International Journal Of Engineering Research And Development
e- ISSN: 2278-067X, p-ISSN: 2278-800X, www.ijerd.com
Volume 16, Issue 11 (November 2020), PP. 09-24

Seismic Analysis and Design of Masonry Structure

Abin Jose[1], Dinesh Rawat[2], Javid Ahmad Bhat[3],Bimalesh Sah[4]and Dinesh Raj Bhattrai[5]

[1,2,3,4]*M.Tech. (Structural Engineering),Department of Civil Engineering,Amity School of Engineering and Technology, Amity University, Uttar Pradesh, India.*
[5]*Artista Engineering Consultancy Pvt. Ltd., Kathmandu, 44600, Nepal*
Corresponding author: Javid Ahmad Bhat

ABSTRACT: *Past world–wide earthquake experience shows that the un-reinforced masonry structure is more vulnerable than reinforced structure. Unreinforced masonry buildings are usually characterized by sudden and dramatic collapse. This review paper is mainly deals with the seismic analysis and design of masonry structure. To carry out the study at first, architectural drawings are prepared considering different functional, geometrical and engineering aspects. The building is located in Himalayan region of Nepal which is in high seismic zone i.e. V. The structure which is analyzed and designed is single storied dressed stone masonry building with metal roof structure and CGI roof sheets.Then the building modeled in FEM-based software (SAP2000, version 18.1.1) for detailed structural analysis including all material properties, loads and its combination. The analysis results like direct stresses, bending stresses, shear stresses, tension, etc. are determined and checked against the limiting value of the material as per codal provision. The drawings are suitably adjusted to bring the building to safe level as per codal requirements.*

KEYWORDS: *Seismic Analysis, Compressive Stress of Masonry, Seismic Analysis, Stresses, Bending Stresses, Shear stresses*

Abbreviations: FEM, Finite Element Method;m, Meter; mm, Millimeter; 3D-ThreeDimensional; 2D, Two Dimensional; NBC Code:IS, Indian Standard; kN, Kilo Newton; EQX, Effect of Earthquake in XDirection; EQY, Effect of Earthquake in Y Direction.

Date of Submission: 18-11-2020 Date of Acceptance: 03-12-2020

I. INTRODUCTION

After April-25, 2015 Gorkha earthquake, number of type designs were developed by various organizations focusing earthquake affected districts. It would be better to construct similar structures for educational infrastructure throughout the country by using advanced materials but due to various limitations like transportation, topography, terrain, climate, population etc, there is need for some conventional type structures using maximum locally available and less imported materials specially for Himalayan region. Hence, this type design for educational structure is prepared for suitable sites of Himalayan region.

1.1. Objectives

i. To Model and analysis one storey masonry structure building.
ii. To know the seismic performance of masonry building under seismic loading.
iii. To obtain induced element stresses and check with code provision

1.2. Scope of the study

This report deals with the appropriate analysis and design for typical school building. It includes the structural analysis and necessary design calculations for the building to stand safe at its intended use and also during design earthquakes as per the prevalent codal provisions.

II. LITERATURE REVIEW

M.shariq, S.Haseed and M.Arif (2017) performed the analysis of existing heritage building which is subjected to earthquake load. For study they choose the existing heritage building situated in Aligarh City and performed the time history analyzed by using the EI-Centro earthquake data, they observed the maximum tensile and maximum shear stress after that they compared this value with permissible stress. The finding obtained was in new location such has dome –wall junction, wall roof-junction and minarets exceed the permissible limits.

Shakeel Ahmad ,Rehan Ahmad Khan ,Hina Gupta (2014) did the seismic performance of masonry heritage structure which was 137 years old masonry heritage school which and located in seismic zone IV.Seismic analysis of unreinforced masonry school was carried out by using the commercially available Finite Element STAAD .Pro software assuming a homogenous and nonlinear behavior of the material.In this study they obtained the first 5 natural frequencies from model analysis which they founded first 4 fundamental frequencies are closely spaced but 5th frequency is widely spaced. Finally they concluded that recommendations regarding the strengthening of various location of wall of structure can only made after the numerical model of the structure under the given condition.

ShyamSundarKhadka (2017) took old traditional ShitalNivas building as a case study for evaluating its seismic efficiency. The construction modeling and analysis is carried out by the computer program SAP2000. Seismic coefficient approach is used for structure analysis. Owing to the difficulty of modeling and the interpretation of the results, the entire building is not taken for the experiment. The findings obtained from the 3-D model North wing are generalized for the entire building's global behavior.

III. METHODOLOGY

At first, architectural drawings are prepared considering different functional, geometrical and engineering aspects. Then the building modeled in FEM-based software (SAP2000, version 18.1.1) for detailed structural analysis including all material properties, loads and its combination. The analysis results like direct stresses, bending stresses, shear stresses, tension, etc. are determined and checked against the limiting value of the material as per codal provision. The drawings are suitably adjusted to bring the building to safe level as per codal requirements.

3.1. About the Structure

The structure which is analyzed and designed is single storied dressed stone masonry building with metal roof structure and CGI roof sheets. The structure has been strengthened with different vertical and horizontal RC bands including steel posts at veranda.

3.2. Analysis Procedure

The structure has been modelled and analyzed with computer software “SAP2000 ver.18.1.1”. The software has very good analysis and design capability which are verified in the verification problems included in the package. It is a Finite Element Method (FEM) based software and requires modelling of the structure by finite-elements. The walls were modelled with area-element of appropriate property and the building is assumed to be supported at plinth level for the analysis of super-structure. The analysis stresses are used to verify the safety of the provided structural members, including walls. Also, these designs are provided to include the minimum requirements for earthquake resistant measures as per relevant Nepal National Building Codes as well.

3.3. Detailed Parameters of the Building

The proposed plan included one typical school building with two class rooms. The detail of the building is presented on the following sections.

Details of Model is shown in Table 1.

Table 1: Details of Model

Architectural Features	
Type of Building	School Building
Location	Typical design for Himalayan region
Number of storey	One-storeyFloor
Height (Typical)	2.8 m (eaves); 4.7m (roof ridge)
Structural Features:	
Foundation Type	Continuous stone wall foundation
Walls	450 mm
Type of walls	Dressed stone walls
Structural System	Load bearing system
Roof	Metal Truss with CGI roof
Geotechnical Features: as per NBC 105:1994	
Type of soil	Soft soil
Seismic zone Factor (Z)	1.0
Importance factor I	1.5 (for School Building)
Str. Performance factor (K)	2.5 (for masonry with added ductile bands, interpolated for ductile bands

	with reference to IS1893:2002, on a safer side)
Fundamental Time period	0.1 sec
Basic Seismic Coeff (C)	0.09
Hor. Seismic Coefficient (Cd)	C.Z.I.K = 0.3375
Note: Thus, horizontal seismic coefficient of 33.75% is used for calculation of earthquake load and applied to the structure with linear vertical distribution as per NBC 105:1994.	
Material Properties:	
Cement	Ordinary Portland cement (OPC)
Stone	Locally available, dressed
Mortar	Cement-Sand mortar (1:6)

3.4. Loading

For the analysis of the building, all the loadings (dead loads and live loads) are calculated based on different parts of IS 875:1987 (NBC has been used wherever applicable). Earthquake load is calculated based on NBC105:1994.

Dead Load (DL) - These are the permanent load which is not supposed to change during the structure's design life. The unit weight of materials are as follows:

Stone Masonry: 26 KN/m^3

Plaster: 20.5 KN/m^3

Steel: 76.97 KN/m^3(7849 Kg/m^3)

Live Load (LL) - These are the loads that may vary its intensity and/or position during design life. Live loads for roofs are calculated as per the functional requirement as specified in IS875 code. As this is one storey building,

Live loads on inaccessible roof: $0.75kN/m^2$

Earthquake Loads (EL) - Earthquake load has been calculated based on NBC 105. Basically, horizontal seismic forces shall be considered for the structures that depend on different parameters.

