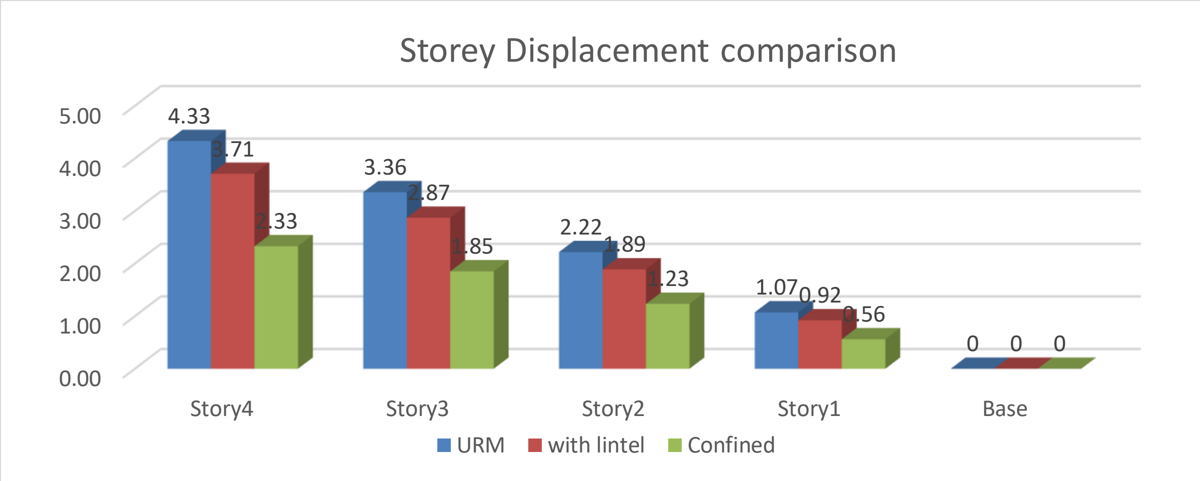


Figure 27:Comparison of maximum storey displacements of models by Uttarkashi earthquake.

5.2.3.2 Storey Drifts

The lateral displacement of one level relative to the level above or below is referred to as storey drift, and the storey drift divided by the storey height is referred to as the storey drift ratio.

Table 21: Storey drifts of Models as per time history method of analysis

Story Number	Drift		
	CM	With Lintel	URM
4	0.000132	0.000125338	0.000261
3	0.000168	0.000173517	0.000286
2	0.000181	0.000209185	0.000277
1	0.000131	0.000190531	0.000217
Base	0	0	0

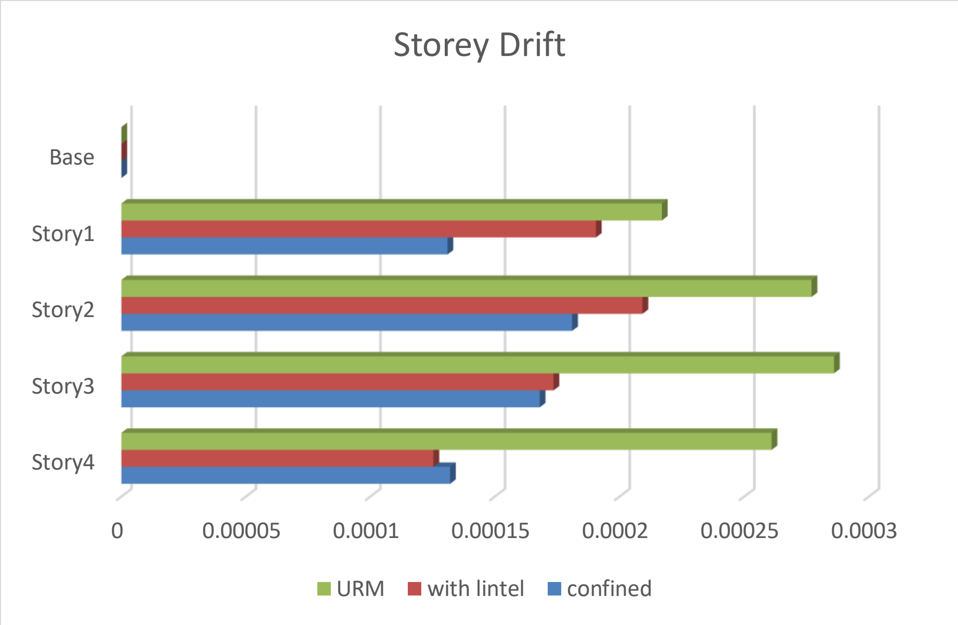


Figure 28: Storey drift comparison of models by time history method of analysis.

Above table shows that URM has highest value of storey drift followed by masonry with lintel and story drift is least in confined masonry.

