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Analysis and Design of Earth Quake Resistant Masonry Building

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Abstract - *Masonry buildings are among the most widely used construction systems for housing worldwide. Brick and stone masonry structures are preferred due to their durability, fire resistance, thermal performance, and architectural flexibility. The easy availability of materials, cost-effectiveness, and adaptability to various environmental conditions make masonry construction common in rural, urban, and hilly regions. Despite these advantages, masonry buildings exhibit poor seismic performance. Post-earthquake investigations have consistently shown that masonry structures are highly vulnerable and have suffered extensive damage during major earthquakes such as Bhuj (2001), Chamoli (1999), Jabalpur (1997), Killari (1993), Uttarkashi (1991), and Bihar–Nepal (1988). A significant proportion of casualties in these events resulted from the collapse of low-strength, non-engineered masonry buildings.*

Although earthquake engineering knowledge has advanced over the past three decades, a comprehensive and standardized procedure for the seismic analysis and design of masonry buildings remains inadequately developed and insufficiently addressed in the Indian academic curriculum, despite nearly 90% of the population residing in masonry structures. This study presents a systematic procedure for the seismic analysis and design of a two-storeyed residential masonry building. The methodology is structured into distinct steps to enhance clarity and promote confidence in designing masonry buildings as engineered structures. The analysis and design are carried out using STAAD.Pro software.

Key Words**: Two Story Masonry Building, Seismic Performance Evaluation, Shear Walls, Reinforced Masonry Building, STAAD Pro V8i.**

1. INTRODUCTION

Recent seismic events in India and worldwide have demonstrated the significant vulnerability of masonry buildings to earthquake-induced lateral forces. Although masonry construction remains the most prevalent housing system due to its durability, thermal efficiency, fire resistance, cost-effectiveness, and availability of local materials, its inherent characteristics—such as low tensile strength, brittle behavior, and limited ductility—result in poor seismic performance. Post-earthquake damage assessments from Bhuj (2001), Chamoli (1999), Jabalpur (1997), Killari (1993), Uttarkashi (1991), and Bihar–Nepal (1988) indicate that the majority of structural failures and associated casualties were attributed to non-engineered, low-strength unreinforced masonry (URM) buildings. In India, masonry structures are generally designed in accordance with IS 1905, while seismic actions are evaluated as per IS 1893 (Part 1). However, comprehensive analytical methodologies for seismic evaluation and design of masonry buildings are still inadequately addressed in practice and academia, despite the high percentage of the population residing in such structures.

The seismic response of masonry buildings is governed by parameters such as material strength, stiffness characteristics, mass distribution, geometric regularity, boundary conditions, and foundation system. Improved seismic resistance can be achieved through the use of higher-grade cement-sand mortar, regular and symmetric plan configurations to minimize torsional irregularities, controlled distribution of openings to maintain shear capacity, and appropriate height-to-thickness ratios to prevent out-of-plane instability. Ensuring a continuous and reliable load path through diaphragms, shear-resisting walls, and foundation elements is critical for effective force transfer. Reinforced masonry shear walls function as vertical cantilever elements resisting in-plane shear, bending, and overturning moments, with boundary reinforcement enhancing ductility and energy dissipation capacity. This study presents a systematic procedure for the seismic analysis and design of a multi-storeyed masonry residential building, demonstrating the feasibility of treating masonry construction as an engineered structural system through analytical modeling and code-based design approaches.

2. LITERATURE REVIEW

Hongwei Qi, Xiaoning Huang (2011): Seismic Reinforcement of Existing Masonry Structure on Conceptual Design:

He Summarized that Earthquake-resistance behaviors of existing masonry structures are evaluated based on a certain masonry building. The existing masonry structure is evaluated in its defects of primary design and construction measurements, seismic bearing capacity and durability. The masonry building is reinforced for avoiding collapse due to the weak positions of its bottom and top layers, stair halls, short walls and wall corners. Evaluation results and the corresponding seismic reinforcement design are significant for safety of both existing and newly designed masonry structures in earth-quake areas. The strengthen building prolong its life, decrease the damage and collapse in severe and strong earthquakes, so as to protect human life and property.

Matthew J. DeJong(2009): Seismic Assessment Strategies for Masonry Structures:

He summarized that Masonry structures are vulnerable to earthquakes, but their seismic assessment remains a challenge. This dissertation develops and improves several strategies to better understand the behavior of masonry structures under seismic loading, and to determine their safety. The primary focus is on historic arched or vaulted structures, but more modern unreinforced masonry structures are also considered. Assessment strategies which employ simplified quasi-static loading to simulate seismic effects are initially addressed. New analysis methods which focus on stability or strength are presented, and the merits of these strategies are clarified. First, a new parametric graphical equilibrium method is developed which allows real-time analysis and illuminates the complex stability of vaulted masonry structures.

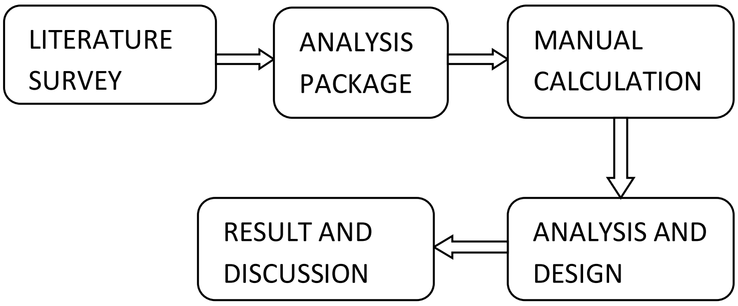
Willis, C. R.1 Griffith, M. C.2 Lawrence, S. J.3(2009): Earthquake Design of Unreinforced Masonry Residential Buildings up to 15 meters in Height:

This report presents deemed to satisfy solutions for earthquake loading of unreinforced masonry structures up to 15 m in height. The structures under analysis are Importance Level 2 buildings and are hence subject to Earthquake Design Category II (EDC II). This study considers the wall forces and associated actions due to earthquake loads. Out-of-plane bending tends to govern as wall span (L), site sub-soil class, hazard factor (Z) and the number of levels increase. This applies for both the office building and the home unit building. This finding is based on the failure criterion of the strength of walls in two-way bending being exceeded. Out-of-plane shear governs in relatively few cases and, when it occurs, it is in conjunction with out-of-plane bending and/or in-plane shear failure. There is no difference in this respect between the office building and the home unit building. This study sets out to identify which seismically induced actions are critical to the life-safety design objective embodied in the earthquake loading code. Out of-plane wall actions are related to the earthquake induced accelerations and in plane wall actions are related to the earthquake induced in-plane shear force. The 15 m height limit for the study means that typical structures up to and including five stories in height are considered. This also covers all domestic construction (Building Class 1a and 1b), whether less than or greater than 8.5 m tall.

C.V.R.Murthy(2012):IITK-BMTPC Earthquake Tips on the Behavior of Reinforced Concrete (RC) and Masonry Buildings

He summarized that Masonry buildings are brittle structures and one of the most vulnerable of the entire building stock under strong earthquake shaking. The large number of human fatalities in such constructions during the past earthquakes in India corroborates this. Thus, it is very important to improve the seismic behaviour of masonry buildings. A number of earthquake-resistant features can be introduced to achieve this objective. Ground vibrations during earthquakes cause inertia forces at locations of mass in the building. These forces travel through the roof and walls to the foundation. The main emphasis is on ensuring that these forces reach the ground without causing major damage or collapse. Of the three components of a masonry building (roof, wall and foundation) , the walls are most vulnerable to damage caused by horizontal forces due to earthquake. A wall topples down easily if pushed horizontally at the top in a direction perpendicular to its plane (termed weak direction), but offers much greater resistance if pushed along its length (termed strong direction). The ground shakes simultaneously in the vertical and two horizontal directions during earthquakes (IITK-BMTPC Earthquake Tip 5). However, the horizontal vibrations are the most damaging to normal masonry buildings. Horizontal inertia force developed at the roof transfers to the walls acting either in the weak or in the strong direction. If all the walls are not tied together like a box, the walls loaded in their weak direction tend to topple. To ensure good seismic performance, all walls must be joined properly to the adjacent walls. In this way, walls loaded in their weak direction can take advantage of the good lateral resistance offered by walls loaded in their strong direction. Further, walls also need to be tied to the roof and foundation to preserve their overall integrity.