3.5. Load Combination

Different load combinations are generated as per NBC105:1994. Wind loads are calculated and used only for the roof truss analysis and design. Total nine load combinations are used for stress analysis of the structure as follows:

i. DL+LL
ii. DL + LL +- EL (total 4-combinatinons for +ve and –ve EL in x & y direction)
iii. 0.7DL +- EL (total 4-combinatinons for +ve and –ve EL in x & y direction)

3.6. Modelling

The structure is modelled in SAP2000 version 18.1.1. Shell elements are used to model walls. Roof loads are manually calculated and applied to corresponding walls and posts as applicable. Major RC bands and steel sections are modelled with frame elements. Different data for modelling are presented in the following sections.

Plan of the Building - The floor plan of the building is shown in Figure 1. Walls are constructed with 18" (450mm) dressed stones in cement mortar. Roofing of CGI sheets are provided with metal trusses.

Three Dimensional Views - The three-dimensional view of FEM model of the structure is as shown in Figure 2. The major structural member is load-bearing wall, supported by different seismic enhancing elements like RC bands, steel posts, etc. For the masonry wall modelling, a three-dimensional four-node shell element having 24 DOFs with 6 DOFs at each node were used while line/frame element having 12 DOFs with 6 DOFs at each node were used.

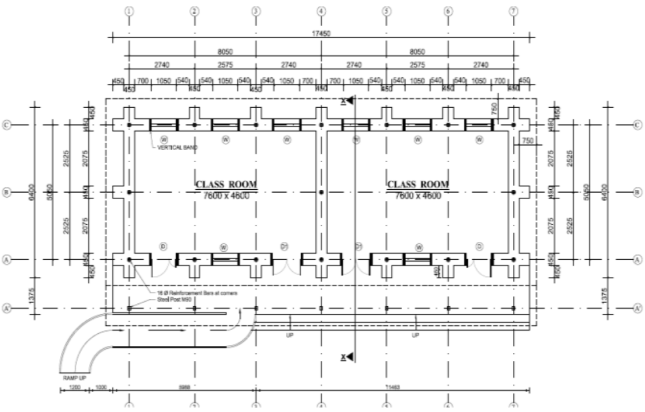


Fig 1: Sectional Plan of Base Model (Regular Frame)

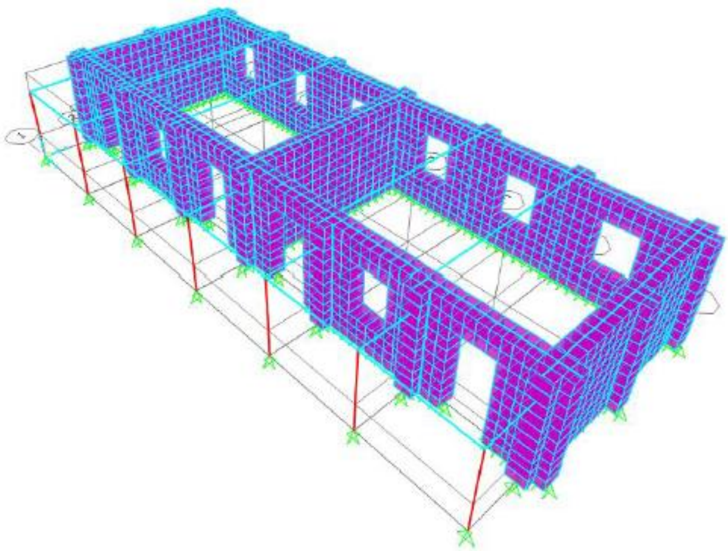


Fig 2: 2D view of 3D model of the building

3.7. Properties and Sections of Structural Elements

The main structural element is stone load bearing walls. The thickness of wall is 450mm as shown in plan. The basic compressive strength of masonry with average rectangular (dressed) stones (with crushing strength not less than 15Mpa) in 1:6 mortar (M2) is taken as 1.03 MPa as per IS1905:1987.

Table 2: Properties and Sections of Structural Elements

Properties of Masonry		
Description	**Value**	**Remark**
Stone Strength	15 Mpa	Safe value based on observation
Mortar	1:6 Cement:Sand	M2 Type
Basic compressivestress of masonry	1.03 MPa	As per IS1905:1987
Compressive Strength (fm)	4.12 Mpa	4 times basic compressive strength
Elasticity of wall (E=550*fm)	2266 Mpa	
Sections of Structural Elements		
Material	**Thickness**	**Remarks**
Stone Wall	450mm	Dressed stone Masonry
Steel Post	101.6mm	ISNB90M
Seismic Enhancing Elements		
Material	**Size (mm)**	**Remarks**
Plinth Band	150x450	RC
Stitch Band	75x450	RC
Sill/Lintel	100x450	RC
Roof Band	150x450	RC
Vertical Band	75x450	RC
Corner and at buttress	16mm diameter	Fe415 rebar

Reinforcement		
Roof Bracing	16mm bars	Diagonal steel ties

IV. RESULTS AND DISCUSSION

4.1. Base Reaction

The total horizontal forces applied on the building on earthquake loading is tabulated in Table 3. The horizontal forces on each X and Y directions are same as these are calculated based on seismic coefficient method for low rise building having very low time-periods.

Table 3. Base Reaction

Output Case	Case Type	Global FX (kN)	Global FY (kN)
EQX	Linear Static	-552.85	0
EQY	Linear Static	0	-552.85
Seismic Load	0	0	1638.11

Here, earthquake load has been applied in accordance with earthquake load calculation. These horizontal forces are distributed to each structural member based on their individual seismic masses and experience lateral forces in addition to vertical dead and live forces.

4.2. Drift in the Building

Any functional building shall have limited deformation on design earthquake in addition to sufficient strength against failure. This is important to maintain the serviceability of building even after the earthquake. The drift ratio of upto 0.4% is allowed for the building structures. The maximum displacement on each floors and the corresponding drifts are shown in table 4 and figure below. Drift are well within limits for all cases.

Table 4.Storey Displacement and Drift

Storey	Elevation	X- Dir	Y-Dir	Drift X	Drift Y	Remarks
Storey 1	2.8	1.0	2.31	0.035%	0.082%	Within Limit
Base	0	0	0	0	0	x

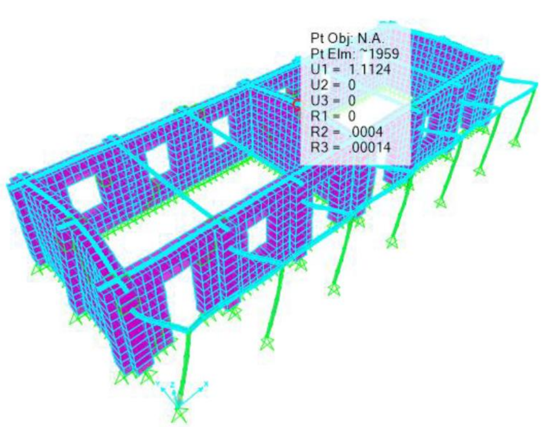


Fig. 3. Deformed Shape in EQX

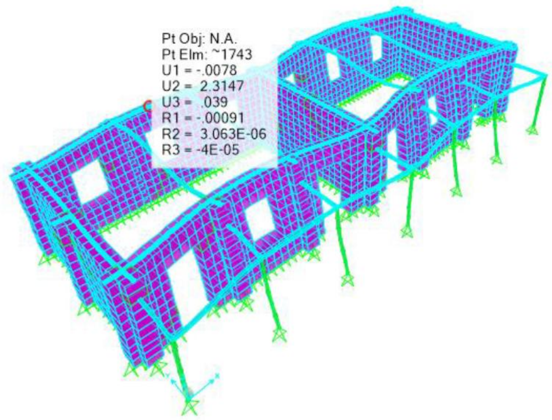


Fig. 4. Deformed Shape in EQY

4.3. Stress Analysis and Design of Elements

The structural system of this building is load bearing. Dressed stone masonry are used as main wall system which are strengthened with horizontal RC bands at plinth, sill, lintel and eaves level. Additionally, horizontal RC stitch bands at corners and junctions, RC vertical bands on the line of jamb of openings and vertical reinforcement with steel tube at corners and junctions of walls are provided that improves the ductility and strength of the masonry.

