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Performance Evaluation of Historical Masonry Structure

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Abstract

Architectural heritage represents an invaluable reflection of the intellectual depth and cultural diversity of contemporary societies, making the structural analysis and safety assessment of historical buildings critically important. Consequently, societies have encouraged the development of specific legislation focused on the protection and preservation of their historical and cultural heritage. Studies have revealed that these structures are vulnerable to powerful outside forces including blasts, high winds, and earthquakes. Many earthquakes strengthening and retrofitting methods for masonry elements and structures have been developed and implemented over the last few decades. The methods utilized to strengthen and adapt Unreinforced Masonry (URM) buildings subjected to seismic and high stress are reviewed and summarized in this study based on prior research. After the performance evaluation of these structures suitable strengthening measures will be explored. These strengthening and retrofitting techniques aim to improve the overall integrity of the masonry structure, decrease the effects of external forces, and increase the load-bearing capability of individual elements.

Keywords**:** Structural health monitoring, Historical masonry, Numerical modelling, Retrofit

1 Introduction

A developing country like Nepal has a huge ratio of masonry structures in comparison to the Reinforced concrete structures. Masonry structures dominate the Reinforced concrete structures in number comprising almost 90% of the total structures in Nepal. Nepal's central, eastern, and western regions were impacted by a powerful 7.8-magnitude earthquake that struck the Barpak neighborhood of the Gorkha district on April 25, 2015. In their most recent contribution, Gautam et al (Gautam, Fabbrocino and de Magistris, 2018) emphasized the fragility of Greco-Roman monuments and classified them as European Macro seismic Scale (EMS-98) vulnerability class B. Most of the Greco-Roman structures in the Kathmandu valley, such as the main administrative center Singha Durbar, Administrative Staff College Jawalakhel, Babar Mahal, and Tangal Durbar, were damaged by the Gorkha earthquake (Pandit *et al.*, 2016). Even though the buildings' construction ages were neither comparable nor similar, the extent of damage to the massive walls was consistently observed.

Although the construction age of such buildings was neither similar nor comparable, damage occurrence was consistent in terms of the extent of damage in the massive walls. Thus, proper study and proper designing of renovation or retrofitting of the masonry structure is very much needed in the context of Nepal. Seto Durbar, which is the building we are currently working on was constructed by the then Rana prime minister, Bir Shamsher JBR in 1893 CE is of huge importance to our country since it still holds the history of our nation. Built in neoclassical and Mughal architecture, this palace is considered as a work of art which most likely need to be preserved and kept intact to pass our heritage to the future generation. The palace, however, has come a long way and has a long history of accidents. The earthquake of 8.0 magnitude which again caused destruction in different parts of the palace forcing for re-renovation of the structure. The structure had not experienced a major outbreak before the 2015 AD earthquake. It is important not only to preserve the old structures but also construct and preserve them using the traditional methods of construction with the then used construction materials. Thus, the retrofitting of this building has been stalled due to the high cost of the operation and since heritage sites need more care during retrofitting such that their originality remains in them. The main objective of the project is to monitor the present health of the building and to find out the current situation that the building stands in. Finding out the flaws in the structure which has destabilized it is one of the main reasons for doing this project. For the objective to be fulfilled, initially an architectural map needs to be prepared. The results obtained from investigating the performance of the current masonry are noted and are used to numerically model the structure. The modelling will provide us with different outputs which will then be used to find out the methods of retrofitting that can be done within the structure. However, the objective is not just to find solutions to the problem but solutions that are cost efficient, suitable, sustainable and environment friendly. The method that can fulfill all these criteria will be considered the ideal method for retrofitting.

Figure 1: Seto Durbar, Durbar Marga

The paper titled seismic Strengthening of the Bagh Durbar Heritage Building in Kathmandu Following the Gorkha Earthquake Sequence by Rabindra Adhikari et al. (Adhikari *et al.*, 2019) explores the seismic vulnerability and retrofitting techniques for the Bagh Durbar heritage building in Kathmandu, Nepal. The building is a load-bearing masonry structure with walls of varying thickness, ranging from 650 mm to 900 mm. The original construction used brick masonry in mud mortar, while later additions incorporated brick masonry in cement mortar. Based on the analysis and field observations, they proposed retrofitting measures aimed at improving seismic safety while preserving the building's historical and architectural integrity. The proposed solutions include Strengthening walls with reinforced concrete (RC) jackets, adding vertical and horizontal reinforcements, installing RC bands at different levels to enhance structural continuity, retrofitting flexible floors with RC slabs or bracings. As the minimum basic strength of masonry per the identified brick strength from tests and weakest mortar according to Indian Standard (IS) 1905:1987 is 0.25 MPa, that corresponds to the average compressive strength of 1 MPa, and the same strength was adopted in this study.

Hwang et al., 2022(Hwang, Kim and Yang, 2022) studied in-plane lateral load transfer capacity of unreinforced masonry walls considering presence of openings. This research investigates the impact of openings on the in-plane lateral load transfer capacity of unreinforced masonry (URM) walls. URM walls are commonly found in historical buildings and are susceptible to damage during earthquakes or other lateral loading events. Openings in these walls, such as windows and doors, further weaken their ability to resist lateral forces.

The authors use a combination of experimental testing and numerical modeling to analyze the behavior of URM walls with different opening configurations. They evaluated factors like:

Opening size and location: How the size and position of openings affect the overall strength and deformation of the wall.

Failure modes: The different ways URM walls with openings can fail under lateral loads, such as diagonal cracking or sliding at the base.

Load-displacement behavior: The relationship between the applied lateral load and the resulting displacement of the wall.