3. METHODOLOGY



4. WORK PROCESS

4.1. BASIC DATA

I. Type of Building – Residential Building

II. Type of Structure – Two Storey Masonry Building
III. Floor to Floor Height – 3.5 m

4.2 PLAN OF RESIDENTIAL BUILDING

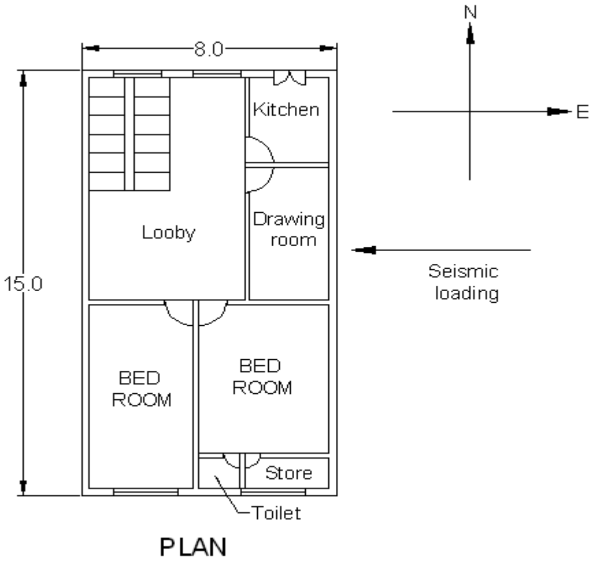


Fig1. Building Plan

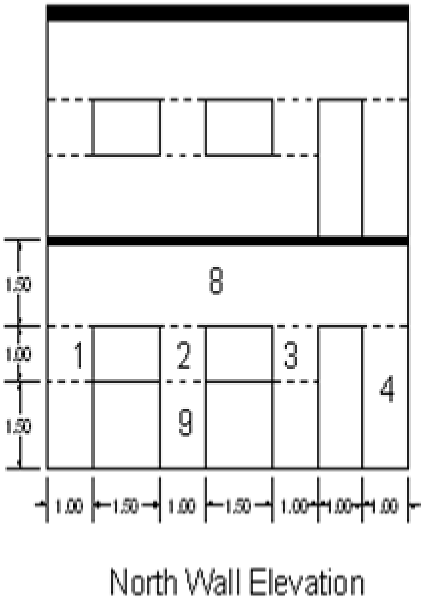


Fig2(a). North Wall Elevation

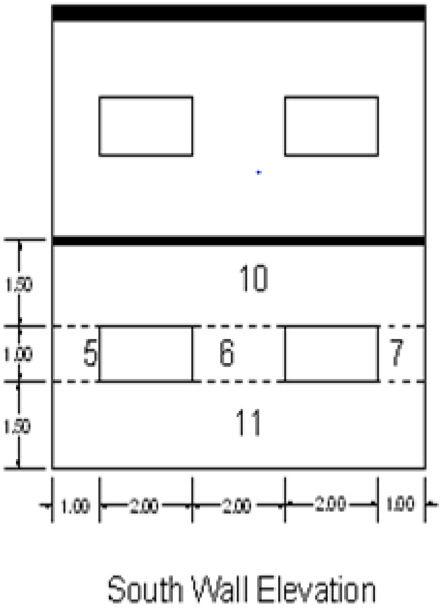


Fig2(b): South Wall Elevation

4.3 MATERIAL STRENGTH

i) Permissible compressive strength (fm) = 2.5 N/mm2
(Assuming unit strength = 35 MPa and mortar H1type)
In code IS 4326 : 1993 specifies that well burnt bricks and solid concrete bricks possessing a compressive / crushing strength < 35 MPa shall be used.

ii) Permissible stresses in steel in tension = 0.05 fy.
(Use high strength deformed bar (Fe 415) i.e. fy = 230 N/mm^2.)

4.4 LOAD

4.4 (a) LIVE LOAD DATA

Live load on roof = 1.0 kN/m2 (for seismic calculation = 0).
Live load on floor = 1.0 kN/m2

4.4 (b) DEAD LOAD DATA

Thickness of floor and roof slab = 120 mm.
Weight of slab = 3 kN/m^2.
(Assuming weight density of concrete = 25 kN/m^3) .
Thickness of wall = 250 mm.
Weight of wall = 5 kN/m^2.
(Assuming weight density of masonry 20 kN/m^3.)

4.4 (c) SEISMIC DATA

i) Seismic zone = V

ii) Zone factor (Z) = 0.36 , Zone factor given is for maximum considered earthquake (MCE) and service life of structure in a zone. The factor 2 is used so as reduce the maximum consider earthquake (MCE) zone factor to the factor for design basis earthquake

iii) Importance factor (I) = 1

iv) Response reduction factor (R) = 3 (as per IS 1893 (Part 1) : 2002)

v) Soil medium type, for which average response acceleration co-efficient are as

$$\frac{Sa}{g} = \begin{cases} 1 + 15T, & 0.00 \leq T \leq 0.10 \\ 2.50, & 0.10 \leq T \leq 0.55 \\ \frac{1.36}{T}, & 0.55 \leq T \leq 4.00 \end{cases}$$

vi)Direction of seismic force = E-W direction.

4.5 LOAD CALCULATION

STEP: 1 DETERMINATION OF DESIGN LATERAL LOAD

For determination of lateral load earthquake " Equivalent Static lateral forces procedure" is adopted.

i. SEISMIC WEIGHT CALCULATION

The seismic weight of each floor is it's full dead load plus appropriate amount of imposed load. While computing the seismic weight of each floor, the weight of walls in any storey shall be equally distributed to the floors above and below the storey. The weight of live load for seismic calculation is taken as zero.

Table -1: Seismic Weight Calculation

Description	**Load calculation**	**Total**
DL and LL load at roof level		
Weight of roof	3 x 8 x 15	360 kN
Weight of walls (Assume half weight of walls at second storey is lumped at roof)	1/2{2(8+15) x 4 x 5}	460 kN
Weight of live load (For seismic calculation, LL on roof is zero)	0 x 8 x 15	0 kN
(Wr) Weight at roof level	360+460+0	820 kN
DL and LL at floor level		
Weight of floor	3 x 8 x 15	360 kN
Weight of walls (Assume half weight of walls at second storey and half weight of walls at first storey lumped at roof).	2 x 1/2 { 2(8+15) x 4 x5}	920 kN
Weight of live load	1 x 8 x 15	120 kN
(Wf) Weight at second level	360+920+120	1400 kN
Total seismic weight of building (Wr) + (Wf)	**820 + 1400**	**2220 kN**

ii.TIME PERIOD CALCULATION

The approximate fundamental natural period of a masonry building can be calculated from the clause 7.6.2 of IS 1893 (Part 1) : 2002 as,

$$Ta = 0.09\, {}^{h}/_{\sqrt{d}}$$

where ,

h = height of building in m, {i.e. 4.0 (first storey) + 4.0 (second storey) = 8.0 m}

d = Base dimension of building at the plinth level in m, along the considered

direction of lateral force (i.e. 8m, assuming earthquake in E-W direction.

$$Ta = 0.09\, {}^{8}/_{\sqrt{8}} = 0.032\ sec$$

$$\frac{Sa}{g} = 2.5,\ for\ T = 0.032$$

$$Ah = \frac{ZISa}{2Rg} = (0.36|2)(1|3)(2.5) = 0.15$$

The total design lateral base shear (VB) along the direction of motion is given by

VB = AhW = 0.15 x 2220 = 333 kN.

iii.DISTRIBUTION OF BASE SHEAR TO DIFFERENT FLOOR LEVELS

The design lateral base shear (VB) is distributed along the height of building as per the following expression.

$$Qi = VB\,\frac{WiHi\,^\wedge 2}{\sum_{i=1}^{n} WiHi\,^\wedge 2}$$

Where ,

Qi = Design lateral force at floor I,

Wi = Seismic weight of floor I,

hi = Height of floor I measured from base.

n = Number of storeys in the buildings is the number of levels at which mass are located.