The stress levels at different critical locations of the masonry walls and stresses on bands are calculated to determine their sufficiency. The forces and stresses as determined from the FEM analysis is presented in the following sections.

Member strength- The capacity of each elements of the masonry system, like walls bands and steel sections are determined to check against the actual stresses coming on the member. This way, they can be checked against their adequacy and specifications. Bands and rebars are specified at critical locations which are given in detail drawings.Capacity calculation of splint and bands areshown in Table 5.

Table 5: Calculation of strength of splints/bands.

With fy415-10mm bars on band					
Bandage Options	B1	B2	B3	B4	Units
Width of Band	450	450	450	450	Mm
Band thickness	75	100	150	150	Mm
Grade of wire	415	415	415	415	N/mm^2
Dia. Of wire	10	12	10	12	mm
No, of wires	2	2	4	4	nos
Cover	30	30	30	30	mm
Spacing of wire	380	378	380	378	mm
Total area of wires	157.08	226.19	314.16	452.39	mm^2
Allowable Tensile strength of wires	230	230	230	230	N/ mm^2
Allowable compressive strength of wires	190	190	190	190	N/ mm^2
Allowable comp. stress in concrete	7	7	7	7	N/ mm^2
Increase in allowable stress in EQ combination	0.25	0.25	0.25	0.25	-
Total allowable tensile force	45.16	65.03	90.32	130.06	kN
Total allowable compressive force	332.62	447.47	665.24	698.07	kN
Bending in 450mm wall					
Wall Thickness	450	450	450	450	mm
Band thickness	75	100	150	150	mm
Overall depth	450	450	450	450	mm
Effective Depth	420	420	420	420	mm
Considering maximum stress in bar equals to allowable stress in bar,					
Lever arm	390	390	390	390	mm
Moment of Resistance	17.61	25.36	35.23	50.72	kN-m
Shear					
Allowable shear stress in masonry	0.1	0.1	0.1	0.1	N/mm^2
Maximum horizontal force (Shear) resisted by band	51.24	73.13	102.47	142.21	kN

Even though, the primary function of masonry elements is to sustain vertical gravity load, structural masonry elements are required to withstand combined shear, flexure and compressive stresses under earthquake or wind load combinations consisting of lateral as well as vertical loads. In these studies, the shear stress, tensile stress and compression stress for working stress load combination for earthquake loading are checked with their respective permissible stress.

Even masonry structures are commonly practiced in Nepal, there are lack of experimental mechanical properties of masonry and guidelines and codes for masonry structures. For this study the permissible strength for masonry are calculated with reference to IS1905:1897.

Compressive Stress of Masonry

Since the stone masonry are strong in compression strength, the analysis were conducted for in-plane compressive stress due to earthquake loading and compressive stress due to one of critical loading combinations were verified with permissible stress.

Permissible Compressive Stress:

Compressive strength of masonry units = 15 N/mm^2

Mortar type M2 (1:6 Cement-sand). Basic compressive strength of wall (f_b) = 1.03 N/mm^2 (from table 8, IS 1905:1987). Permissible compressive stress (fc) = fb X Ks X Ka X Kp. Slenderness ration (most common) = h/t or l/t = 2800/450 = 6.22. Stress reduction factor (Ks) =0.95 for above slenderness ratio. Area reduction factor (Ka) = 0.7+1.5 A, A being the area of section in m2

Area reduction factor (Ka), takes into consideration smallness of the sectional area of the elements and is applicable when sectional area of the element is less than 0.2 m^2. But minimum area is 0.65m2 for smallest area. i.e. Sectional Area (A) > 0.2 m^2. Thus, Ka = 1.Shape modification ratio (Kp) = 1.0 (for H/W approx. 1 for dressed stone, table 10 IS 1905:1987)Hence,permissible compressive stress in Masonry (fc) =1.03*0.95*1*1 =0.978N/mm^2

Though different walls have different values of slenderness and hence, stress reduction factor, for this report, critical value is selected and used for all walls for the checking. Compressive stress checks due to earthquake load for different walls are given in following sections.

Permissible Tensile Stress:

As a general rule, design of masonry shall be based on the assumption that masonry is not capable of taking any tension. For this case tensile stresses are taken by vertical bands and their sufficiency are checked. Tensile stress checks due to earthquake load for different walls are given below in following sections.

Permissible Shear Stress:

Stone masonry are not much strong in shear strength due to lateral loading. Diagonal cracks develop due to shear forces. Hence shear stress due to in-plane lateral forces (earthquake loading) was verified for one of the critical load combination with permissible stress. Shear Capacity of masonry is taken as: 0.1+Fd/6 (where Fd=Compressive stress due to dead load). Shear stress checks due to earthquake load for different walls are given in following sections.

Element Forces/ Stresses and their design

The forces and stresses in each elements of the building are presented below.

Compressive Stresses:
As illustrated in the minimum stress diagram for different grids of walls, the maximum compressive stress (minimum stress) on wall is 0.402MPa which is far below the compressive stress limit of 0.97MPa of the considered masonry system.

