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SIESMIC EVALUATION OF UNREINFORCED MASONRY STRUCTURES

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ABSTRACT

It is common knowledge that masonry structures sustain severe damage and incur a significant number of fatalities during earthquakes. The majority (more than 70 percent) of the fatalities associated with the earthquake in the 20th century were caused by building collapse, with masonry buildings accounting for the majority of these fatalities. In India, the vast majority of tenements are Unreinforced Masonry (URM) structures, which are unstable and susceptible to even mild earthquakes. On the other hand, a quick scan of the research on earthquake-resistant structures reveals that RC structures are the focus of the majority of studies. It is obvious that much more work needs to be done to comprehend masonry constructions that are vulnerable to dynamic loads caused by earthquakes. The primary goal of this thesis is to investigate the methods used in the literature to assess the seismic vulnerability of unreinforced masonry structures using linear and non-linear static and dynamic analyses, as well as to conduct experimental studies to determine the applicability of these techniques. As part of this research, an experimental programme has been run to accomplish the aforementioned goals. In-plane monotonic lateral stresses were evaluated on sixteen wall panels of varied sizes. During testing, a constant axial compressive load was kept on each specimen. For eight of the sixteen test specimens, a window opening was supplied at the designated location of the test specimen, and its in-plane monotonic lateral load behaviour was examined.The in-plane monotonic lateral load behaviour of four additional specimens with a door opening and a window opening was examined. Four complete walls without any openings were also examined, and their performance was compared to that of panels with openings of a comparable design. The results of the experiment are contrasted with those obtained using the current pushover analysis iii approach (ASCE/SEI 41-06) for URM buildings. The comparisons demonstrate that the ASCE/ uctures. Based on the experimental inquiry, a set of modifications are suggested for the pushover analysis of URM buildings. Comparing the proposed adjustments to the current pushover analysis approach (ASCE/SEI 41-06), they consistently perform better. An existing URM building in Guwahati, India (Zone V) is subjected to a model seismic evaluation utilising the equivalent static approach and response spectrum method (IS 1893: 2002), which is then followed by pushover aSEI 41-06 approach regularly overestimates the stiffness and strength of URM strnalysis in accordance with ASCE/SEI 41-06 with a suggested modification.

***KEYWORDS**: Siesmic, Evaluation, Unreinforced, Masonry, Structures*

INTRODUCTION

It is well known that masonry buildings suffer a great deal of damage during earthquakes. This is especially true for the unreinforced masonry (URM) buildings built in rural and semi-urban areas of developing countries. Fig. 1.1 shows a typical load bearing URM building. Many heritage buildings around the world are of old and thick walled masonry. Their value, historic, artistic, social or financial, is great and damage to them in anearthquake involves very costly repair.

Fig.1.1: Typical load bearing masonry construction for a residential building

Normally thick walled URM buildings were designed for vertical loads, since masonry has adequate compressive strength the structure behaves well as long as the loads are vertical. When such a masonry structure is subjected to lateral inertial loads during an earthquake, the walls develop shear and flexural stresses. The strength of masonry under these conditions often depends on the bond between brick and mortar. A masonry wallcan also undergo in-plane shear stresses if the lateral forces are in the plane of the wall. Shear failure in the form of diagonal cracks is observed due to this. However, catastrophic collapses take place when the wall experiences out-of-plane flexure. Thiscan bring down a roof and cause more damage. Fig. 1.2 shows typical failure of an URM building during 2010 Haiti earthquake.

Fig.1.2: Failure of an URM building during 2010 Haiti earthquake

Masonry buildings with light roof such as tiled roof are more vulnerable to out-of-plane vibrations since the top edge can undergo large deformations, due to lack of lateral restraint. Damage to masonry buildings in earthquakes may be influenced by four general categories: quality of materials and construction, connections between structural elements, structural layout and soil-structure interaction.

OBJECTIVE

i) To assess pushover analysis methodology prescribed in ASCE/SEI 41-06 for unreinforced masonry buildings through experimental investigation and to propose improvement if required

ii) To develop equivalent frame model for nonlinear analysis of URM building

iii) To carry out a case study of seismic evaluation of an existing URM building using the improved pushover analysis.