Lateral Force at Roof Level

$$Qi = VB\,\frac{WiHi\,^\wedge 2}{\sum_{i=l}^{n} WiHi\,^\wedge 2}$$

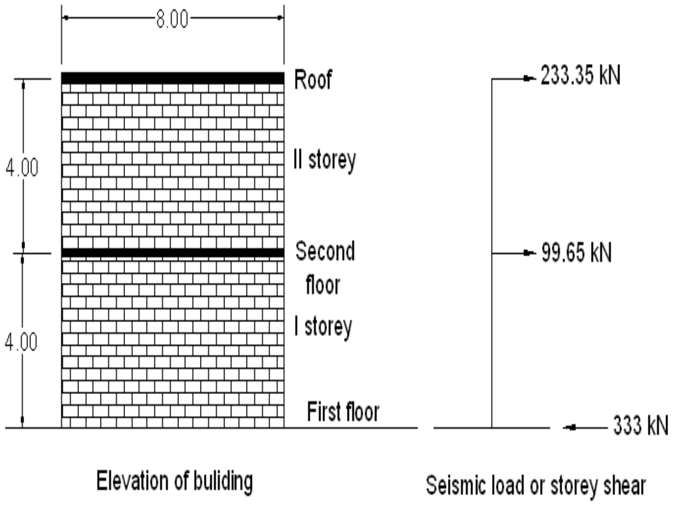
= (333x 820 x 82) / (820 x 82 + 1400 x 42)
= 233.35 kN

Lateral Force at Roof Level

$$Qi = VB\,\frac{WiHi\,^\wedge 2}{\sum_{i=l}^{n} WiHi\,^\wedge 2}$$

= (333 x 1400 x 42) / (820 x 82 + 1400 x 42)

= 99.65 kN



(a) (b)

Fig:4.5 (a) Elevation of Building (b) Seismic load or Storey shear

STEP: 2 DETERMINATIONS OF WALL RIGIDITIES

Rigidity of North Shear Wall

Δ_{wall} = $\Delta_{solid\ wall(c)}$ - $\Delta_{strip\ A(c)}$ + $\Delta_{1,2,3,9,4(f)}$

$\Delta_{1,2,3,9,4(f)}$ = 1/ $R_{1,2,3,9,4(f)}$

$R_{1,2,3,9,4(f)}$ = $R_{1,2,3,9(f)}$ + $R_{4(f)}$

$R_{1,2,3,9(f)}$ = 1/$\Delta_{1,2,3,9(f)}$

$\Delta_{1,2,3,9(f)}$ = $\Delta_{solid1,2,3,9(c)}$ - $\Delta_{stripB(f)}$ + $\Delta_{1,2,3(f)}$

$\Delta_{1,2,3(f)}$ = 1/($R_{1(f)}$ + $R_{2(f)}$ + $R_{3(f)}$)

Rigidity of cantilever pier is given by RC = E_t / $\{4(h/d)^3 + 3(h/d)\}$

Rigidity of fixed pier is given by R_f = E_t / $\{(h/d)^3 + 3(h/d)\}$

$R_{solid\ c)}$ = E_t / $\{4(4/8)^3 + 3(4/8)\}$

= 0.5 E_t

$\Delta_{solid(c)}$ = 2.0/ E_t

$R_{stripA(c)}$ = E_t / $\{4(2.5/8)^3 + 3(2.5/8)\}$

= 0.944 E^t

$\Delta_{stripA(c)}$ = 1.06/E_t

$R_{soild1,2,3,9(f)}$ = E_t / $\{(2.5/6)^3 + 3(2.5/6)\}$

= 0.756 E_t

$\Delta_{soild1,2,3,9(f)}$ = 1.322/ E_t

$R_{stripB(f)}$ = E_t / $\{(1/6)^3 + 3(1/6)\}$

= 1.98 E_t

$\Delta_{stripB(f)}$ = 0.546/E_t

($R_{1(f)}$ + $R_{2(f)}$ + $R_{3(f)}$) = E_t / $\{(1/1)^3 + 3(1/1)\}$

= 0.25 E_t

$\Delta_{1,2,3(f)}$ = 1.33 / E_t

$\Delta_{1,2,3,9(f)}$ = 1.322/ E_t – 0.5046/ E_t + 1.33/ E_t

= 2.15/ E_t

$R_{1,2,3,9(f)}$ = E_t/2.15 = 0.465 E_t

$R_{4(f)}$ = E_t / $\{(2.5/1)^3 + 3(2.5/1)\}$

= 0.043 E_t

$\Delta_{1,2,3,9,4(f)}$ = 1.968/ E_t

Δ_{wall} = 2.0/ E_t – 1.06/ E_t + 1.96/ E_t

= 2.908/ E_t

R_{wall} = 0.343 E_t

Rigidity of South Shear Wall

Δ_{wall} = $\Delta_{solid\ wall(c)}$ - $\Delta_{strip\ A2(c)}$ + $\Delta_{5,6,7(f)}$

$\Delta_{5,6,7(f)}$ = 1/ $R_{5,6,7(f)}$

$R_{5,6,7(f)}$ = $R_{5(f)}$ + $R_{6(f)}$ + $R_{7(f)}$

$R_{5(f)}$ = $R_{7(f)}$ = Et/$\{(1/1)^3 + 3(1/1)\}$

= 0.615 E_t

$R_{5,6,7(f)}$ = 2 x 0.25 E_t + 0.615 = 1.115 E_t

$\Delta_{5,6,7(f)}$ = 1/ $R_{5,6,7(f)}$ = 0.896/ E_t

$R_{solid(c)}$ = $E_t/\{4(4/8)^3 + 3(4/8)\}$

= 0.5 E_t

$\Delta_{solid(c)}$ = 2.0/ E_t

$R_{stripA2(c)}$ = $E_t/\{4(1/8)^3 + 3(1/8)\}$

= 2.612 E_t

$\Delta_{stripA2(c)}$ = 0.382/ E_t

Δ_{wall} = $\Delta_{solid\ wall(c)} - \Delta_{strip\ A2(c)} + \Delta_{5,6,7(f)}$

= 2/ E_t – 0.382/ E_t + 0.896/ E_t

= 2.513/ E_t

R_{wall} = 0.398 E_t

Relative Stiffness of Walls

North shear wall = 0.343/(0.343 + 0.398) = 0.462

South shear wall = 0.398/(0.343 + 0.398) = 0.538

STEP: 3 DETERMINATIONS OF TORSIONAL FORCES

Location of the center of Mass

Centre of mass, XCM and YCM is calculated by taking statical moments about a point, say southwest corner, using the respective weights of walls as forces in the moment summation. Because of symmetrical layout of building, the centre of the mass will occur near the centre of building i.e. XCM = 4.0m, YCM = 7.5m.However for methodology purpose the calculations for the centre of mass is shown in Table

Table – 2 : Calculation of centre of mass

	Weight i kN	**X(m)**	**Y(m)**	**WX(kNm)**	**WY(kNm)**
Roof slab	8 x 15 x 3 = 360 kN	4.0	7.5	1440	2700
N - Wall	8 x 4 x 5 = 160 kN	4.0	15	640	2400
S - Wall	8 x 4 x 5 = 160 kN	4.0	0.0	640	0
E - Wall	15 x 4 x 5 = 300 kN	8.0	7.5	2400	2250
W - wall	15 x 4 x 5 = 300 kN	0.0	7.5	0	2250
	ΣW = 1280			**ΣWX = 5120**	**ΣWY = 9600**