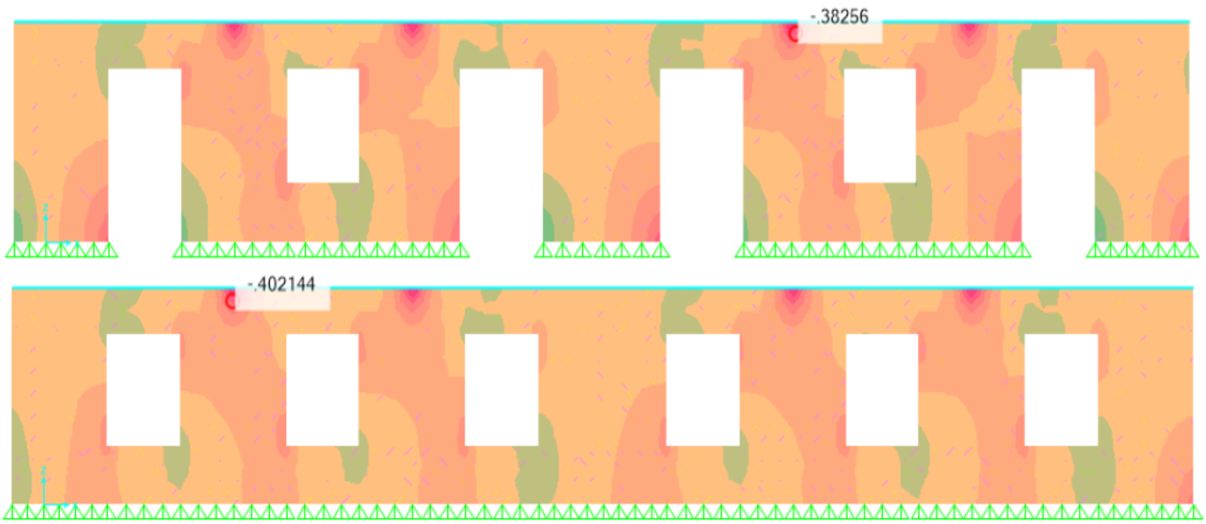


Fig.5. Compressive Stress due to DL+LL+ EQX load combination at Grid 22 and 33

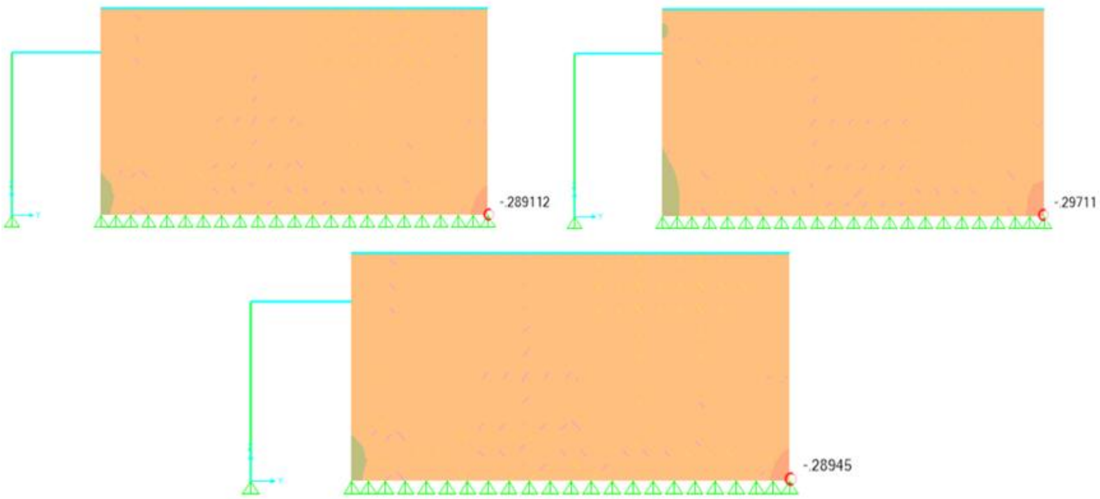


Fig.6. Compressive Stress due to DL+LL+EQY load combination at Grid AA, DD and GG

Tensile Stress:
As illustrated in the maximum tension on the wall is about 62KN/m. Vertical bands are provided at jambs of each opening having tensile strength of at least 67KN each and vertical reinforcement at corner and location of buttresses they will be able to resist all these tensions form the masonry thus limiting the tensile cracks on the masonry.

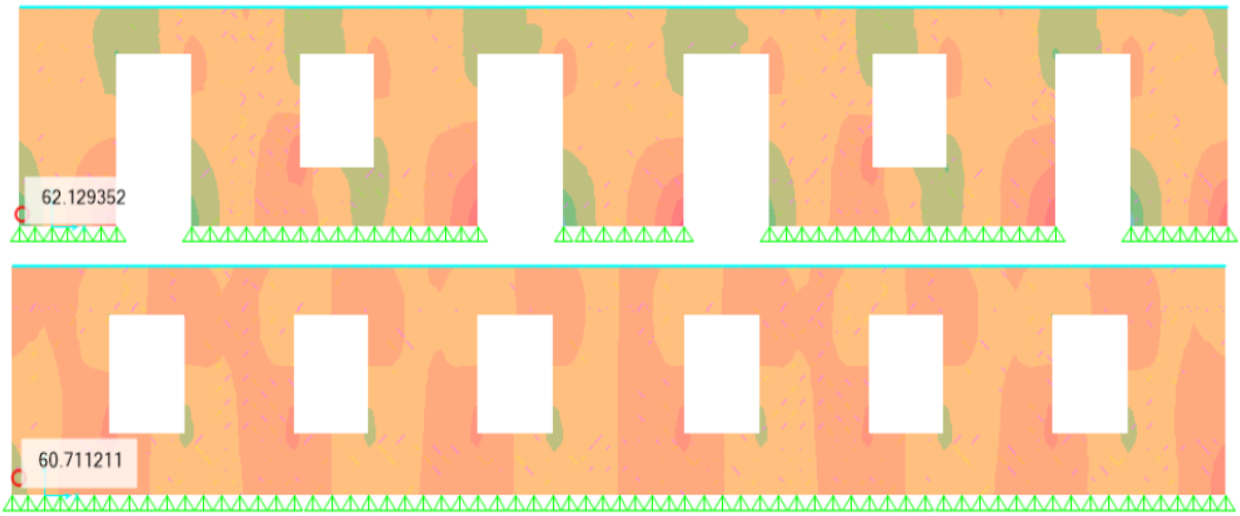


Fig.7. Tensile forces due to EQX (DL+LL+EQX) combination

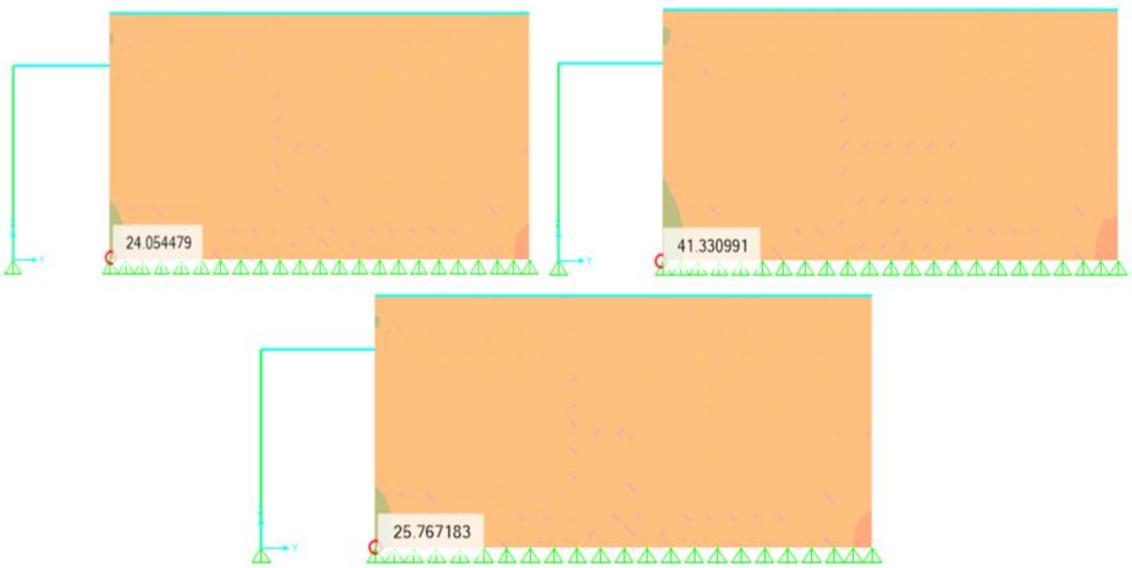


Fig.8. Tensile stresses on cross walls (DL+LL+EQY) combination

Shear Stresses:

Shear stresses on different walls and piers are illustrated in the following figures. These stresses are nearly equal to the least limiting value of 0.1 MPa for masonry in cement mortar in some location where truss rests on the masonry wall due to concentration of load but eve band of 150mm provided at the eves level distribute the load coming from the truss and such load concentration on masonry would not occur. The vertical stress near sill level is 0.08MPa, thus making the allowable shear of (0.1+0.08/6) =0.11 Mpa. Moreover, horizontal and vertical bands are provided that enhances the shear capacity of the wall. Here as the shear stress has not exceeded the limiting value of masonry, the building is safe in shearing as well.