Research on the seismic performance of URM buildings has been continuous. According to earlier studies and regulations from various nations, URM walls' shear behavior is classified. Nevertheless, the application of the shear behavior mode and the equation of shear strength of URM walls is limited by variations in building materials, such as masonry units, mortar types, construction procedures, and other factors. Toe crushing failure, diagonal tension failure, sliding shear failure, and rocking failure are the four main shear behavior modes expressed by URM walls. Aspect ratio, vertical axial load, and the compressive strength of the masonry unit all influence these behavior types. The following equations provide strength acceptance requirements based on the behavior mode. The predicted lateral strength of URM walls or pier components is to be based on the expected bed-joint sliding shear strength or the expected rocking strength, as stated in ('FEMA 273', 1997). Furthermore, the diagonal tension stress or toe compressive stress shall limit the lower bound lateral strength of the current URM walls or pier components. URM walls or piers must have a lateral strength that is less than the Q_{CL} values determined by these four equations.

Rocking strength (V_R): Rocking resistance refers to the ability of a structure to withstand forces that cause it to rock back and forth on its base. The structures are designed to resist seismic shaking. During an earthquake, a building found on a shallow foundation may rock back and forth on the soil.('FEMA 273', 1997) recognizes rocking as a significant contributor to a building's ability to resist lateral forces caused by earthquakes.

$$Q_{CL} = V_R = 0.9\alpha\sigma_0 A_n\left(\frac{L}{H}\right) \tag{1}$$

Sliding resistance (V_s): Sliding resistance refers to the force that opposes the tendency of one object to slide over another. This sliding can lead to catastrophic failures. We analyze Sliding resistance to ensure structures can withstand forces trying to push them sideways.

$$Q_{CL} = V_s = v_{me}A_n \tag{2}$$

where, $v_{me} = 0.5(0.75v_{te} + \sigma_o)$

Toe crushing resistance (V_T)

It refers specifically to the ability of a structure, particularly unreinforced masonry (URM) walls, to resist failure at its base due to excessive compressive forces. Material strength, wall geometry, and the load on the wall all influence toe crushing resistance. FEMA discusses how the weight of the building (axial stress) and lateral earthquake forces can cause deformations and potential crushing at the base (toe) of URM walls.

$$Q_{CL} = V_T = \alpha\sigma_o A_n\left(\frac{L}{H}\right)\left(1 - \frac{\sigma_o}{0.7f_m{}'}\right) \tag{3}$$

Diagonal shear cracking resistance (V_D)

It refers to the ability of a URM wall to resist cracking or failure due to combined shear stress and axial stress acting diagonally. Diagonal cracks are a telltale sign of failure due to diagonal shear stresses exceeding the wall's V_D capacity.

$$Q_{CL} = V_D = f_{dt}A_n\left(\frac{L}{H}\right)\sqrt{1 + \frac{\sigma_o}{f_{dt}}} \tag{4}$$

Bhattacharya rt al., 2014 (Bhattacharya, Nayak and Dutta, 2014) carried out a critical review of retrofitting methods for unreinforced masonry structure. The authors reviewed different retrofitting techniques applied to URM structures regarding the effectiveness of various retrofitting methods for improving structural safety and performance, more particularly under seismic loads. This paper tries to summarize available research and assess the benefits and limitations of several retrofitting approaches.

Retrofitting Methods:

1. External Reinforcement**:** Steel plates and fiber-reinforced polymers (FRP) are applied to its exterior to increase the masonry's rigidity and load-bearing capability.

2. Grouting: Grout is injected into masonry walls during the process to fill in gaps and strengthen the wall's overall structural integrity.
3. Seismic Retrofitting**:** Methods like installing diagonal braces or shear walls are used to improve earthquake resistance.
4. Jacketing**:** This method increases strength and ductility of the existing structure by encasing it in steel or reinforced concrete.

The different retrofitting techniques according to their effectiveness, benefits, and limitations have been assessed by the authors through the thorough analysis of available literature of experimental investigations, theoretical analyses, and case studies.

1. Effectiveness**:** The efficiency of various techniques varies based on the structural and environmental variables. Although they can be costly and complicated, external reinforcement and jacketing are usually effective.
2. Challenges: A few crucial things to think about include potential aesthetic alterations to the building, financial repercussions, and material incompatibilities.
3. Recommendations**:** The report urges more investigation of innovative materials and technologies in addition to the creation of uniform standards for URM structure retrofitting.

The review provides a useful summary of URM building retrofitting techniques and their limitations and practical uses. The study highlights how important it is to carry out further research to advance these methods and address contemporary problems. This provides an overview of the study's findings, emphasizing the need for continued innovation in the field and a critical assessment of retrofitting options for unreinforced masonry.

In the paper by Sharma and Khare 2016 (Sharma and Khare, 2016) entitled Pushover Analysis for Seismic Evaluation of Masonry Wall the authors assessed the applicability of pushover analysis in evaluating the seismic performance of masonry walls. Masonry structures, especially the unreinforced masonry ones, are very vulnerable to seismic activities because they lack ductility and tensile strength. Classic seismic vulnerability evaluation techniques, such as linear static and dynamic analysis, have various limitations for the correct prediction of nonlinear behavior under seismic loads in masonry walls. Pushover analysis is a nonlinear static procedure used to estimate the seismic performance of structures. It involves incrementally applying lateral forces to a building model until a target displacement is reached, representing the expected maximum displacement during an earthquake. The resulting pushover curve plots base shear against roof displacement, providing insights into the structure's capacity and potential failure modes. In the case of masonry walls, the pushover analysis has just recently received wide acceptance as one of the powerful seismic evaluation tools. In the ability to better represent nonlinear behavior for masonry under different cyclic loads, this method shows damage progression and plastic hinge development at its critical stages, very important for understanding the failure mechanisms of this kind of structure. The authors anchor this chapter on the techniques and assumptions made when performing a pushover analysis, which in turn rely on how the material properties, boundary conditions, and load-deformation characteristics of the masonry walls have been represented. A review of various modeling approaches has been carried out, including macro-modeling and micro-modeling. While the application of the pushover analysis to masonry walls has their limitations, the present study underlines that modeling techniques must be improved in terms of accuracy, the interaction between masonry units and mortar joints must be captured, and that pushover analysis needs to be integrated with other seismic evaluation methods. This integrated approach will give rise to the expectations of attaining a more complete assessment of seismic risk and coming up with effective retrofitting strategies for masonry buildings.