SCOPE OF THE STUDY

The present study is limited to medium strength clay brick masonry wall. Fly ash brick masonry, hollow block masonry, etc. are kept outside the scope of the present study.

URM wall with strip footing is considered in the study. In the computer model the footing is modelled with fixity.

Two-dimensional wall panels are used for experimental testing to define in-plane lateral load-deformation behaviour of the wall panel. Out-of-plane lateral strength of the wall is ignored in the present study as it is very small compared to in-plane lateral strength

METHODOLOGY

The steps undertaken in the present study to achieve the above-mentioned objectives are as follows:

a) Carry out extensive literature review, to establish the objectives of the research work.
b) Develop a computer model with nonlinear line elements to represents unreinforced masonry wall.
c) Carry out experimental program for following three types of masonry walls with varying dimensions: (i) solid wall, (ii) wall with window opening and (iii) wall with door and window opening to obtain lateral force versus top displacement relation.
d) Carry out nonlinear static (pushover) analyses of above mentioned different wall panels as per the proposed model with line elements considering nonlinear hinges as defined by ASCE/SEI 41-06.
e) Compare the lateral load deformation behaviour of the selected wall panels obtained from experimental investigation and pushover analysis.
f) Propose improved nonlinear hinge model to carry out nonlinear analysis of unreinforced masonry wall if required.
g) Carry out a detailed case study of pushover analysis on a typical unreinforced masonry building with proposed modelling approach and nonlinear hinges, if any.

LITERATURE REVIEW

There are a number of research papers and design guidelines found on the structural properties of unreinforced masonry buildings A number of studies were carried out by Jai Krishna and Chandra (1965) and Jai Krishna *et. al.* (1966). They studied the static in-plane strength of walls with and without reinforcement. They carried out the building analysis by considering the shear walls alone, with different parameters such as the aspect ratio of shear walls and size and location of openings in shear walls. Arioglu and Anadol (1973) refer to the several earthquakes in Turkey and point out that plain masonry buildings are most vulnerable to earthquake damage. They refer to the special indigenous technique of producing horizontal wooden reinforcement on both faces at some vertical intervals to prevent collapse of masonry structures. Such practices have been traditionally in vogue in Turkey.

Abrams (1992) examines the in-plane lateral load behaviour of un-reinforced masonry elements under monotonic and cyclic loading. He argues that although masonry is considered to be brittle it has considerable deformation capacity after the development of first crack. Several suggestions have been made to evaluate the masonry strength characteristics under seismic loading.

Bruneau (1994) makes a number of observations on the seismic performance of un-reinforced masonry buildings (URM). Some of the types of failures are listed as

a) Lack of anchorage between floor and walls

b) Anchor failure when joists are anchored to walls

c) In-plane failure

d) Out-of-plane failure

e) Combined in-plane

Among these he emphasis that URM buildings are most vulnerable to flexural our-of-plane failure. In-plane failure may not right away lead to collapse since the load carrying capacity of a wall is not completely lost by diagonal cracking. However, our-of-plane failure leads to unstable and explosive collapse. Sometimes an initial in-plane failure may weaken the wall and subsequent out-of-plane motion can lead to collapse.

Rai and Goel (1996) also studied the seismic strengthening of un-reinforced masonry piers with steel elements. They considered the in-plane behaviour of masonry piers. The strengthening system showed significant improvement in stiffness and ductility.

Scrivener (1996) has done a survey of the damage to old masonry buildings in earthquakesaround the world. He also reported the cause of the damage under four headings: quality of materials and construction, connections between structural elements, structural layout and soil- structure interaction.