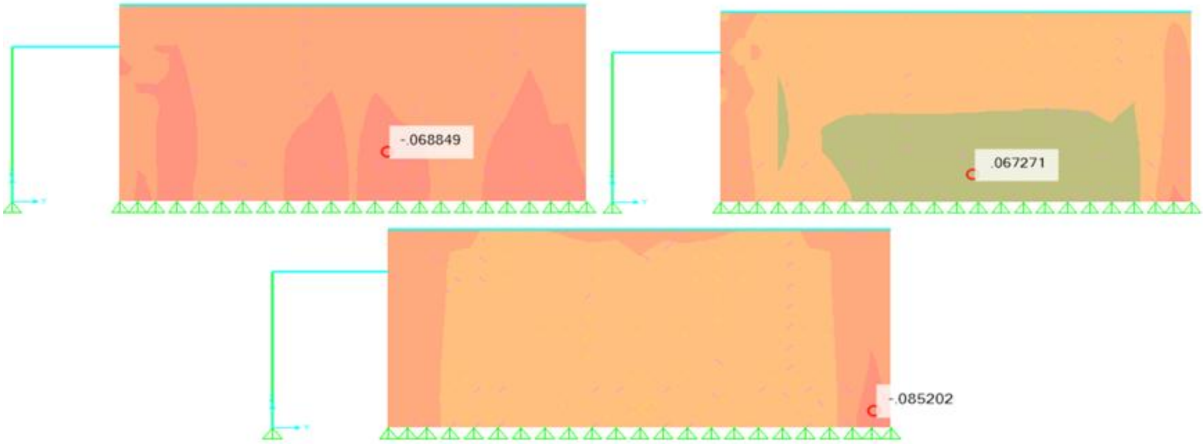


Fig.9. Shear stress on short wall due to EQY (DL+LL+EQY) combination

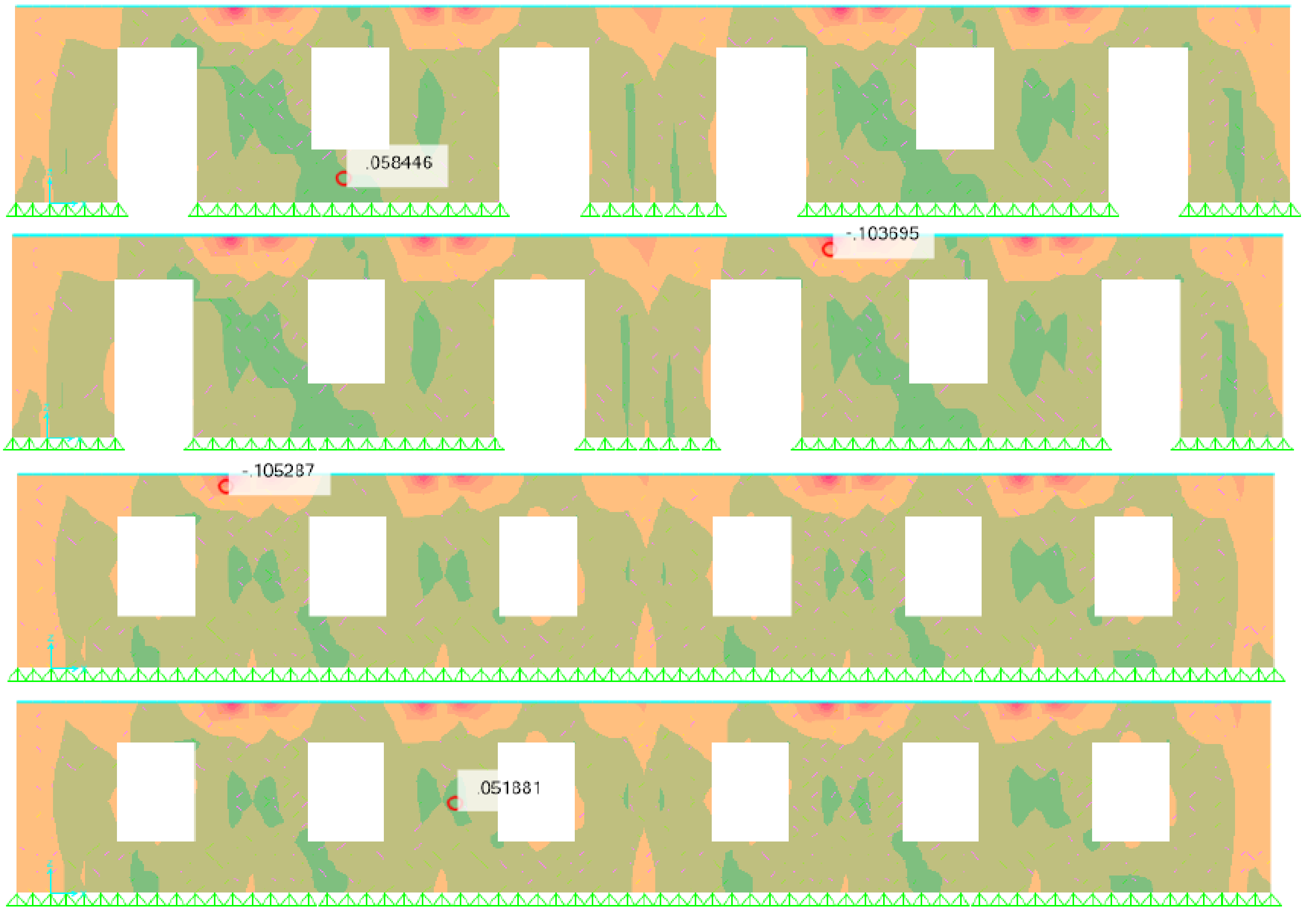


Fig.10. Shear stress on long wall due to EQX (DL+LL+EQX) combination

Horizontal and vertical Bending:

The maximum horizontal bending of wall is 18.49 KN-m/m on the wall above lintel in classrooms. However, apart from roof band, lintel alone has the moment of resistance of 25.36KN-m and will be able to resist these bending in the walls.

Similarly, maximum vertical bending on wall is about 15.88 KN-m /m which is effectively resisted by provided bands on the jambs of the openings and corners and junctions.