According to the paper regarding Seismic retrofitting of the historical masonry structures using numerical approach by Akcay et al., 2016 (Akcay *et al.*, 2016), restoration applications have become necessary in historical buildings due to the deterioration of structural members, including slabs and walls, that complete their life cycle over time because of environmental conditions. This research focuses on assessing old masonry structures in terms of seismic resistance. A case study of historical masonry construction is used to demonstrate the complete procedure. This study involves the restoration of a historical building using laboratory investigations and numerical calculations. In the first phase of the research, samples taken from the building are subjected to mechanical testing and plaster examinations. The building's current 3D computer model was created in the second stage, known as numerical analysis, and materials and members that are insufficiently strong were identified. Analysis and the findings of laboratory and numerical analysis, restoration applications are covered in the third step. Structural flaws in the walls were fixed using the injection process; volta slab (brick floor arches), outer facade walls, interior walls, and door/window gaps were fortified using various ways. The goal of this study was to improve material durability and structural strength in historical structures by combining traditional and modern procedures through laboratory tests and numerical methodologies.

The paper entitled Modelling Methods of Historic Masonry Buildings under Seismic Excitation (Mistler, Butenweg and Meskouris, 2006) critically examines various modeling techniques used to predict the seismic response of historical masonry buildings. The authors review methods including linear elastic models, which are useful for preliminary assessments but may oversimplify structural behavior, and nonlinear static models that provide insights into failure mechanisms through push-over analyses. They also explore nonlinear dynamic models and finite element approaches that offer detailed and accurate simulations of seismic events but require significant computational resources and data. The hybrid approach is also done by combining different modeling techniques such as integrating nonlinear static and dynamic analyses. The modelling of the brittle nonlinear behavior of masonry is carried out on the macro-level. The findings that they have done are accuracy vs. complexity, data requirements and modeling challenges. The paper presents a procedure for assessing the seismic behavior of historical masonry buildings based on measurement of the natural frequencies and numerical simulation. Biaxial failure criterion and a smeared crack approach are used for crack modelling.

In the paper Performance evaluation of strengthening options for institutional brick masonry buildings: A case study of Pulchowk Campus (Motra and Paudel, 2021), the authors evaluated the most effective and economical way to strengthen masonry structures is the primary goal. For the examination, four workable retrofitting options were chosen. Thus, four chosen options for strengthening the restored URM building were combined with a simulation, and the resulting data was examined. While RC columns combined with brick piers via shear keys resulted in a moderate reduction in stress, both sides of RC-jacketing significantly decreased the stress level in masonry piers. Though there is a slight reduction in stress levels with options 4 and 5, there is still stress concentration at the termination junction. To assess the capacities of the four chosen retrofitting options in addition to the URM building as-built, static pushover analyses were conducted at the end. The improvement in masonry wall performance following the implementation of various retrofit options was assessed. The fundamental period of vibration is moderately reduced (24–29% compared to URM) when strengthening options are used, suggesting that stiffness enhancement has an impact on the structural performance. The most cost-effective retrofitting method for masonry structures was discovered to be RC-jacketing (which increased capacity by 80% and reduced roof displacement by 48%); however, the facade's aesthetic could not be maintained. Shear-keyed brick piers and RC columns worked well together because they reduced roof displacement by 26% and increased capacity by 57% when compared to URM buildings. To evaluate the local and global behavior and performance of the structure, a thorough structural condition assessment should be carried out prior to choosing a suitable retrofitting option that is compatible with the type of existing structure. Although the restoration of masonry walls is a simple process, it is recommended that it be combined with the creation of box-action between adjacent walls and elements to maximize the global capacity of the structure. Avoid placing heavy point loads on the masonry piers as they are susceptible. Although it is possible and should be done, there should be appropriate policies in place before beginning any incremental retrofitting. Past post-earthquake observations revealed that, despite RC-jacketing being the most effective strengthening technique for masonry buildings, it may delaminate due to corrosion issues with cracking and detaching concrete cover, as mentioned in.(Meda *et al.*, 2016). High performance fiber RC jacketing was used by (Meda *et al.*, 2016) to guarantee that the structure would last long enough. Enough concrete cover was supplied to prevent corrosion when the reinforcing technique of RC jacketing in conjunction with masonry pier was introduced. To prevent corrosion and problems with delamination caused by cracking and separation of the concrete cover, a thorough investigation is advised.

2 Methodology of study

Seto Durbar, currently operating as Rastriya Banijay Bank, Durbar Marg Branch is the selected site. The architectural plan of the building has been prepared as shown in Figure 6 and Figure 7 A rebound hammer test has been performed on different wall faces. For material characterization, due to unavailability of testing instruments and apparatus, past literature has been reviewed and the paper titled. Seismic Strengthening of the

Bagh Durbar Heritage Building in Kathmandu Following the Gorkha Earthquake Sequence (Adhikari *et al.*, 2019) has been referenced. It is because the Bagh Durbar which is the masonry building analyzed in the paper resembles Seto Durbar. Three walls of dimension (LxH) Wall 1 (3000x3000) mm, Wall 2 (5000x3000) mm and Wall 3 (5000x3000) mm and thickness 1000mm as shown in Figure 3, Figure 4 and Figure 5 respectively have been modelled in SAP2000. The base of the wall was fixed in all directions. A suitable mesh size, for future analysis, has been selected after verifying the software analysis with manual calculations (section 4). Analysis of the whole building with the adopted mesh size will be done and the need of retrofitting will rely on the fact that the failure mode stresses (section 3.1.) will exceed the actual stresses developed on the wall.