Tomazevic (1999) and his colleagues carried out a large number of Earthquake Resistant Masonry Structures. He has discussed a number of concepts for designing earthquake resistant masonry and for retrofitting partially damaged masonry structures. The following concepts maybe mentioned;

a) Traditional stone masonry walls with horizontal RC bond beams connecting the walls around the building at vertical spacing of 1.0 m or 2.0 m depending on the expected seismic intensity.
b) Masonry confined in its own plane by RC bond beams and columns. The columns have to be connected to the walls through shear keys. The spacing of columns is not more than 4.0 m.

c) Vertical reinforcement is provided n grouted holes of hollow block masonry and small pockets inside brick masonry. Horizontal reinforcements in the shape of truss like arrangements are also provided in bed joints. There are Euro code specifications for such reinforcements.
d) Horizontal tie rods are provided as a retrofitting measure in grooves cut in the mortar, below the floor level, on both sides of a wall. They are anchored to steel plates at bothends of the wall.

SEISMIC EVALUATION METHODS

The following are the methods recommended for detailed seismic evaluation of buildings: (i) Linear static analysis – Equivalent static analysis, (ii) Linear dynamic analysis – Response spectrum analysis and (iii) Non-linear static analysis – Push-over analysis. It is recommended that all the above methods be performed sequentially for a proper assessment of the seismic vulnerability in a building. It may be noted that more rigorous analysis (nonlinear dynamic time- history analysis) is possible, but this is not recommended as it is more involved and time consuming and not recommended for normal building.

This section briefly explains the linear static and linear dynamic analyses as recommended in Indian Standard IS 1893: 2002. The main purpose of these analyses, from the seismic evaluation perspective, is to check the demand-to- capacity ratios of the building components and thereby ascertain code compliance. The non-linear static analysis (pushover analysis) is explained in the next section. The two different linear analysis methods recommended in IS 1893: 2002 are explained in this Section. Any one of these methods can be used to calculate the expected seismic demands on the lateral load resisting elements.

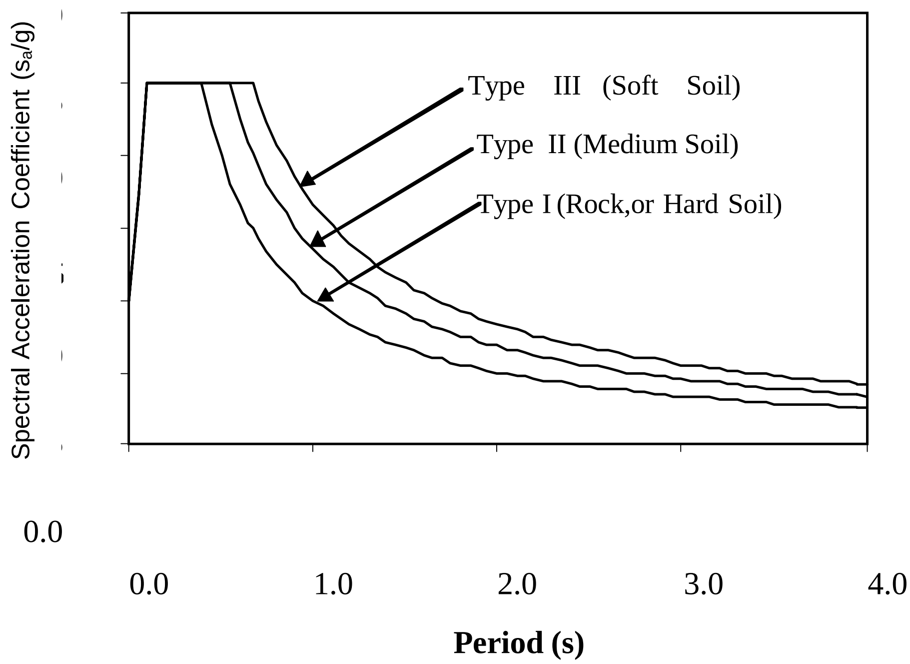


Fig. 2.1: Response spectra for 5 percent damping (IS 1893: 2002)