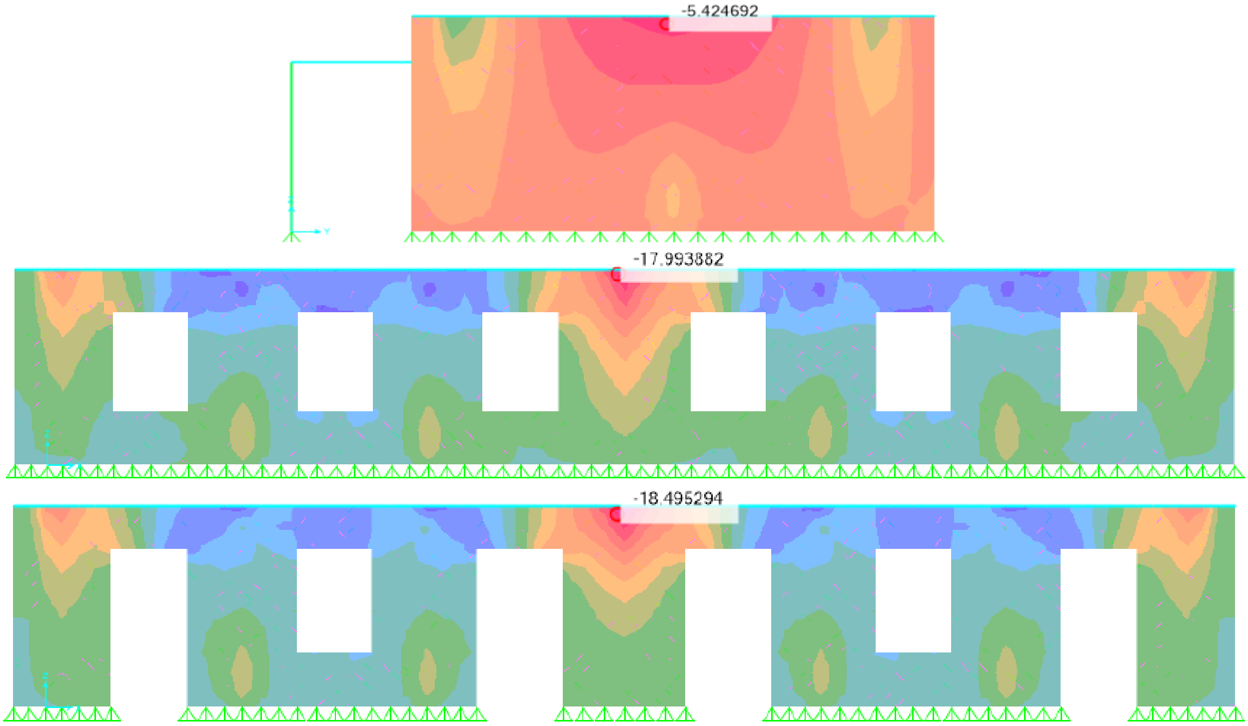


Fig.10. Horizontal bending on long wall

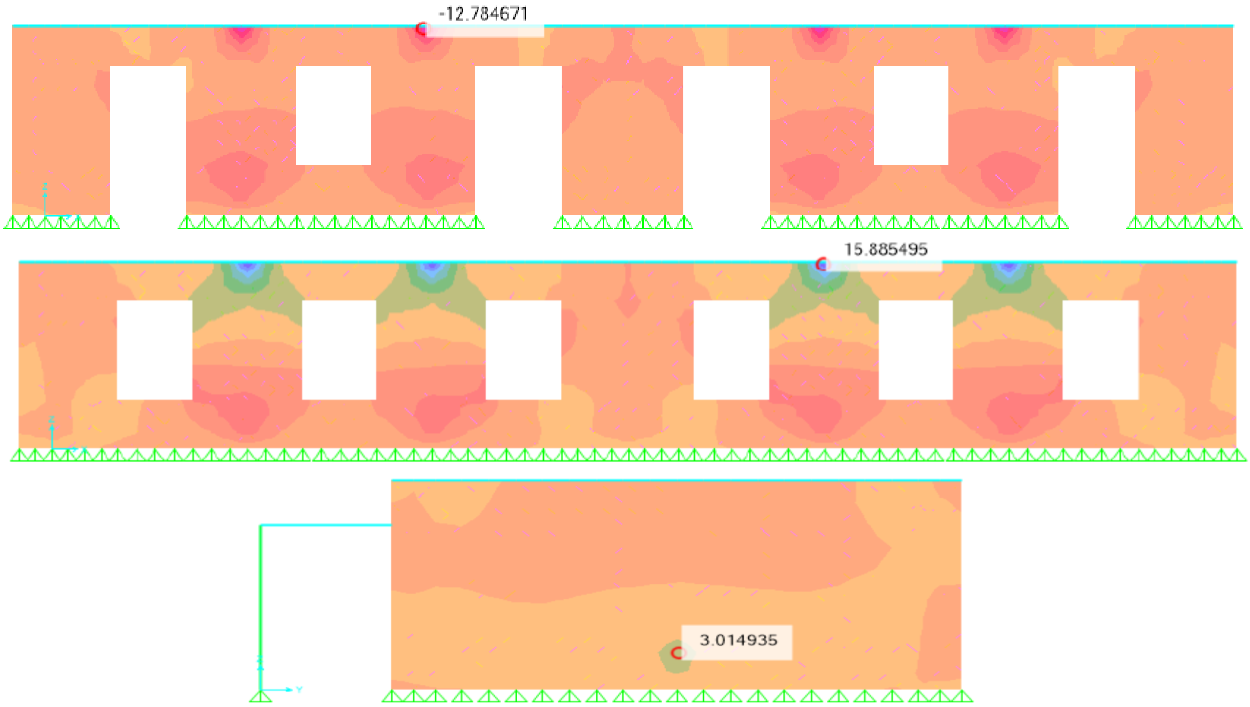


Fig.11. Vertical bending on long wall

Hence, in the current design of the masonry, there are no any excessive stresses on the masonry, and vertical and horizontal bands are sufficient to provide the strength, stiffness and stability to the masonry system for its overall strength.

4.4. Foundation Design

The design of foundation depends on the maximum force on the structure footing during service and during extreme cases of load combination.

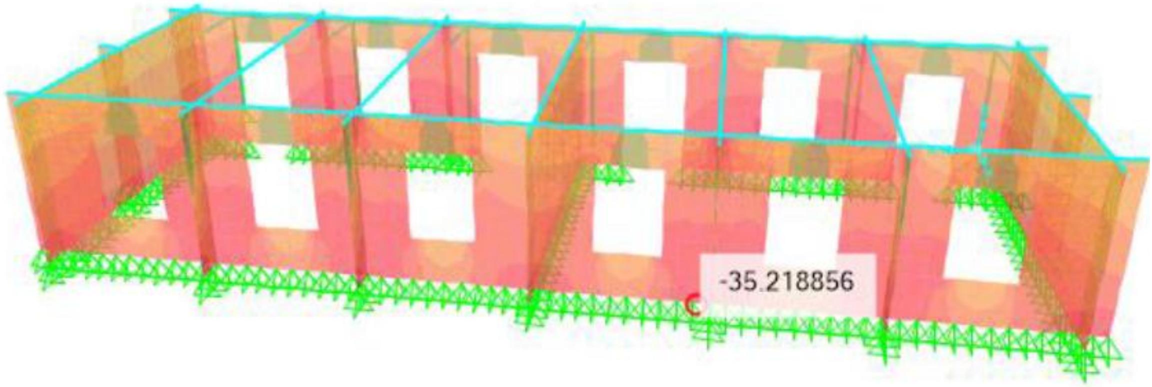


Fig.12. Load per meter at plinth level in the building under service.

The analysis shows the maximum pressure under service condition is about 33 KN/m. Adding the stress of 18 KN/m (for 1.5m footing depth) for plinth, a total of 51 KN/m load as acting from foundation.Thus, footing width is provided based on minimum requirement of NBC of 900mm. Thus the bearing pressure works out to be (51/0.9) 56.67KN/m2 only. Hence, this size is sufficient for lower range of soft soil.

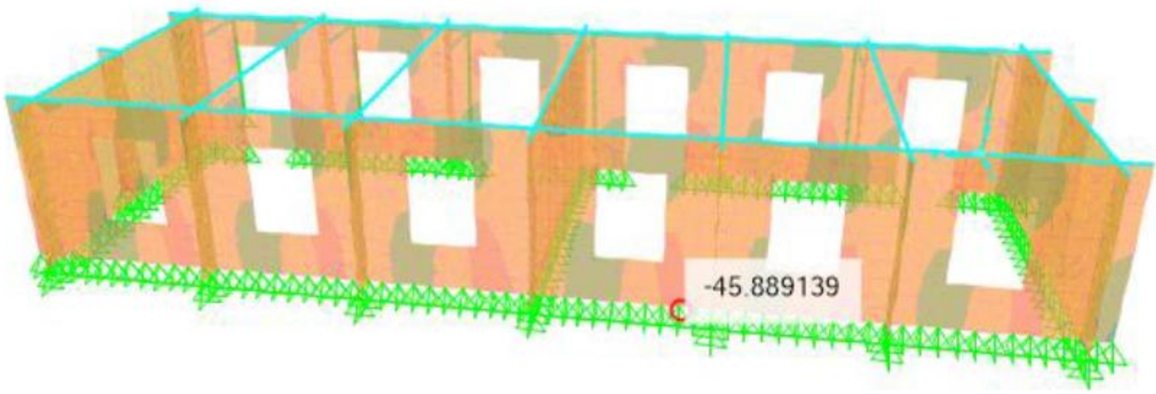


Fig.13. Load per meter with earthquake loading.