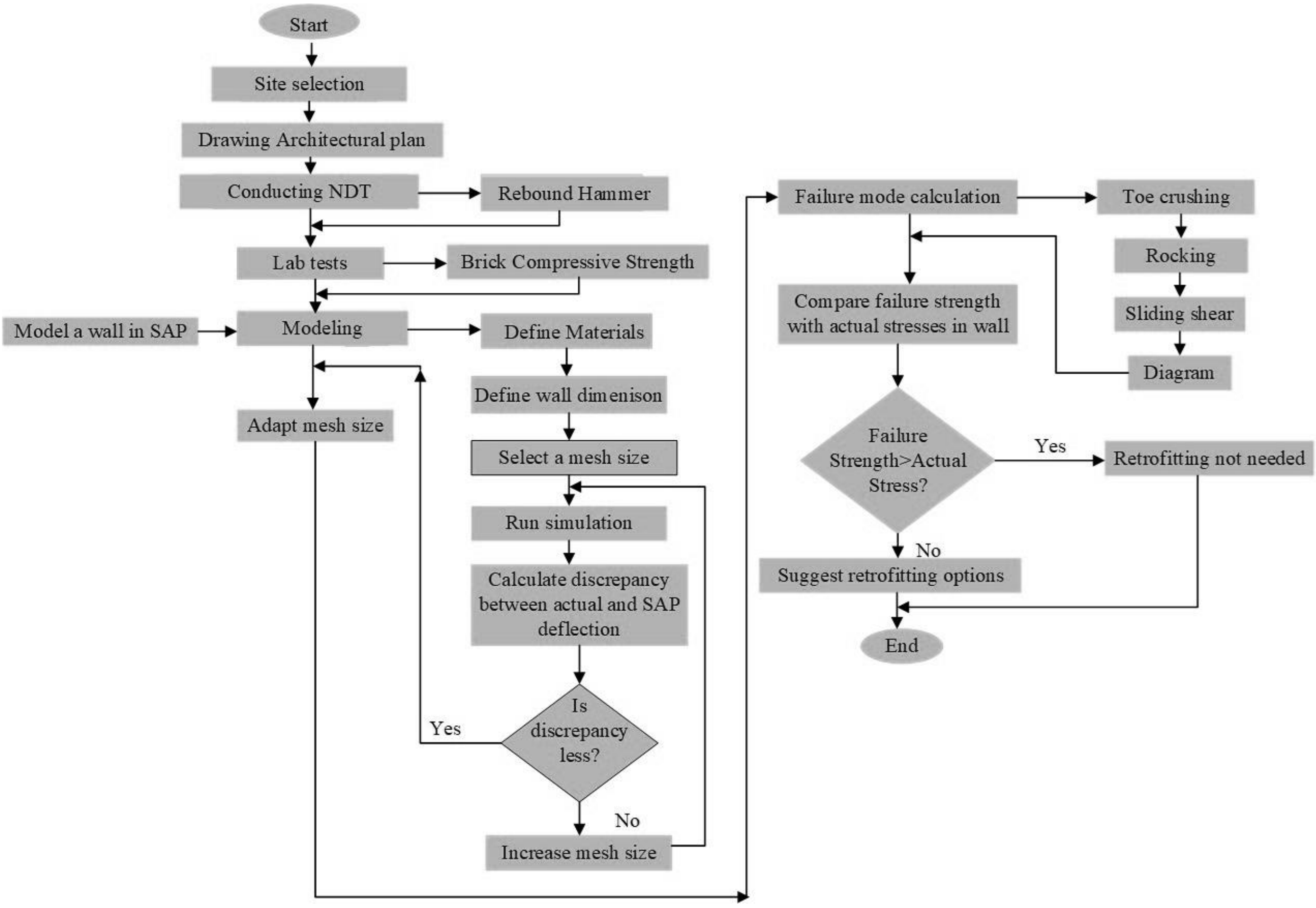

Figure 2. Methodology Flowchart

Various Material properties extracted from (Adhikari *et al.*, 2019) are presented in the table

Table 1. Material Characterization

Parameter	Test Result	Remarks
Shear strength of masonry wall	0.1 Mpa, 0.1 Mpa and 0.08 Mpa for ground floor (GF), first floor (FF) and second floor (SF)	Two tests were carried out on each floor and the average value was taken. Due to lesser dead load the value was low at the top floor while because of the slight damp conditions the values did not increase on the ground floor.
Compressive strength of Brick	6.63MPa, 1.14Mpa and 5.3Mpa for three units	There was restriction in obtaining more samples, thus 3.5Mpa bricks were considered to match the categorization.
Compressive strength of mortar	Varied from 0.1 Mpa to 0.32 Mpa in 11 locations	Though not accurate a penetrometer test was conducted that gave an average strength of 0.18 Mpa.