$\overline{\text{X}}$CM = ΣWX/ΣW = 4.0m from west wall

$\overline{\text{Y}}$CM = ΣWY/ΣW = 7.5m from east wall

Location of the Centre of Rigidity

The centre of rigidity, XCR and YCR, is calculated by taking statical moments about a point, say south-west corner using the relative stiffness of the walls as forces in the moment summation. The stiffness of slab is not considered in the determination of centre of rigidity. The calculation for the rigidity is shown in Table

Table – 3 : Calculation of Centre of Rigidity

Item	**RX**	**RY**	**X(m)**	**Y(m)**	**Y RX**	**X RY**
N - Wall	0.462	-	-	15	6.93	-
S - Wall	0.538	-	-	0.0	0	-
E - Wall	-	0.5	8.0	-	-	4.0
W - wall	-	0.5	0.0	-	-	0.0
	Σ RX = 1.0	**Σ RY =1.0**			**Σ Y RX = 6.93**	**Σ XRY = 4.0**

$\overline{\text{X}}$CR = Σ XRY/ΣRY = 4.0m from W- Wall

$\overline{\text{Y}}$CR = Σ YRX/ΣRX = 6.93m from S – Wall

Torsional Eccentricity

Torsional Eccentricity in Y – direction

Eccentricity between centre of mass and centre of rigidity

e_y = 7.50 – 6.72 = 0.78m

Add minimum 5% accidental eccentricity

= 0.05 x 15 = 0.75m

Total eccentricity = 0.78 + 0.75 = 1.53m

Torsional Eccentricity in X – direction

Eccentricity between centre of mass and centre of rigidity

e_x = 4.0 – 4.0 = 0.0 m

Add minimum 5% accidental eccentricity

= 0.05 x 8 = 0.40m

Total eccentricity = 0.0 + 0.40 = 0.40 m

Torsional Moment

The torsional moment due to E – W seismic force rotate the building in Y – direction, hence

$M_{TX} = V_x\, e_y$ = 333 x 1.53 = 509.50 kNm

Similarly, if considered seismic force in N – S direction

$M_{TY} = V_y\, e_x$ = 333 x 0.40 = 133.2 kNm

($V_y = V_x$,because Sa/g is considered value of 2.5 for the ime period $0.11 \le T \le 0.55$)

Distribution of direct shear force and torsional shear force

Since, we are considering the seismic force only in E – W direction, the in N – S direction will resist the forces and the walls in E – W direction may be ignored. Table shows the calculation of distribution of direct shear and torsional shear.

Table.4 - Distribution of forces in North and South shear walls

Item	Rx	dy (m)	Rx dy	Rx dy2	Direct shear force (kN)	Torsional shear for(kN)	Total shear (kN)
N - Wall	0.462	8.07	3.728	31.67	153.85	+33.94	187.80
S - Wall	0.538	6.93	3.728	24.30	179.15	-33.94	179.15
				Σ = 55.96			

Distance considered wall from centre of rigidity (15 – 6.93 = 8.07 m)

Torsional forces in N wall = $\frac{Rxdy}{\Sigma Rxdy^{\wedge}2} Vxey$

= (3.728 x 509.44)/55.96

= 33.94 kN

Torsional forces in S wall = $\frac{Rxdy}{\Sigma Rxdy^{\wedge}2} Vxey$

= (3.728 x 509.44)/55.96

= 33.94 kN

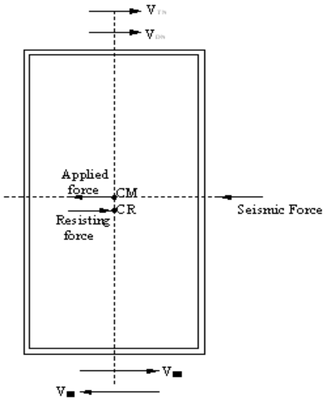


Fig – 3 : Torsional Forces

The torsional forces additive on the north wall and subtractive on the south wall as shown. Since the code directs that negative torsional shear shall be neglected. Hence the total shear acing on the south wall is simply direct shear only.

Distribution of the Total Shear to Individual Piers within the Wall

The shear carried by the north and south shear wall is now distributed to individual piers on the basis of their respective stiffness.

Table.5 – North Shear Wall

Pier Group	Stiffness	Relative stiffness	Shear Force
1,2,3,9	0.465	0.915	171.80 kN
Pier 4	0.043	0.085	15.96 kN

Shear171.8kN in pier group 1,2,3,9 is further divided in vertical piers 1, 2 and 3 in proportion to their stiffness. The stiffness of pier 1,2 and 3 are 0.25 each so the shear force carried by each pier is

Table.5 – North Shear Wall

Pier	Stiffness	Relative stiffness	Shear Force
1	0.25	0.33	56.70 kN
2	0.25	0.33	56.70 kN
3	0.25	0.33	56.70 kN
4	0.043	0.085	15.96 kN
5	0.25	0.225	40.30 kN
6	0.615	0.55	98.53 kN
7	0.25	0.225	40.30 kN

STEP: 4 DETERMINATION INCREASE IN AXIAL LOAD DUE TO OVERTURNING

Total overturning moment due to lateral force acting on the building is,

M_{ovt} = Total shear (V_x) x vertical distance between second floor level to critical

plane of weakness, assuming at the level of sill + applied overturning

moment at second floor level.

Assume the stiffness of second storey walls is the same as first storey, the total direct shear in E – W direction of seismic load i.e. in x – direction is divided in North and South shear wall in the proportion to their stiffness

Direct Shear in North wall (V_{NX}) = 153.85 kN

Direct Shear in South wall (V_{SX}) = 179.15 kN

Distribution of lateral force along the height of North and South wall is

North Shear Wall

Lateral force at roof level = $$V_{nx} \times \frac{W_r H_r{}^{\wedge}2}{\sum_{i=1}^{n} W_i H_i{}^{\wedge}2}$$

= 153.85(820 x 82)/{(820 x 82) + (1400 x 42)}

= 107.78 kN

Lateral force at second floor level = $$V_{nx} \times \frac{W_2 H_2{}^{\wedge}2}{\sum_{i=1}^{n} W_i H_i{}^{\wedge}2}$$

=153.85(1400x 42)/{(820 x 82) + (1400 x 42)}

= 46.07 kN

South Shear Wall

Lateral force at roof level = $$V_{sx} \times \frac{W_r H_r{}^{\wedge}2}{\sum_{i=1}^{n} W_i H_i{}^{\wedge}2}$$

= 179.15(820 x 82)/{(820 x 82) + (1400 x 42)}

= 125.56 kN

Lateral force at second floor level = $$V_{sx} \times \frac{W_2 H_2{}^{\wedge}2}{\sum_{i=1}^{n} W_i H_i{}^{\wedge}2}$$

= 179.15(1400x 42)/{(820 x 82) + (1400 x 42)}

= 53.59 kN.

Increase in axial load in piers of north shear wall :

Overturning moment in north wall (M_{ovt}) is

M_{ovt} = (Total shear at second floor x Critical height) +

(lateral load at roof level x Storey height)

Movt = (153.85 x 2.5) + (107.78 x 4) = 815.75 kN.m

Increase in axial load due to overturning moment

$$P_{ovt} = M_{ovt} \frac{l_i A_i}{I_n}$$

Where ,

l_{iAi} = Centroid of net section of wall.

I_n = Moment of inertia of net section of wall.

Table.6 – Calculation of centroid of net section of wall

Pier	**Area (Ai) m2**	**L m**	**Ail (m3)**
1	1 x 0.25 = 0.25	0.5 m	0.125
2	1 x 0.25 = 0.25	3.0 m	0.750
3	1 x 0.25 = 0.25	5.5 m	1.375
4	1 x 0.25 = 0.25	7.5 m	1.875
	Σ = 1.0		**Σ =4.125**

Distance from left edge to centriod of net section of wall = 4.125 / 1.0 = 4.125 m.