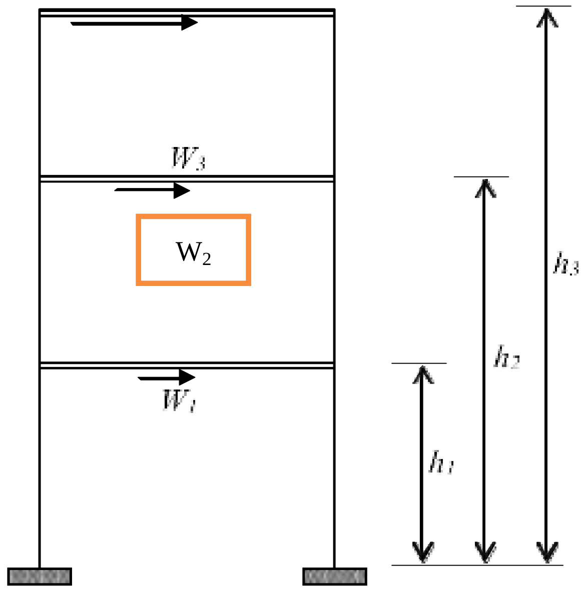
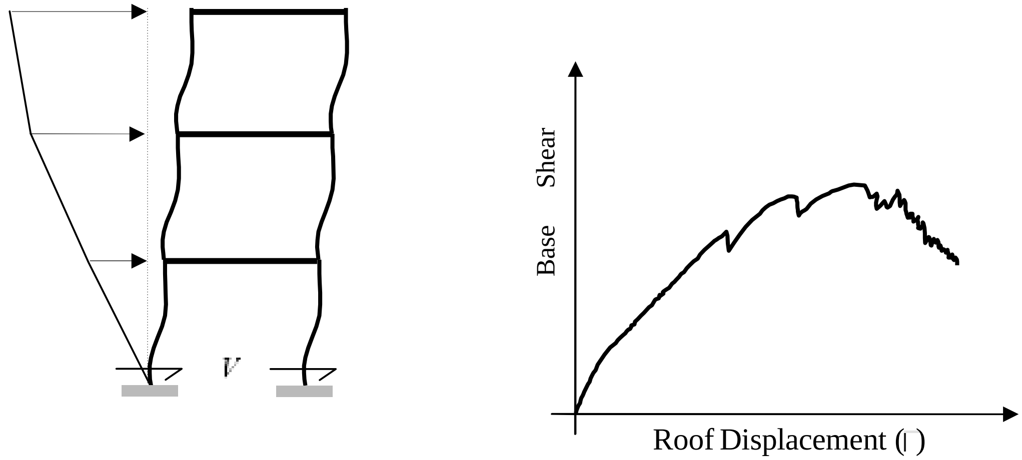


Fig. 2.2: Building model under seismic load

PUSHOVER ANALYSIS PROCEDURE

Pushover analysis is a static nonlinear procedure in which the magnitude of the lateral load is increased monotonically maintaining a predefined distribution pattern along the height of the building (Fig. 2.3a). Building is displaced till the ‗control node' reaches ‗target displacement' or building collapses. The sequence of cracking, plastic hinging and failure of the structural components throughout the procedure is observed. The relation between base shear and control node displacement is plotted for all the pushover analysis (Fig. 2.3b). Generation of base shear– control node displacement curve is single most important part of pushover analysis. Thiscurve is conventionally called as pushover curve or capacity curve. The capacity curve is thebasis of ‗target displacement' estimation as explained in Section 2.4.3. So the pushover analysis may be carried out twice: (a) first time till the collapse of the building to estimate target displacement and (b) next time till the target displacement to estimate the seismic demand. The seismic demands for the selected earthquake (storey drifts, storey forces, and component deformation and forces) are calculated at the target displacement level. The seismic demand is then compared with the corresponding structural capacity or predefined performance limit state to know what performance the structure will exhibit. Independent analysis along each of the two orthogonal principal axes of the building is permitted unless concurrent evaluation of bi- directional effects is required.



a) Building model b) Pushover curve Fig. 2.3:
Fig. 2.3: Schematic representation of pushover analysis procedure

CAPACITY SPECTRUM METHOD (ATC-40)

The basic assumption in Capacity Spectrum Method is also the same as the previous one. That is, the maximum inelastic deformation of a nonlinear SDOF system can be approximated from the maximum deformation of a linear elastic SDOF system with an equivalent period and damping. This procedure uses the estimates of ductility to calculate effective period and damping. This procedure uses the pushover curve in an acceleration-displacement response spectrum (ADRS) format. This can be obtained through simple conversion using the dynamic properties of the system. The pushover curve in an ADRS format is termed a _capacity spectrum‘ for the structure. The seismic ground motion is represented by a response spectrum in the same ADRS format andit is termed as demand spectrum