Under earthquake, the vertical load increased to 45.89KN/m with a total of 63.89KN/m that gives the bearing pressure of only 70.98KN/m. Hence, the foundation width is sufficient for service load as well as seismic loading condition.

4.5. Design of Roof Truss

The roof truss has been designed with metal truss with CGI roof sheets. The sections are designed using FEM analysis using SAP2000 and accordingly detailing are prepared.

FEM Analysis Model

The FEM model of the 2D truss system was prepared. The sections are optimized using different trial sections to find the optimum design.

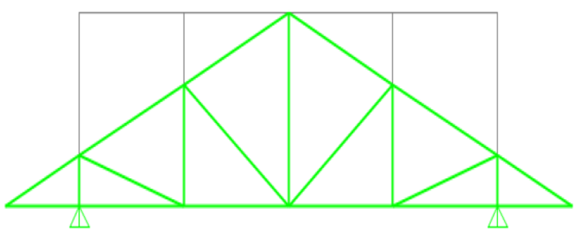


Fig. 14.FEM model of the 2D truss system

Loads in the structure

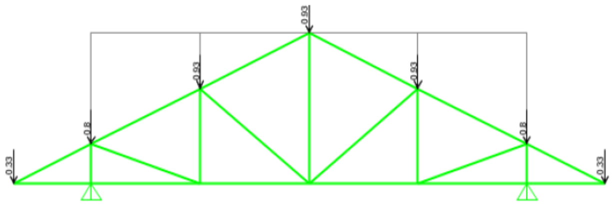
Major load in the roof truss system is the dead load, wind load, live load and their combinations. The wind load calculation is given in Table 6.

Table 6. Wind Load Calculation

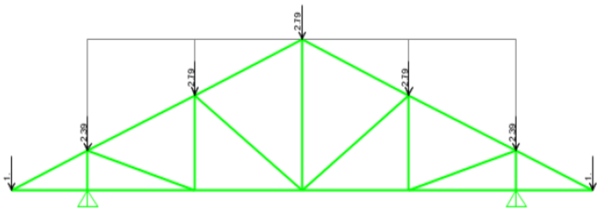
1) Wind load calculation			
Input Data			
A) Wind pressure in Roof	Sign	Input	Unit
Width of Building	W	6.90	m
Length of building	L	18.65	m
Eaves height	H	2.80	m
Pitch height	P	1.78	m
Basic wind speed	Vb	55	m/sec
Probability Factor	K1	1	
Terrain/height/size factor	K2	1	
Topography factor	K3	1	refer IS 875 part3
Design Wind Speed	Vz	55	m/sec
Design wind Pressure	Pd	1815	N/mm^2
Height to width ratio	H/w	0.41	ratio
Pitching angle	Φ	27.29	Degree
Structure type		A	
Terrain category		2	
Structure Class	General building	50.00	
Wall opening	> 20% wall opening		
Plan building ratio	L/W	2.70	
1) For Wind normal to ridge External wind pressure coefficient windward	Cpe	-0.40	(choose appropriate from Table B7 and B9)
For windward slope	Cpe	-0.17	
External wind pressure coefficient Leeward	Cpe	-0.40	
For lee ward slope	Cpe	-0.40	
Internal pressure coefficient For normal permeability	Cpi	0.70 - 0.70	
Combined external and internal wind pressure			
Windward slope		952.97	N/m2
		- 1,588.03	N/m2
leeward slope		544.50	N/m2
		- 1,996.50	N/m2
2) Wind parallel to ridge External wind pressure coefficient	Cpe		
On both slopes for 1/4th length of building	Cpe	- 0.70	
On both slopes for 1/2th length of building	Cpe	- 0.60	
Internal pressure coefficient for normal permeability	Cpi	0.70, -0.70	
Combined External + Internal wind pressure On both slopes for 1/4 th length of building On both slopes for 1/2 th length of building		-2,541.00 181.50 -2,359.50	N/m2 N/m2 N/m2
So the design wind pressure is		952.97	Downward
and		-2,541	Upward
Design wind pressure	**-2541**	**N/m2**	**Upward**
Wind load (Normal to roof)			
joint load (J1)	**-4.73**	**kN**	
joint load (J2)	**-9.45**	**kN**	
joint load (J3)	**-8.10**	**kN**	
joint load (J4)	**-3.38**	**kN**	
wind load (vertical upward)			
joint load (J1)	**-4.27**	**kN**	
joint load (J2)	**-8.53**	**kN**	
joint load (J3)	**-7.31**	**kN**	
joint load (J4)	**-3.05**	**kN**	
wind load (horizontal)			
joint load (J1)	**-2.03**	**kN**	
joint load (J2)	**-4.07**	**kN**	
joint load (J3)	**-3.48**	**kN**	
joint load (J4)	**-1.45**	**kN**	
Live load			
joint load (J1)	**1.39**	**kN**	

joint load (J2)	**2.79**	**kN**	
joint load (J3)	**2.39**	**kN**	
joint load (J4)	**1**	**kN**	
Dead Load			
joint load (J1)	**0.46**	**kN**	
joint load (J2)	**0.93**	**kN**	
joint load (J3)	**0.80**	**kN**	
joint load (J4)	**0.33**	**kN**	

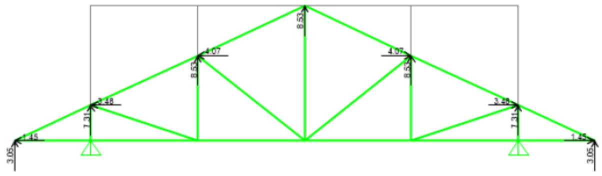
Thus, maximum wind pressure of 2541.0 N/m2 has been found. Maximum live load of 0.75 KN/m2 is considered on the room.
The analysis has been carried out for different load combinations. The axial loads in the truss member for one of the critical combination is shown in figure 15.



Dead Load on Truss



Live Load on Truss



Wind load on Truss

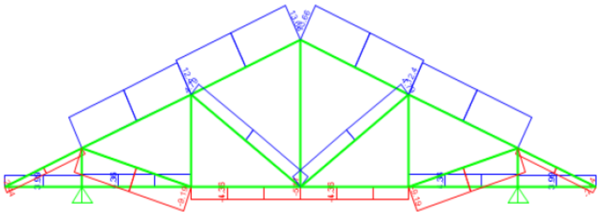


Fig.15. Member forces for Load combination 1.2(DL+LL+WL)

The members are so selected to optimize the design. The final members selected for the truss is as shown in figure16.

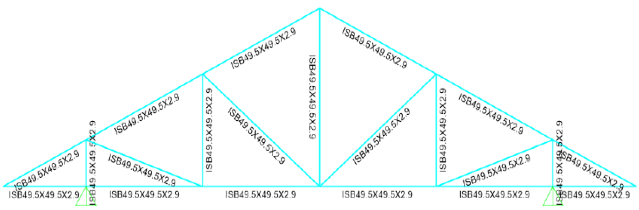


Fig.16. Design member sections

The truss design result shows that, top, bottom and diagonal members are of 49.5x49.5x2.9 mm size.

Design of Purlin:

Taking maximum wind uplift less the dead load of the purlin, the purlin are required to resist the maximum uplift of 2.44KN/m. Purlin of square section is provided to meet the stress and deflection criteria as shown in Table 7.