Table.7 – Calculation of moment of inertia of net section of wall

Pier	**Ai m2**	**li m**	**Ai li m3**	**Ai li2 m4**	**I = td3/12**	**In=I+ Ai li2**
1	0.25	3.625	0.906	3.285	0.02	3.305
2	0.25	1.125	0.281	0.316	0.02	0.326
3	0.25	1.375	0.344	0.472	0.02	0.492
4	0.25	3.375	0.844	2.848	0.02	2.868
	Σ = 1.0					**Σ=6.99=7m4**

Increase in axial load in individual piers of north wall is determined in table

Table.7 – Increase in axial load in individual piers of north wall

Pier	**A_i l_i m3**	**P_{ovt}= M_{ovt} x (A_i l_i / I_n) kN**
1	0.906	105.58
2	0.281	32.75
3	0.344	40.09
4	0.844	98.36

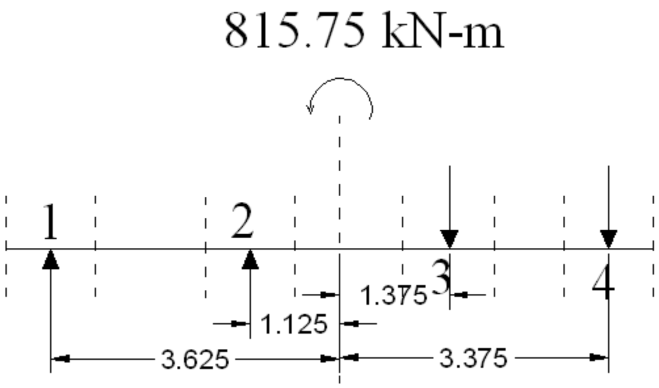


Fig -4: Over turning moment in pier of north wall

M_{ovt} = 815.75 kN.m

I_n = 7.0 m^4

Increase in axial load in piers of south shear wall :

Overturning moment in south wall (M_{ovt}) is,

M_{ovt} = (Total shear at second floor x Critical height) + (lateral load at roof level x storey height)

M_{ovt} = (179.15 x 2.5) + (125.56 x 4.0) = 950.12 kN.m

Increase in axial load due to overturning moment

$$Povt = Movt \frac{liAi}{In}$$

Where,

A_il_i = centriod of net section of wall.

l_n = moment of inertia of ne section of wall.

Table.8 – Calculation of centroid of net section of wall

Pier	**A_i m²**	**l m**	**A_il m²**
5	0.25	0.5	0.125
6	0.50	4.0	2.0
7	0.25	7.5	1.875
	Σ = 1.0		**Σ = 4.0**

Distance from left edge to centroid = 4.0 / 1.0 =4.0 m

Table.9 – Calculation of moment of inertia of net section of wall

Pier	**A_i m²**	**l_i m**	**A_i l_i m³**	**A_i l_i^2 m⁴**	**I = td³/12**	**I_n=I+ A_i l_i^2**
5	0.25	3.50	0.875	3.06	0.02	3.08
6	0.50	0	0	0	0.04	0.04
7	0.25	3.50	0.875	3.06	0.02	3.08
	Σ = 1.0					**Σ=6.20 m⁴**

Increase in axial load in individual piers of south shear wall is determined as in table.

Table 10 – Increase in axial load in individual piers of south shear wall

Pier	**Ai li m³**	**P_{ovt}= M_{ovt} x (A_i l_i / I_n) kN**
5	0.875	134.08
6	0	0
7	0.875	134.08

M_{ovt} = 950.12 kN.m, ln = 6.20 m^4

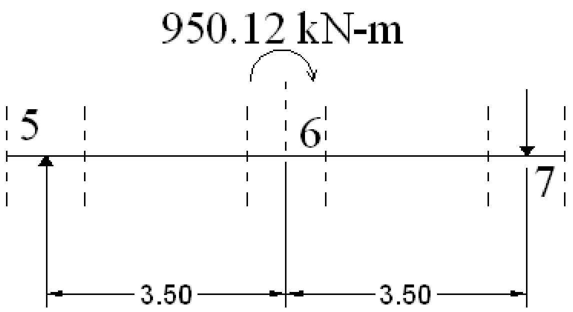


Fig – 5 : Over turning moment in pier of south wall

STEP: 5 DETERMINATIONS OF PIER LOADS, MOMENTS AND SHEAR

The total axial load (due to dead load, live load and overturning), shear and moment in the individual piers of both the shear walls are calculated in tables

Table 11 – Axial load, moment, shear in piers of North shear wall

North Wall: First Storey

Pier	Effective width of pier	Pd$_1$ (kN)	PL^2 (kN)	P$_{ovt}$(kN)	Shear V$_E$ for moment	Moment(kNm) = V$_E$ x h/2
1	1.75	135.62	26.25	105.58	56.70	56.70 x ½ =28.35
2	2.5	193.75	37.50	32.75	56.70	56.70 x ½ =28.35
3	2.25	174.37	33.75	40.09	56.70	56.70 x ½ =28.35
4	1.5	116.25	22.50	98.36	15.96	15.96 x 2.5/2 =28.35

Table 12 – Axial load, moment, shear in piers of South shear wall

South Wall: First Storey

Pier	Effective width of pier	Pd$_1$ (kN)	PL$_2$(kN)	P$_{ovt}$(kN)	Shear V$_E$ for moment	Moment(kNm) = V$_E$ x h/2
5	2	155	30	134.08	40.30	40.30 x ½ =20.15
6	4	310	60	0	98.53	98.53x ½ =49.27
7	2	155	30	134.08	40.30	40.30x ½ =20.15

I. Pd = effective loading width of pier x dead load intensity in kNm

Effective loading width of pier = width of pier + ½ of each adjacent opening of pier dead load intensity is calculated as (per meter length of wall)

North Wall : First Storey

Weight of first storey(from level of IInd to sill level)
= 2.5 x 0.25 x 20 = 12.5 kNm
Weight of second storey= 4 x 0.25 x 20 = 20 kNm
Weight of floor at IInd storey level(Assume North and South shear wall will equal amount of load)
= ½ (0.12 x 15 x 25) = 22.5 kNm
Weight of roof = ½ (0.12 x 15 x 25) = 22.5 kNm
Total Load = 77.5 kNm

South Wall: First Storey

Weight of first Storey (from level of IIndto sill level)
= 2.5 x 0.25 x 20 = 12.5 kNm
Weight of second Storey = 4 x 0.25 x 20 = 20 kNm
Weight of floor at IInd storey level(Assume North and South shear wall will equal amount of load)
= ½ (0.12 x 15 x 25) = 22.5 kNm
Weight of roof = ½ (0.12 x 15 x 25) = 22.5 kNm
Total Load = 77.5 kNm

II. PL = effective loading width of pier x live load intensity in kN/m

Effective loading width of pier = width of pier + ½ of each adjacent opening pier live load intensity (per meter length of wall) calculated as

North Wall: First Storey

Live load on floor (1 kN/m^2)(Assume North and South shear wall take equal amount of load) = ½ (1 x 15) = 7.5 kN/m

Live load on roof (1 kN/m^2)(Assume North and South shear wall take equal amount of load) = ½ (1 x 15) = 7.5 kN/m
Total Load = 15 kNm

South Wall: First Storey

Live load on floor (1 kN/m^2)(Assume North and South shear wall take equal amount of load) = ½ (1 x 15) = 7.5 kN/m

Live load on roof (1 kN/m^2)(Assume North and South shear wall take equal amount of load) = ½ (1 x 15) = 7.5 kN/m
Total Load = 15 kNm

STEP: 6 DESIGN OF SHEAR WALLS FOR AXIAL LOADS AND MOMENTS

North Shear Wall

Table 13 – Determination on jamb steel at the pier boundary

Pier	Moment (kNm)	Effective depth (mm)	Area of jamb steel AS*(mm2)	No of bars	P(kN)
1	28.35	900	152.17	2 @ 10Φ	267.45
2	28.35	900	152.17	2 @ 10Φ	264
3	28.35	900	152.17	2 @ 10Φ	248.21
4	19.95	900	107.08	2 @ 10Φ	237.11