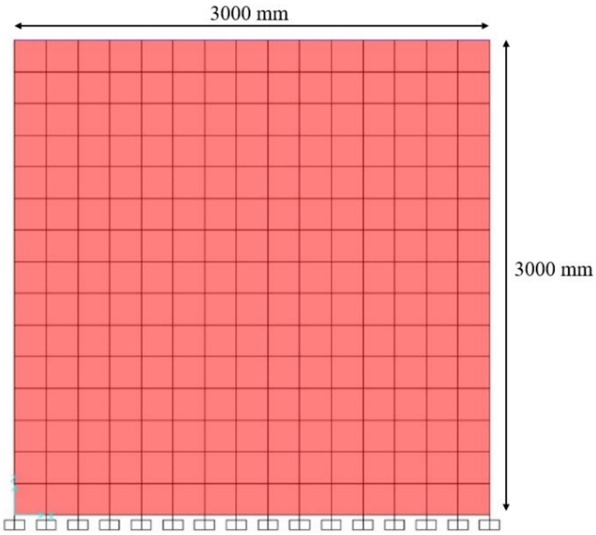

Figure 3. Wall 1 (3000x3000) mm.

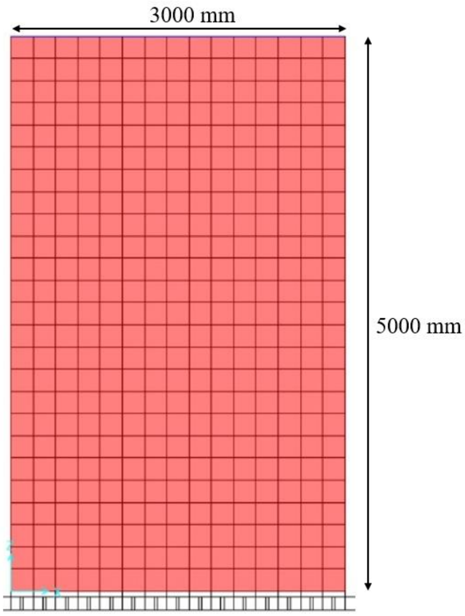

Figure 4. Wall 2 (3000x5000) mm.

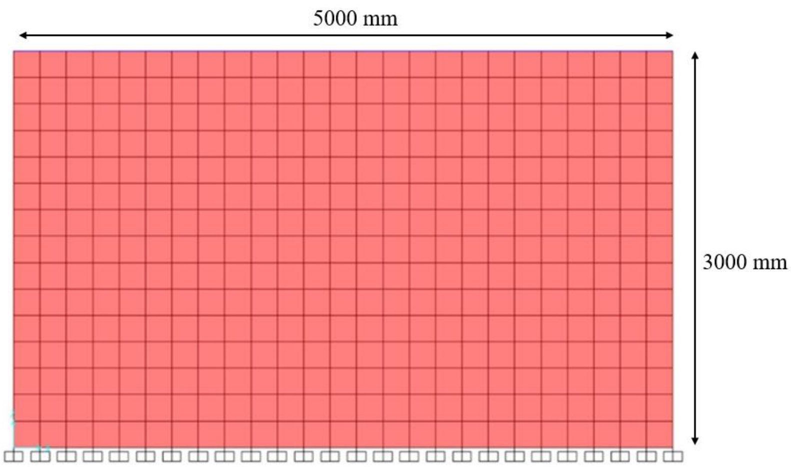

Figure 5. Wall 5 (5000x3000) mm.

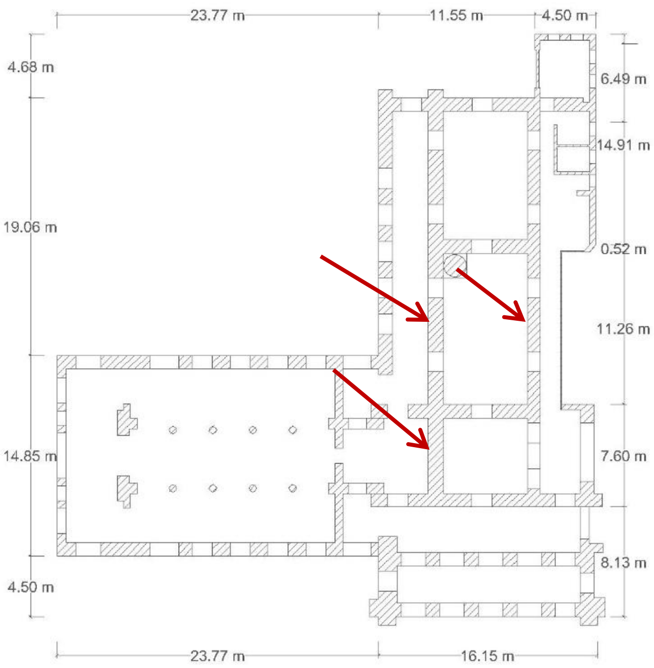

Figure 6. Ground Floor Plan

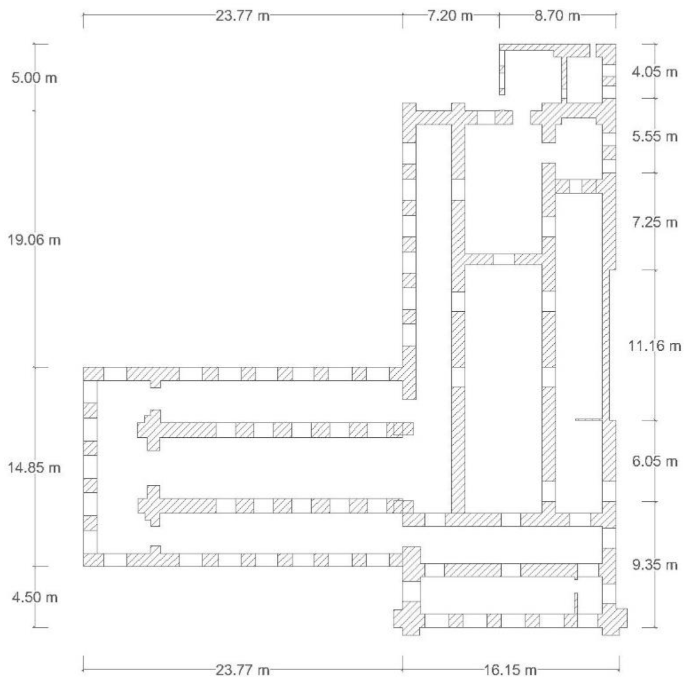

Figure 7. First Floor Plan

3 Theoretical evaluation of masonry wall panel

3.1 Failure mode calculations

Failure mode calculations according to Equations 1,2,3 and 4 and governing mechanisms are presented below,