Table 7.Design of Purlin

Spacing	2.65	M
W max	2.44	kN/m
M max	1.713	kN-m
S per	21.9.45	N/mm^2
Z required	7.81	cm^3
Provided	ISB72x72x3.2	
Z	18.42	cm^3
I	66.32	cm^4
E	2.10E+05	N/mm^2
Stress Criteria:		
Z required	7.81	cm^3
Z provided	18.41	cm^3
Remarks	Pass	
Deflection Criteria:		
Del permissible	10.6	mm
Del actual	11.2	mm
Remarks	Fail	
Next Trial		
Section Provided:	ISB 75x75x3.2	
Z	20.41	cm^3
I	75.53	cm^4
E	2.10E+05	N/mm^2
Stress Criteria:		
Z required	7.80	
Z provided	20.41	
Remarks	Pass	
Deflection Criteria		
Deflection permissible	10.6	mm
Deflection actual	9.9	mm
Remarks:	pass	

Thus, though ISB 72x72x2.9 section pass in stress criteria but fails in deflection criteria, so purlin of ISB75x75x3.2 shall be provided to meet deflection criteria.Detailing requirements - The maximum vertical reaction at hinge support for design of base plate is 42.8 KN.Maximum uplift force for design of anchor is 31.12KN at hinge support. Maximum shear force is 1.68 KN at truss support.

Design of Base plate/Anchor bolt

i. Data Required: Uplift Force = 31.12 kN, Shearing force = 1.68 kN.Punching force = 42.80kN, Concrete Grade = M 20. Factor of safety, Base Plate thickness = 8.00 mm. Permissible shear stress of bolt (σvf) = 80.00 N/mm^2. Permissible bearing stress of bolt (σpf) = 250.00 N/mm^2. Permissible bearing stress of concrete (σc) = 4.00 N/mm^2. Permissible bending stress in steel (σs) = 185.00 N/mm^2. Permissible bond stress in concrete (τbd) = 1.44 N/mm^2. Bolt diameter (Φ) = 16.00 mm.

ii. Design of Bearing Plate - Area of Bearing plate required = 10,700.00 mm^2. Provide bearing plate of size, Length = 200.00 mm. Breadth = 200.00 mm, Total area = 40,000.00 mm^2. Hence, ok. Intensity of pressure between plate and concrete,W = 1.07 N/mm^2 .Thickness of bearing plate: Thickness required for critical moment, $t1 = (6M/\sigma_c)^{1/2}$. Cantilever distance at critical section,x = 37.50 mm. Critical Moment, $M = wx^2/2$ = 752.34 N-mm per unit width. Now, t1 = 4.94 mm, provided thickness of bearing plate, t = 8.00 mm, 2 nos of 8 mm.

iii. Design of Anchor Bolt - Shear Force = 1.68 kN, X-section area of bolt for shear = 16.80 mm^2. Use16.00mm dia bolt, No of bolts required = 0.08 nos.So, provide 4 nos, Length of Anchor bolt, L = 107.48 mm. Provide 250 mm length of each anchor bolt

V. CONCLUSION

A type design for school building has been designed by meeting all the architectural and structural requirements as envisaged by National Nepal Building Code and DOE standards. The building has been structurally analyzed with computer program SAP2000 v 18.1.1 and correspondingly designed. All the tensile, bending, compression and shear stresses on the buildings are well within the permissible limits even under the most adverse combinations of different loads, including Earthquakes as per Nepal Building Code. Similarly, truss is designed for critical combinations of different loads and required detailing are presented. Strict Control over quality of materials and workmanship is required for expected performance of building in future. Hence, following shall be considered during construction works to obtain expected results.

5.1. General:

- All works to be carried out in accordance with current best practice, Building Regulations, the project specification and relevant Nepal Building Code (NBC), Indian Standards and Codes of Practice. Materials and components to be appropriate for their intended use.
- The construction-works shall only be carried-out by trained mason with supervision of Engineer.
- During construction, the contractor shall be responsible for maintaining the structure in a stable condition and ensuring no part shall be damaged under construction activities.
- Workmanship and materials are to be in accordance with the relevant current Standards including all amendments and the local statutory authorities, except where varied by the contract document.
- All coarse aggregate used shall be crushed stone aggregate. The nominal size of coarse-aggregate for RC bands and splints shall not exceed 12.5mm.
- Clean sand, with minimum silt and free from clay and organic materials shall be used.
- Ordinary Portland cement conforming to IS 269:1976 shall be used for all cement works.
- At least 48 hours' notice shall be provided for all engineering inspections.

5.2. Structural:

- Cast-In-Situ concrete/micro-concrete shall have minimum 28 days compressive cube strength of 20N/mm² for all structural members unless otherwise stated.
- The concrete compressive strength shall be measured on 150*150*150mm cube at 28 days, for various structural elements.
- Reinforcing steel shall be TOR having minimum yield strength of 415N/mm². However TMT rebar with ultimate strain not less than 14.5% can also be used.
- Cover to main reinforcing steel be in accordance with IS 456:1978 & as specified in the structural drawings.
- Clear Cover of Concrete shall not be less than that given below:

a. Concrete surface at soil = 50mm
b. Concrete on PCC, Bricks, STONE, etc = 25mm

- Unless otherwise specified, all horizontal & vertical construction joints shall be roughened.
- A minimum of 48 hours' notice shall be given to the Engineer before applying plaster, concrete/micro concrete is poured, in order that the formwork and/or reinforcement may be inspected.
- All R.C.C work shall be continuously cured for 14-days.
- All cement plaster works shall be continuously cured for 7 days.
- Any damage to surface during erection/construction is to be made good.

FUTURE SCOPE

In order to understanding seismic behaviour of masonry structure, this present study may not be sufficient .Therefore, there is a need of further investigation on non-linear analysis like pushover analysis and time history analysis in order to understand the exact behavior or response when subjected to seismic excitation.

ACKNOWLEDGEMENTS

My sincere gratitude goes to our guide Mr. Dinesh RajBhattrai, Artista Engineering Consultancy Pvt. Ltd., Kathmandu, 44600, Nepal, for his constant guidance and encouragement during the course of project work.

Conflict of Interest: No conflict of interest as the study is based on the extensive literature review and expert opinion were taken for the analysis.

REFERENCES

[1]. NBC109:1994 [Load Bearing Masonry]
[2]. NBC105:1994 [Nepal National Building Code: Seismic Design of Buildings in Nepal]
[3]. IS1893:2002 [Criteria for earthquake resistant design of structure]
[4]. IS 875-1987 (Part 1) Code of practice for design loads (other than earthquake) for buildings and structures-Dead Load, Bureau of Indian Standards, New Delhi.
[5]. IS 875-1987 (Part 2) Code of practice for design loads (other than earthquake) for buildings and structures-Live Load, Bureau of Indian Standards, New Delhi.
[6]. Pan, Y., Wang, X., Guo, R., & Yuan, S. (2018). Seismic damage assessment of Nepalese cultural heritage building and seismic retrofit strategies: 25 April 2015 Gorkha (Nepal) earthquake. *Engineering Failure Analysis*,87,80-95.
[7]. Shariq, M., Haseeb, S., &Arif, M. (2017). Analysis of Existing Masonry Heritage Building Subjected to Earthquake Loading. *Procedia Engineering*, *173*,1833-1840. https://doi.org/10.1016/j.proeng.2016.12.229.
[8]. Ahmad, S., Khan, R.A., Gupta, H., (2014). Seismic Performance of a Masonry Heritage Structure, *International Journal of Engineering and Advanced Technology(IJEAT)*, 3(4), pp -335-340.
[9]. Chalise, B., and Suwal, R., (2015). Seismic Performance of Load Bearing Masonry Buildings during Recent GorkhaEarthquake in Nepal. https://www.researchgate.net/publication/2901014424.

Javid Ahmad Bhat, et. al. "Seismic Analysis and Design of Masonry Structure." *International Journal of Engineering Research And Development*, vol. 16(11), 2020, pp 09-24.