Table 14 – **Check for Adequacy for piers

Pier	P(kN)	d(m)	t(m)	F_a/F_a	F_b/F_b	F_a/F_a+ f_b/F_b	
1	267.45	1	0.25	0.427	0.217	0.644	OK
2	264	1	0.25	0.422	0.217	0.639	OK
3	248.21	1	0.25	0.397	0.217	0.614	OK
4	237.11	1	0.25	0.379	0.153	0.532	OK

South Shear Wall

Table 15 – Determination on jamb steel at the pier boundary

Pier	Moment(kNm)	Effective depth (mm)	Area of jamb steel AS*(mm2)	No of bars	P(kN)
5	20.15	900	108.15	2 @ 10Φ	309.08
6	49.27	1800	132.23	2 @ 10Φ	370.00
7	20.15	900	108.15	2 @ 10Φ	319.08

Table 16 – **Check for Adequacy for piers

Pier	P(kN)	d(m)	t(m)	Fa/Fa	Fb/Fb	Fa/Fa+ fb/Fb	
5	309.08	1	0.25	0.494	0.154	0.648	OK
6	370.00	2	0.25	0.296	0.094	0.390	OK
7	319.08	1	0.25	0.456	0.154	0.610	OK

* Jamb steel at the pier boundary is given by,

AS = M / (fs x 0.9 x deffective)

fs = 0.55 Fe = 0.55 x 415 = 230 N/mm2

$d_{effective}$ = d_{total} – Cover

** Adequacy of individual piers under compression and moment is checked by interaction formula i.e.

(fa/Fa) + (fb/Fb) $\leq$ 1.33

fa = [Ptotal i.e.(Pd+PL+Povt)] / [width of pier (d) x t]

fb = $M/(td^2/6)$

Fa = Permissible compressive stress = 2.5 N/mm^2(as per IS:1905)

Fb = Permissible bending stress = 2.5 + 0.25 x 2.5

= 3.125 N/mm^2 (as per ISs: 1905)

STEP 7: DESIGN OF SHEAR WALLS FOR SHEAR

Shear in building may be resisted by providing the bands or bond beams. The bands represent a horizontal framing system, which transfer the horizontal shear induced by the earthquakes from the floors to shear (Structural) walls.

Design of Bond Beam:

Total seismic shear in E – W direction = 333kN

Moment produced (M) = V x L/8 = 333 x 15/8

= 624.37 kNm

T = M/d = 624.37/8 = 78.04kN

A = T/fs = 78.04 x 1000/230= 339.33 mm^2

Use 2 @ 16 Φ (= 402 ssmm2)

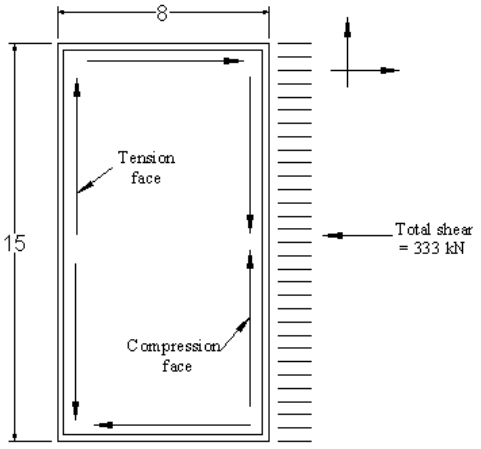


Fig – 6 : Design of Bond Beam

5. STRUCTURAL MODELLING AND DESIGN OF A TWO STOREYED MASONRY BUILDING

In this design, analysis of the two stoyered masonry building is done. Based on the analysis, all the Surfaces are designed for strength and serviceability. STAAD-PRO is used for the analysis.

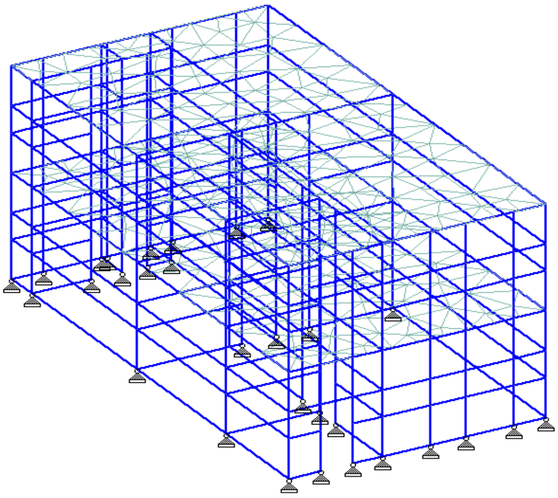


Fig – 7 : Masonry Building 3D view

5.1 Load Combinations

Loads on any structure may be arranged in design so that the maximum force or moment is achieved at the point in the structure being considered. Hence all load combinations must be considered. Following are the different load combinations used in this design of masonry building.