Table 2. Failure mode and governing mechanism

Description	Wall 1	Wall 2	Wall 3
Rocking Resistance	4.05	2.43	11.25
Sliding Shear Resistance	159.750	159.750	266.25
Toe crushing Resistance	4.48	2.68	12.446
Diagonal Shear Cracking Resistance	154.3	92.66	257.39
Governing Mechanism	Rocking Failure	Rocking Failure	Rocking Failure

The above calculations gave us an idea about the mechanism of the failure in the wall.

3.2 Elastic site spectra

According to NBC 105-2077('Nepal Building Code 105', 2020),

For horizontal loading

$$C(T) = C_h(T)Z * I \tag{5}$$

$C_h(T)$= Spectral shape factor as per 4.1.2

As per clause 4.1.3.4; table 4-4

For Kathmandu, Soil type= D

From table 4-4

T_a= 0 (for Equivalent Static Method), T_c= 2.0, α= 2.25, K=0.8

From 5.1.2 Time period of wall

$$T = K_t * H^{\frac{3}{4}} \tag{6}$$

= $0.05*3^{3/4}$

=0.1140 seconds

Here, T_a<T<T_c,

C_h(T)= α = 2.25 from equation 4.1(2) in clause 4.1.2

Z= Seismic Zoning factor as per 4.1.4

Table 4-5 for Kathmandu Municipality, Z=**0.35**

I= Importance factor as per 4.1.5

Table 4-6; Importance class= III

So, I= 1.5

From equation 5

C(T)= 2.25*0.35*1.5= 1.1813

Clause 4.2: Elastic site spectra for Serviceability limit state

$$C_s(T) = 0.2 * C(T) \qquad 7$$

= 0.20*1.1813= 0.2363

Horizontal base shear coefficient,

Clause 6.1.1

For Ultimate limit state

$$C_d(T_1) = \frac{C(T1)}{R\mu *\ \Omega u} \qquad 8$$

$C(T_1)$ = Elastic site spectra as per 4.1.1= 1.1813

Rμ= Ductility factor as per 5.3, **Ω**u= Over strength factor for ULS as per 5.4

From table 5-2 Rμ= 2.0, **Ω**u= 1.2

$C_d(T_1) = \frac{1.1813}{(2.0*1.2)}$

= 0.4922

For Serviceability limit state,

$C_d(T_1) = \frac{Cs(T_1)}{\Omega s}$

$C_s(T_1)$ = Elastic site spectra for serviceability limit state as per 4.2= 0.2363

Ωs= Over strength factor for SLS as per 5.4

From table 5-2 **Ω**u= 1.1

$C_d(T_1) = \frac{0.2363}{1.1}$

= 0.2148

These manual calculations were performed to obtain the earthquake load and permissible stress.

4 Numerical modelling.

Theoretical displacement of a wall 1, having thickness of 1000mm, modulus of elasticity of 550 MPa, and a lateral force of 1000 N at top is calculated in Section 5.1 and found to be 0.0125 mm .The model is then analyzed in SAP2000 with different meshing sizes for linear shell area element. The lateral displacement for each case is found out and the result obtained is studied and compared with the manually calculated displacement and hence the suitable meshing size is selected for further analysis. The displacement values for different mesh size for linear shell area element, % difference between theoretically calculated and software calculated displacement is shown in table 1.

Table 3. Displacement for linear shell area element

Panel Meshing (mm*mm)	Linear Displacement (mm)	% difference between linear and theoretical displacement
No mesh	0.0089	28.8
1000x1000	0.0111	11.2
900x900	0.0114	8.8
700x700	0.0116	7.2
600x600	0.0116	7.2
500x500	0.0117	6.4
200x200	0.0119	4.8

From the above Table 3,the following features were observed:

1. On meshing towards smaller meshing size, the percentage difference between calculated lateral deformation and software results are reducing.

2. The percentage difference between theoretical and software displacement for the mesh size 200x200 is the minimum in the table.

3. Instead, a mesh size of 500mmx500mm is selected instead, to reduce the computational time for the analysis of the whole building later.

5 Results and Discussions

5.1 Validation of numerical model

Time period

According to (‘Nepal Building Code 105’, 2020) clause 5.1.2

Time period of wall(T)=k_t $H^{3/4}$ = 0.05*$3^{3/4}$ = 0.1140 seconds

Software result Time Period= 0.1139 seconds

Deflection of Wall due to lateral load(1000N).

$$\Delta = \frac{Ph^3}{3EI} + \frac{1.2Ph}{AG}$$

$$\Delta = \frac{1000 * 3000^3}{3 * 550 * 2.25 * 10^{12}} + \frac{1.2 * 1000 * 3000}{3 * 10^6 * 229.17}$$

$$\Delta = 0.0125$$

Software result of deflection (Figure 8) = 0.0119 mm

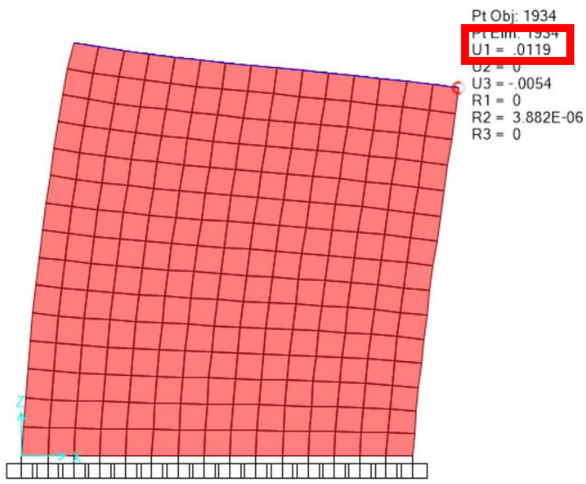

Figure 8. Deflection of Wall 1

5.2 Stress calculation

Check for Compressive Stress

Compressive strength of masonry unit = 1 N/mm^2

Mortar type L2.