5.2.3.3 Storey forces And Base shear

storey	Height	Storey Shear in KN			
			URM	With lintel	CM
Story4	12	Top	-983.0325	-675.0604	-690.4907
		Bottom	983.0325	686.6005	690.4907
Story3	9	Top	-1881.9893	-1471.9599	-1514.7983
		Bottom	1881.9893	1480.9347	1514.7983
Story2	6	Top	-2440.1274	-2024.2494	-1940.4812
		Bottom	2440.1274	2029.778	1940.4812
Story1	3	Top	-2517.8689	-2285.4331	-1787.4598
		Bottom	2517.8689	2287.1798	1787.4598

Base shear: It's the total lateral force exerted on the building at its base, which is equal to the bottom storey's storey shear. Base shear in URM was found to be 2517.8689kN, while in lintel it was 2287.1798kN and in CM was least that is 1672.79kN by time history method of analysis.

5.2.4 Pushover analysis

Pushover analysis is a non-linear static technique that gradually increases the amplitude of the lateral load while maintaining a predetermined distribution pattern throughout the building's height. Throughout the operation, the sequence of cracking, plastic hinge failure, and structural component failure may be witnessed. Until it falls, the structure is distorted. In all pushover investigations a curve is drawn that shows the connection between base shear and displacement. The curve is often known as the pushover curve and is the most important component of the nonlinear pushover analysis. “The seismic demand is then compared to the relevant structural capacity or preset performance limit state for the structure” FEMA 356 (2000) [13]

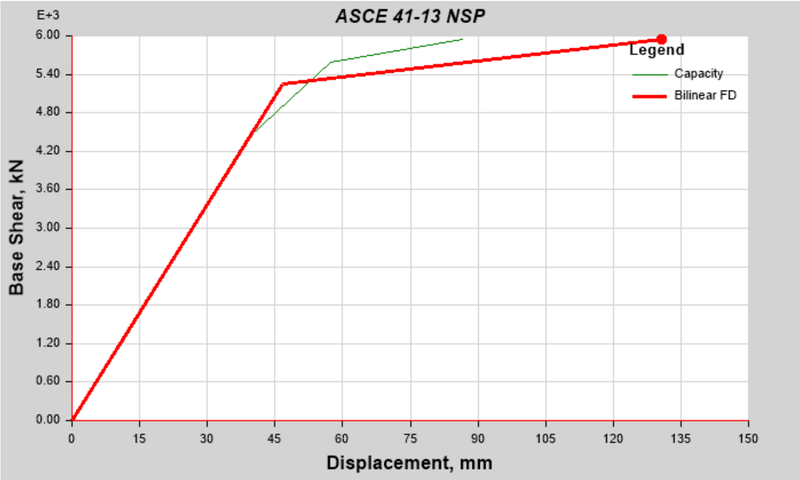


Figure 29: Pushover Curve according to ASCE41-13 of confined masonry model on Etabs.

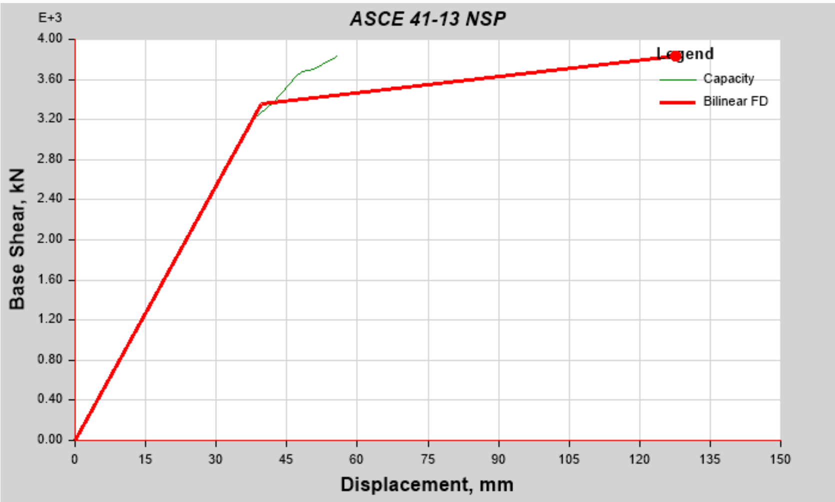


Figure 30:Pushover Curve according to ASCE41-13 of masonry with lintel band model on Etabs.

Confined masonry and masonry with lintel band model was also subjected to controlled displacement pushover analysis for up to 300 mm displacement. The Pushover analysis was not performed as to perform pushover analysis of under reinforced masonry it has to be converted to equivalent frame model and then that frame is subjected to pushover analysis of confined masonry and lintel band model is as shown above, confined masonry pushover results suggest that building fails at displacement of 49.58 mm at shear of 4634.88 kN, while that of masonry with lintel band shows it at 27.48 mm at shear of 3348.20kN.