TEST SPECIMENS

Sixteen wall panels of varying dimensions were tested for in-plane monotonic lateral loads. For each specimen the axial compressive load was maintained as a constant during testing. A window opening at prescribed location of the test specimen was provided for eight of the sixteen specimens and its in-plane monotonic lateral load behaviour was studied. Four additional specimens with a door opening in combination with a window opening were tested for their in-plane monotonic lateral load behaviour. Four solid walls with ut any opening were also tested and compared with the behaviour of similar panels with openings.

Deta ls of the test specimens are shown in the Table 3.1. The objective was to study the effect of the presence of opening on the panel behaviour, when subjected to monotonic lateral loading. In the table, _S‘ stands for Solid Wall specimen whereas _W‘ and _D‘ denotes for the window and door opening respectively. Location for the opening in the specimen is defined by the parameters c, d, and e as shown in the Fig. 3.1. Sizes of the window opening are defined by the width (a) and height (b) and this for door opening is defined by Width (w) and height (h).

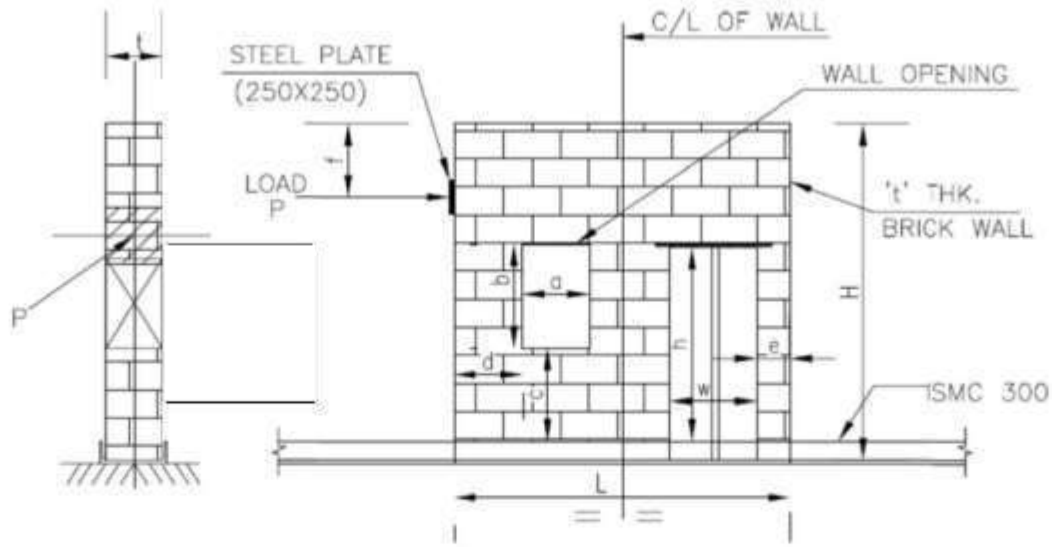


Fig. 3.1: Details of a typical sample

BUILDING DESCRIPTION

An existing load bearing unreinforced masonry building located in Guwahati (seismic zone V) presented in this paper. Figs. 4.1 and 4.2 show the typical floor plan and 3D computer model ofthe building respectively. It is a two storey residential buildings (2×3.2m height from the ground level) with door and window openings. Plan dimensions of the building are 11.4m × 9.5m. Standard brick of size 230mm × 110mm × 75mm and mortar grade of M1 (IS 1905:1987) were used for the construction of the building using Flemish Garden wall bond (IS 2212:1991). The building is approximately five years old. Thickness of all the outer walls is 230mm and all inner walls are of 110mm thick. The slabs are 150mm thick for all the floor levels in the buildings. Visual inspection did not reveal any deterioration in buildings. The sub-soils were assumed to be medium (Type II) as geotechnical data were not available. Walls were supported on 350 thick and 1000mm deep brick wall.