Brick dimension is (195*125*45) mm.

Permissible compressive stress (f_c) in masonry shall be based on the value of basic compressive stress (f_b) and multiplying this value by factors known as stress reduction factor (k_s), Area reduction factor (k_a) and shape modification factor (k_p).

Basic compressive strength of wall fb = 0.25 N/mm^2 (From Table 8, IS 1905- 1987)

Height=3000mm

Effective height=1.5H (clz 4.3.1 table 4)

Thickness= 1000mm

Effective thickness= 1000mm (clz 4.5.1)

Slenderness ratio= Effective height/ Effective thickness= 4.5

Stress reduction factor k_s = 1 for of 4.5 (From Table 9, IS 1905 1987)

Area reduction factor (k_a) = 1, A being greater than 0.2 m^2

Shape modification ratio (k_p) = 1.0 (From Table 10, IS 1905-1987)

Permissible compressive stress in masonry (fc)

= 0.25* 1 * 1 * 1

= 0.25 MPa = 250 kN/m^2

Check for Shear Stress

According to IS:1905-1987 the Permissible shear strength is calculated as

$$f_s = 0.1 + \frac{f_d}{6} \quad (9)$$

$$= 0.1 + \frac{0.003}{6}$$

=0.1005 N/mm^2 i.e. 100.5 kN/m^2

5.3 Results from numerical modelling

The maximum and minimum stress developed in walls are presented in Table 4

Table 4. Stress developed on walls.

Wall	**Normal Stress (kN/m²)**		**Shear Stress (kN/m²)**	
	Maximum	Minimum	Maximum	Minimum
Wall 1	33.898	-33.898	38.93	-1.75
Wall 2	86.434	-86.434	66.415	-1.893
Wall 3	26.826	-26.826	39.780	-3.684