1. 1.5 DL+LL

   1.5 Dead Load + Live Load

2. 1.2DL+1.2LL+1.2ELX

   1.2Dead Load + 1.2 Live Load + 1.2 Earthquake load at X direction

3. 1.2DL+1.2LL-1.2ELX

   1.2Dead Load + 1.2 Live Load - 1.2 Earthquake load at X direction

4. 1.2DL+1.2LL+1.2ELZ

   1.2Dead Load + 1.2 Live Load + 1.2 Earthquake load at Z direction

5. 1.2DL+1.2LL-1.2ELZ

   1.2Dead Load + 1.2 Live Load - 1.2 Earthquake load at Z direction

6. 1.5DL + 1.5 ELX

   1.5Dead Load + 1.5 Earthquake load at X direction

7. 1.5DL- 1.5 ELX

   1.5Dead Load - 1.5 Earthquake load at X direction

8. 1.5DL+1.5 ELZ

   1.5Dead Load + 1.5 Earthquake load at Z direction

9. 1.5DL - 1.5 ELZ

   1.5Dead Load - 1.5 Earthquake load at Z direction

10. 0.9DL + 1.5 ELX

    0.9 Dead Load + 1.5 Earthquake load at X direction

11. 0.9 DL – 1.5 ELX

    0.9 Dead Load – 1.5 Earthquake load at X direction

12. 0.9 DL + 1.5 ELZ

    0.9 Dead Load + 1.5 Earthquake load at Z direction

13. 0.9DL – 1.5 ELZ

    0.9 Dead Load – 1.5 Earthquake load at Z direction

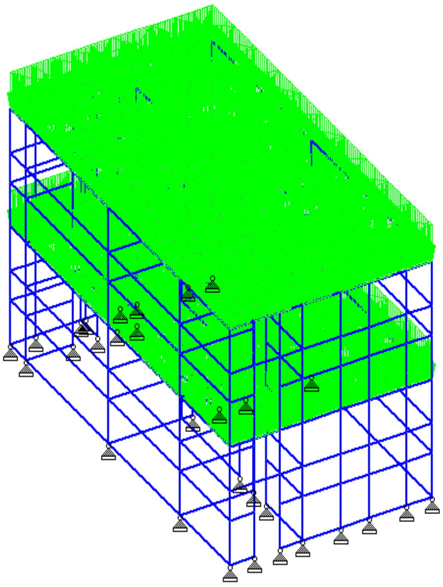


Fig – 8 : DL and LL acting on slab

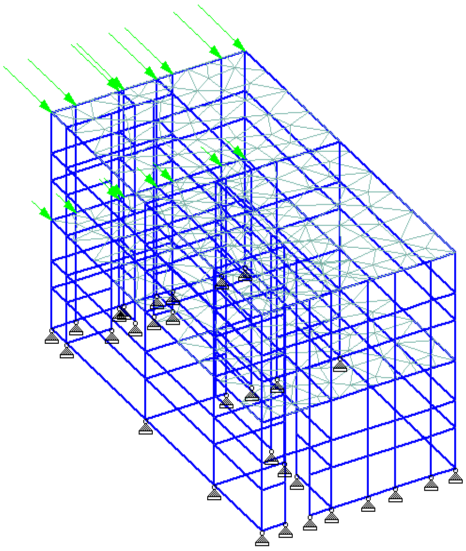


Fig – 10 : Seismic motion acting on the building in Z direction

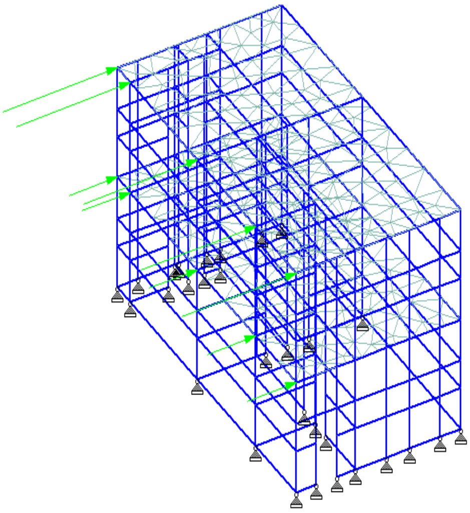


Fig – 9 : Seismic motion acting on the building in X direction

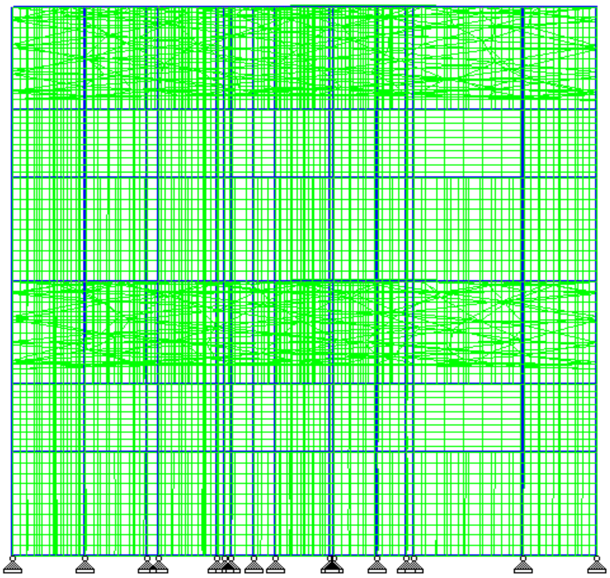


Fig – 11 : Displacement occurred due to load combinations

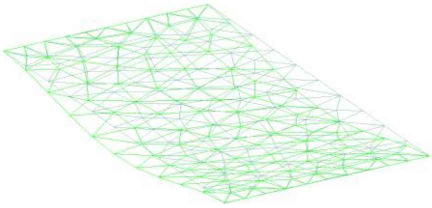
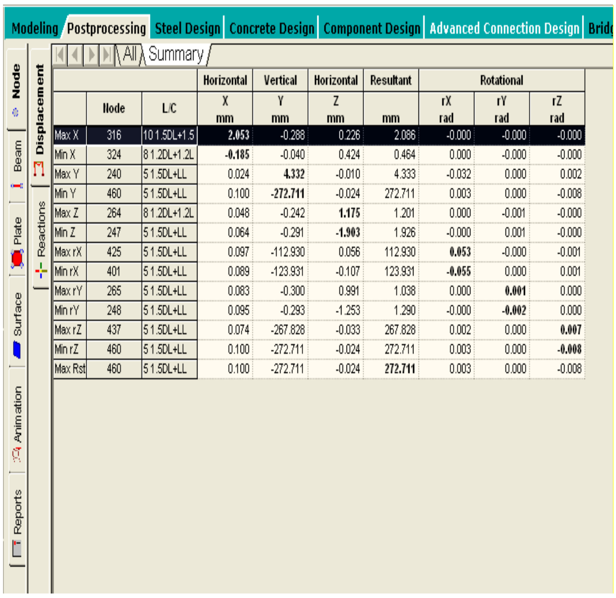


Fig – 12 : Displacement occurred on the slab



	Node	L/C	Horizontal X mm	Vertical Y mm	Horizontal Z mm	Resultant mm	Rotational rX rad	 rY rad	 rZ rad
Max X	316	10 1.5DL+1.5	**2.053**	-0.288	0.226	2.086	-0.000	-0.000	-0.000
Min X	324	8 1.2DL+1.2L	**-0.185**	-0.040	0.424	0.464	0.000	-0.000	-0.000
Max Y	240	5 1.5DL+LL	0.024	**4.332**	-0.010	4.333	-0.032	0.000	0.002
Min Y	460	5 1.5DL+LL	0.100	**-272.711**	-0.024	272.711	0.003	0.000	-0.008
Max Z	264	8 1.2DL+1.2L	0.048	-0.242	**1.175**	1.201	0.000	-0.001	-0.000
Min Z	247	5 1.5DL+LL	0.064	-0.291	**-1.903**	1.926	-0.000	0.001	-0.000
Max rX	425	5 1.5DL+LL	0.097	-112.930	0.056	112.930	**0.053**	-0.000	-0.001
Min rX	401	5 1.5DL+LL	0.089	-123.931	-0.107	123.931	**-0.055**	0.000	0.001
Max rY	265	5 1.5DL+LL	0.083	-0.300	0.991	1.038	0.000	**0.001**	0.000
Min rY	248	5 1.5DL+LL	0.095	-0.293	-1.253	1.290	-0.000	**-0.002**	0.000
Max rZ	437	5 1.5DL+LL	0.074	-267.828	-0.033	267.828	0.002	0.000	**0.007**
Min rZ	460	5 1.5DL+LL	0.100	-272.711	-0.024	272.711	0.003	0.000	**-0.008**
Max Rst	460	5 1.5DL+LL	0.100	-272.711	-0.024	**272.711**	0.003	0.000	-0.008