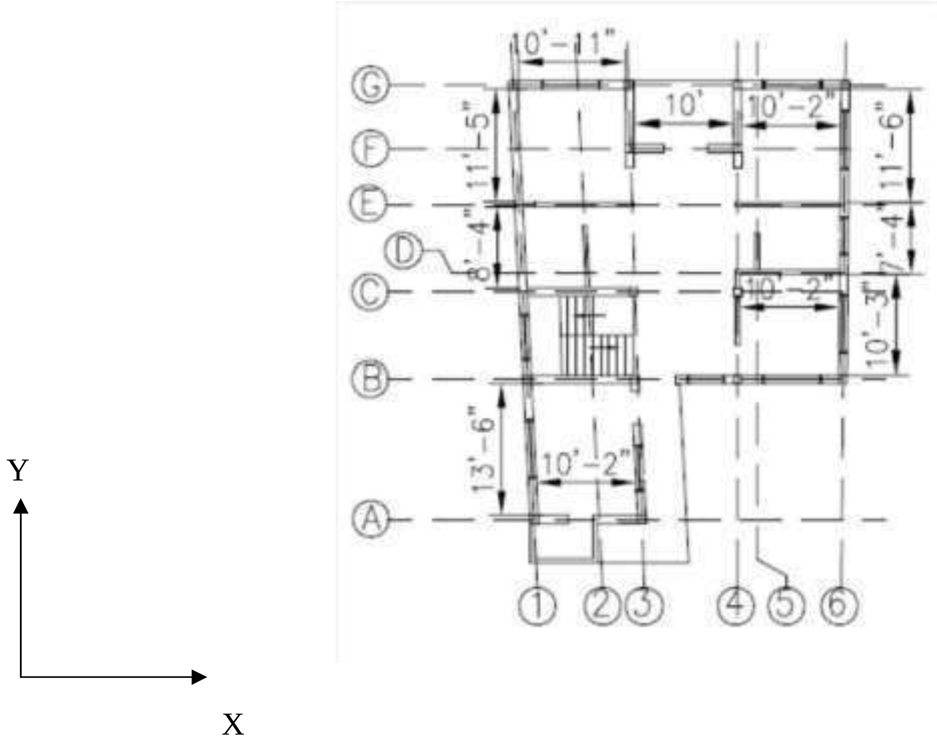


Fig. 4.1**:** Typical floor plan with the gridlines of the Building

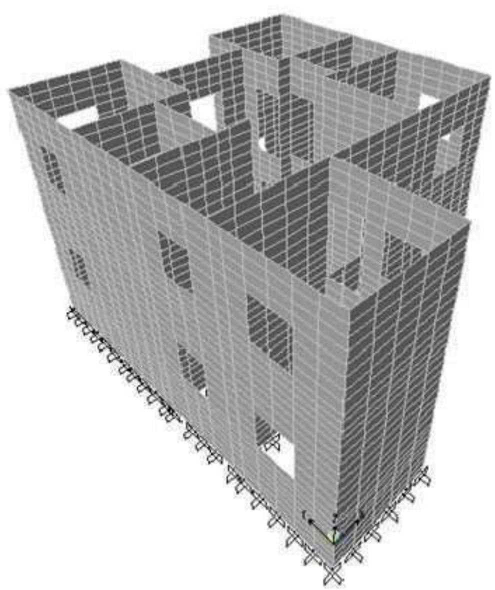


Fig. 4.2**:** 3D computer model of the Building

RESULTS AND DISCUSSIONS

The building model was analysed using Equivalent Static Method (linear static method) and Response Spectrum Method (linear dynamic method) according to IS 1893:2002. Pushover Analysis (nonlinear static method) was also carried out. The pushover analysis provides an insight into the structural aspects which control the performance during earthquakes. It also provides data on the strength and ductility of a building. The analyses were done by using the finite element analysis software, SAP2000. All the three analyses expose various design weaknesses that are present in a building.