5.3 Observation

- Storey displacement of Unreinforced masonry was found to be 108% higher than in the confined masonry and 65.06% greater than in the case where masonry is provided with the lintel bands, on comparing displacements of masonry with lintel bands and confined masonry the displacements in former was found to be 26.38% more than the later according to response spectrum analysis. Hence, for the same loading and lateral conditions confined masonry was found to show less deflection than both masonry with lintel bands and unreinforced masonry.

- As per time history method of analysis storey displacement of Unreinforced masonry was found to be 74.51% higher than in the confined masonry and 49.67% greater than in the case where masonry is provided with the lintel bands, on comparing displacements of masonry with lintel bands and confined masonry the displacements in former was found to be 16.60% more than the later. Hence, for the same loading and lateral conditions confined masonry was found to show less deflection than both masonry with lintel bands and unreinforced masonry, as observed by both method of analysis.

- Storey drift in Unreinforced masonry was observed to be 88.24% more than confined and 25.84% more when compared to masonry provided with lintel band according to response spectrum method of analysis.

- Storey drift in Unreinforced masonry was observed to be 97.73% more than confined and 108.24% more when compared to masonry provided with lintel band according to response spectrum method of analysis.
- Base shear values were maximum in case of unreinforced masonry and 9.16% more than masonry with lintel band and 40.86% more than confined masonry, masonry with lintel band had more base shear than in the case of confined masonry with the difference of 27.96% on basis on analysis as per response spectrum method of analysis.
- Base shear values was maximum in case of unreinforced masonry and 28.82% more than masonry with lintel band and 77.77% more than confined masonry, masonry with lintel band had more base shear than in the case of confined masonry with the difference of 37.99% on basis on analysis as per time history method of analysis.
- Storey displacement as per time history was found to be more than those obtained from response spectrum in all the cases, it was found to 32.06 percent more in case of confined masonry, around 19.75% more in case of masonry with lintel band and 10.48% more in the unreinforced masonry.
- Storey drift obtained from time history method was found to be 10.924% more in case of confined masonry, it was 29.58% less in case of masonry with lintel band and 16.51% more in case of unreinforced masonry, compared to subsequent values obtained from response spectrum analysis.
- As per response spectrum analysis, it can be seen that storey stiffness is comparatively

in URM structure that is 1268347.105 KN/m for storey 4, while it increases when we provide lintel in masonry to 1332354.011 KN/m and it can still be increased by confining the walls with tie elements to 2097926.572 kN/m. Hence, CM can be preferred type of construction as it has least amount of deformation and possess high degree of stiffness for same given conditions.

- Base shear in URM was found to be maximum which is 2527.6448 kN, while in lintel it was 1962.0879 kN and in CM was least that is 1421.86996kN by response spectrum method of analysis. Base shear in URM was found to be 2517.8689kN, while in lintel it was 2287.1798kN and in CM was least that is 1672.79kN by time history method of analysis.

- Base shear values obtained from time history are found to be greater than that from response spectrum by 0.38%, 14.21% and 20.45% in unreinforced, masonry provided with lintel bands and confined masonry respectively.

- Performance point from pushover analysis of confined masonry and masonry lintel band was observed at 49.58 mm at shear of 46.34.88 kN and 27.84 mm at shear of 3348.20 kN, showing that confined masonry is well capable of undergoing larger deflections before collapsing, confined undergoes 78.08% more displacements before collapsing then the masonry with lintel band.

CHAPTER-6

6.1 Conclusion

- About 80% of buildings in our country is still made of non-Engineered masonry type, which is not safe during earthquake. From observed analysis results it can be seen that by observing values of displacements, base shears and drift of stories values, confined masonry has been found to perform better under seismic conditions compared to that of masonry in which lintel bands has been provided at suitable location and underenforced masonry for same amount of vertical gravity and seismic lateral loads
- Tie columns and beams help providing ductility and better load distribution characteristics to masonry walls in confined masonry
- Confined Masonry can sometimes be efficient as reinforced concrete structures, and so confined masonry technique can be used for construction of small-scale residential buildings.
- According, to other studies cost of confined masonry is also not much and it can perform nearly as good as reinforced concrete structures, and so confined masonry technique can be used for construction of small-scale residential buildings.
- Confining masonry improves ductility of structures and hence, confined masonry can take much more load upto failure than masonry with lintel and unreinforced masonry.

6.2 Further Scope

- Macro model has been used in the above study, masonry being a homogenous material is made up of bricks and mortar unit which both have different behavior and properties, micro modelling can be used for enhanced and better understanding of behavior of masonry

- Masonry has non-linear and brittle behavior which can be better understand and analyzed by better FEM software such as ANSYS, MIDAS etc.

- Cost to benefit ratio and Comparison of Confined masonry, RCC with infill and unreinforced masonry can be studied for techno-economical purposes.

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