Fig – 13 : Displacement Deatils

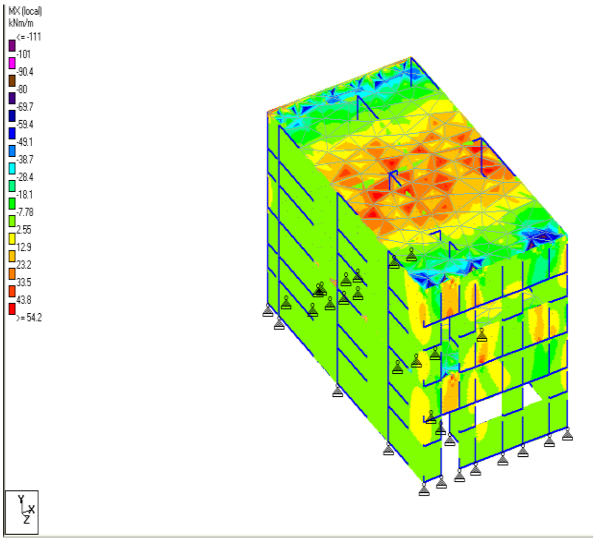


Fig – 14 : Effect of Maximum Moment at X direction

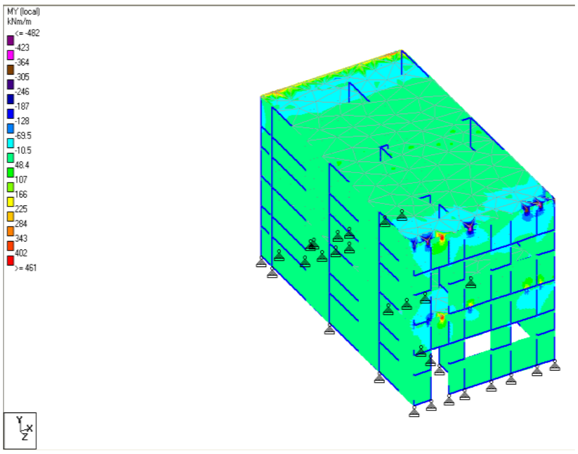


Fig – 15: Effect of Maximum Moment at Y direction

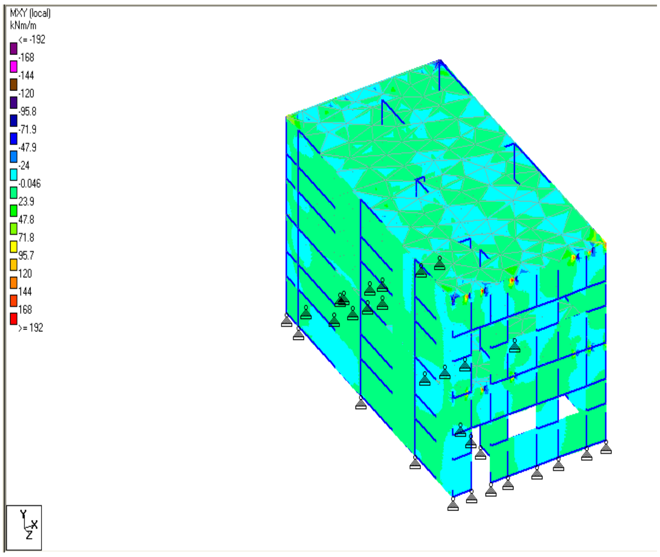


Fig – 16: Effect of Maximum Moment at XY direction

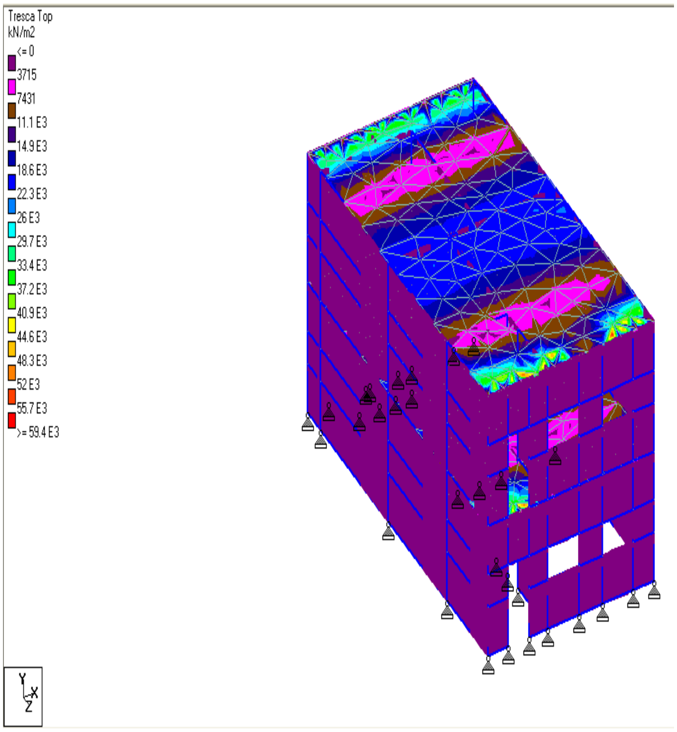


Fig – 17: Effect of Maximum Stress at Top

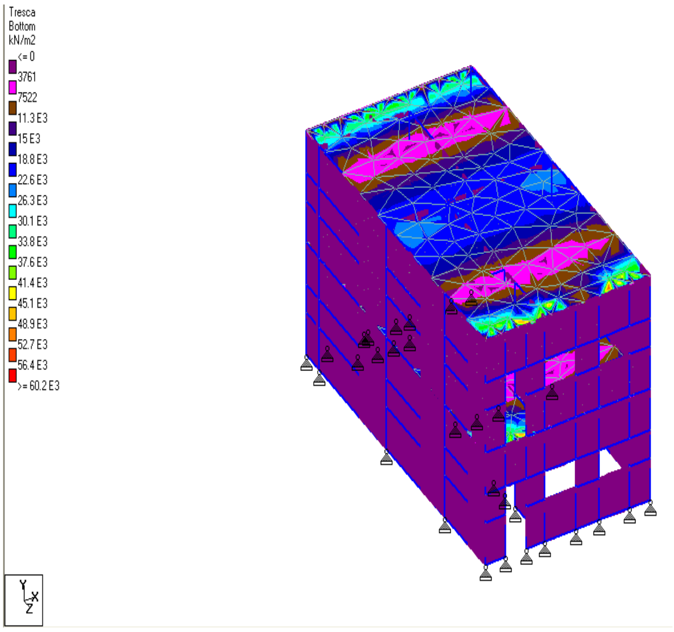


Fig – 18: Effect of Maximum Stress at bottom

5.2 DESIGN OF SHEAR WALLS FOR AXIAL LOADAND MOMENTS

North Shear Wall

Table 17- Determination of jamb steel at the pier boundary

Pier	Moment (kNm)	Effective depth (mm)	Area of jamb steel AS*(mm2)	No of bars	P(kN)
1	13	900	69.77	2 @ 10Φ	97.47
2	8.6	900	46.16	2 @ 10Φ	6.28
3	59.78	900	320.88	2 @ 10Φ	107.13
4	7.93	900	42.57	2 @ 10Φ	60.2

Table 18- ** Check for Adequacy of piers

Pier	P(kN)	d(m)	t(m)	fa/Fa	fb/Fb	fa/Fa+ fb/Fb	
1	97.47	1	0.25	0.389	0.312	0.255	OK
2	6.28	1	0.25	0.025	0.206	0.075	OK
3	107.13	1	0.25	0.428	1.43	0.374	OK
4	60.2	1	0.25	0.2408	1.14	0.461	OK

South Shear Wall

Table 19 – Determination of jamb steel at the pier boundary

Pier	Moment (kNm)	Effective depth (mm)	Area of jamb steel AS*(mm2)	No of bars	P(kN)
5	6.42	900	34.46	2 @ 10Φ	7.25
6	21.05	1800	56.49	2 @ 10Φ	18.09
7	15.34	900	82.34	2 @ 10Φ	5.79

Table 20 – **Check for adequacy of piers

Pier	P(kN)	d(m)	t(m)	fa/Fa	fb/Fb	fa/Fa+ fb/Fb	
5	7.25	1	0.25	0.029	0.154	0.061	OK
6	18.09	2	0.25	0.036	0.126	0.054	OK
7	5.79	1	0.25	0.023	0.368	0.126	OK

* Jamb steel at the pier boundary is given by,

AS = M / (fs x 0.9 x deffective)

fs = 0.55 Fe = 0.55 x 415 = 230 N/mm2

$d_{effective} = d_{total} - \text{Cover}$

** Adequacy of individual piers under compression and moment is checked by interaction formula i.e.

$(f_a/F_a) + (f_b/F_b) \leq 1.33$

$f_a = [P_{total}\ \text{i.e.}(P_d+P_L+P_{ovt})] / [\text{width of pier (d) x t}]$

$f_b = M/(td^2/6)$

F_a = Permissible compressive stress = 2.5 N/mm^2(as per IS:1905)
F_b = Permissible bending stress = 2.5 + 0.25 x 2.5

= 3.125 N/mm^2 (as per ISs: 1905)

CONCLUSION

Masonry buildings are the most common type of construction used for all housing around the world. But the post-earthquake survey has proved that the masonry buildings are most vulnerable to and have suffered maximum damages in the past earthquakes. A survey of the affected areas in past earthquakes demonstrated that the major losses of lives were due to collapse of low-strength masonry buildings. Due to the brittleness of the masonry material, lack of ductility, strength and locally used traditional material in a traditional manner without the earthquake-resistant features are the main causes of collapse of building during earthquake.

The present work is a step towards with regard to illustrate a procedure for seismic analysis and design of masonry building. The procedure has been presented by considering each clause as mentioned in IS 1905 and IS 4326:1993 with the help of an example of a three-storeyed residential masonry building.

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