To evaluate the performance of this building, a performance based approach was adopted. The performance based approach identifies a target building performance level under an anticipated earthquake level. The building performance is broadly categorized under the levels of (a) collapse prevention, CP, (b) life safety, LS, and (c) immediate occupancy, IO. The two commonly used earthquake levels are design basis earthquake (DBE) and maximum considered earthquake (MCE). For the present buildings, CP under MCE was selected as the safety objective.

Table 4.1: Time periods and modal participation for the first three modes

Mode	Natural Period (s)	Mass Participation Ratio (%)	
		UX	UY
1	0.075	43	19
2	0.069	22	46
3	0.054	02	04

Table 4.1 provides the period and the predominant direction of vibration for the first three modes of the building as obtained from the modal analysis of the elastic model. The table also shows the percentage of mass participation for each of the three modes. It is clear from the table that all the three modes are coupled translational-torsional mode.
This is due to the irregul shape of the building in plan and irregular opening distribution in the wall.

As the base shear found in response spectrum analysis (V_B) is lesser than design base shear ($\bar{V}_B$) as per IS 1893:2002, shear stress demand from response spectrum analysis was scaled up by a factor equal to the ratio of the two base shears ($\bar{V}_B$/V_B). Table 4.2 shows the comparison between (V_B) and ($\bar{V}_B$).

Table 4.2: Comparison of Base Shear

	V_X (kN)	V_Y (kN)
Equivalent Static ($\bar{V}_B$)	575.8	575.8
Response Spectra (V_B)	236.94	237.52
$\bar{V}_B / V_B$	2.43	2.42

Table 4.3: Deficient walls in the building

Wall Grid		Shear strength(MPa)	Equivalent Static Analysis		Response spectrum analysis	
			Shear Demand (MPa)	DCR	Shear Demand (MPa)	DCR
X- Panels						
Ground Floor Walls	B3-B4	0.12	0.30	2.5	0.29	2.0
	D4-D6	0.14	0.21	1.5	0.23	1.6
	E1-E3	0.14	0.23	1.6	0.25	1.9
	F3-F4	0.12	0.35	3.0	0.37	3.2
1st Floor Walls	B3-B4	0.12	0.26	2.4	0.24	2.1
	D4-D6	0.14	0.14	1.2	0.12	1.1
	E1-E3	0.14	0.14	1.2	0.11	1.0
	F3-F4	0.12	0.30	2.9	0.26	2.5
Y- Panels						
Ground Floor Walls	A3-B3	0.15	0.21	1.4	0.25	1.7
	B4-C4	0.13	0.19	1.4	0.20	1.5
	B6-D6	0.14	0.41	2.9	0.49	3.5
	G1-E1	0.12	0.29	2.3	0.26	2.2

1st Floor Walls	A3-B3	0.15	0.21	1.7	0.16	1.3
	B4-C4	0.13	0.13	1.2	0.09	0.8
	B6-D6	0.14	0.34	2.8	0.31	2.6
	G1-E1	0.12	0.19	1.7	0.17	1.6

CONCLUSIONS

Based on the work presented in this paper following point-wise conclusions can be drawn:

Modelling walls with plate element performs well in linear analysis but it is difficult to model nonlinear element properties with the plate modelling. Hence the URM building has to be modelled with equivalent frame (line) element for the non-linear analysis. The wall portion in between two openings should be considered as pier and the portion above and below the opening should be considered as spandrel. Width of pier is the clear distance between adjacent openings and depth of the pier is the thickness of wall. Similarly depth of spandrel should be the depth of wall segment available above orbelow opening and thickness is same as wall thickness.

i) The total stiffness of the URM building is going to be altered (reduced) due to the frame modelling as the connectivity gets reduced in the frame model. To account for this reduction in stiffness Young‘s modulus of the material needs to be suitably modified in frame model to match the elastic modal properties of the URM building building. All other material constants can be kept similar to that of brick masonry.

ii) The piers and the spandrels should be modelled with cracked section modulus instead of gross section modulus. Cracked moment of inertia of URM wall is found to be 40% of the gross moment of inertia of the same section.

iii) Experimental results show that the pushover analysis procedure given in ASCE/SEI 41-06 for URM wall panels is un-conservative for strength and stiffness estimation.

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