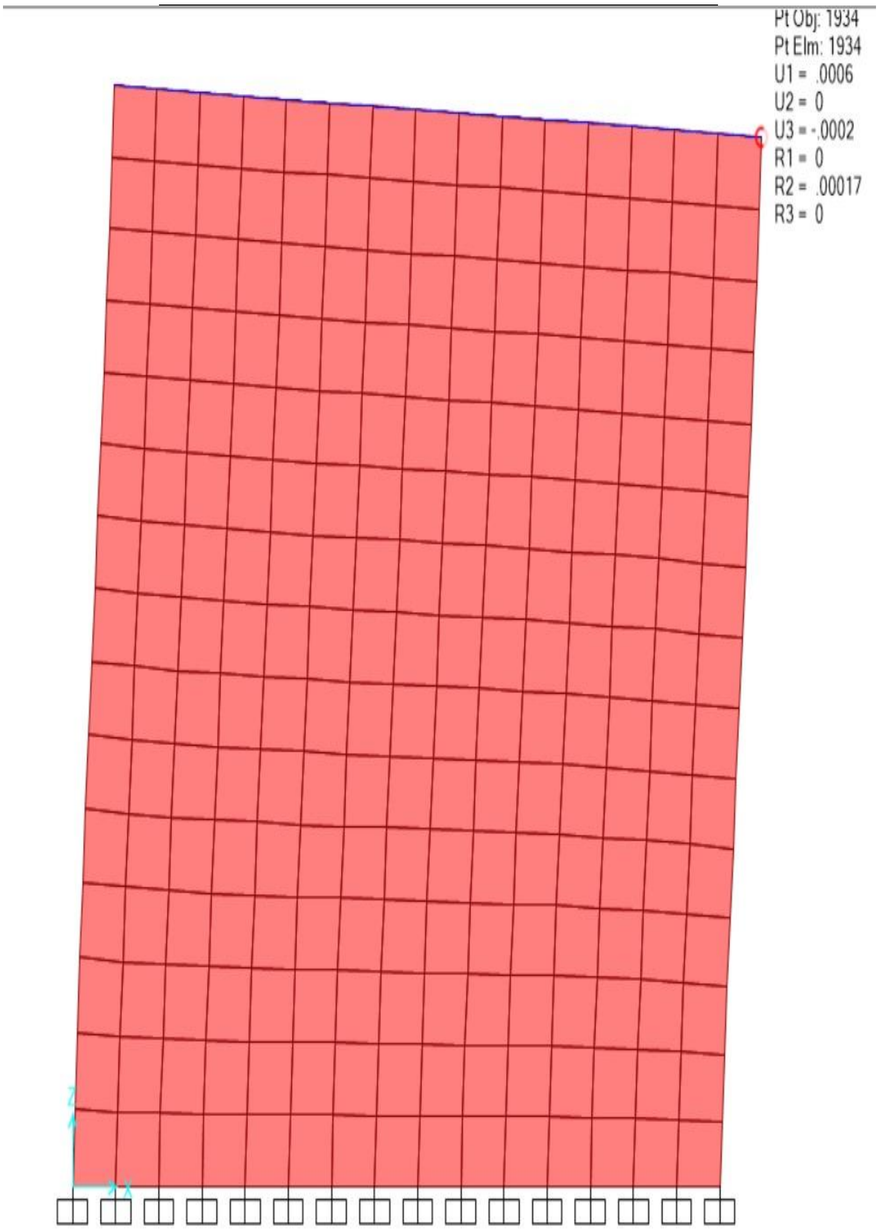

Figure 9. Deflection of wall 1 due to Eqx ultimate

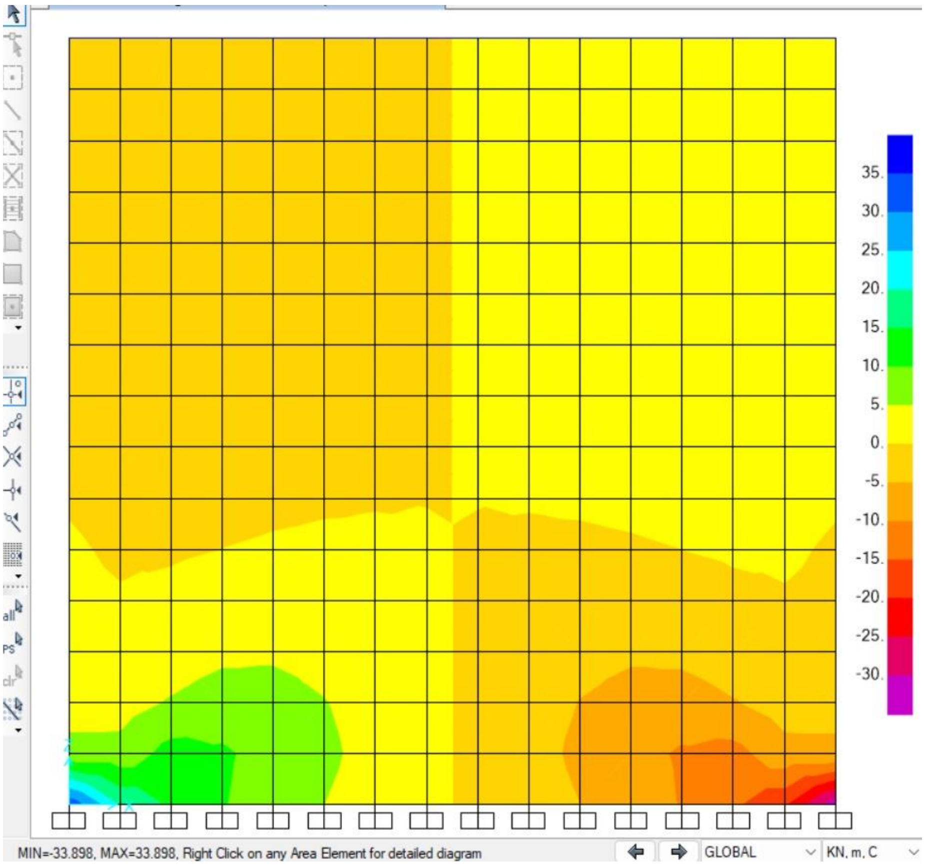

Figure 10. Normal Stress on Wall 1 due to Eqx ultimate

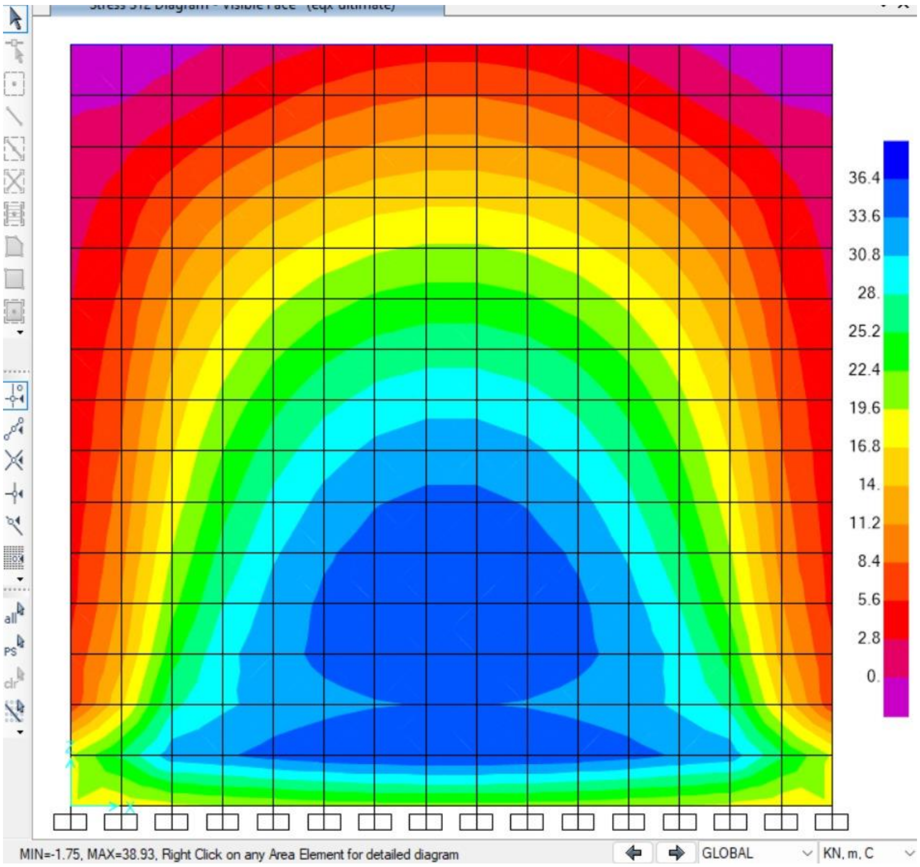

Figure 11. Shear stress on wall 1 due to Eqx ultimate

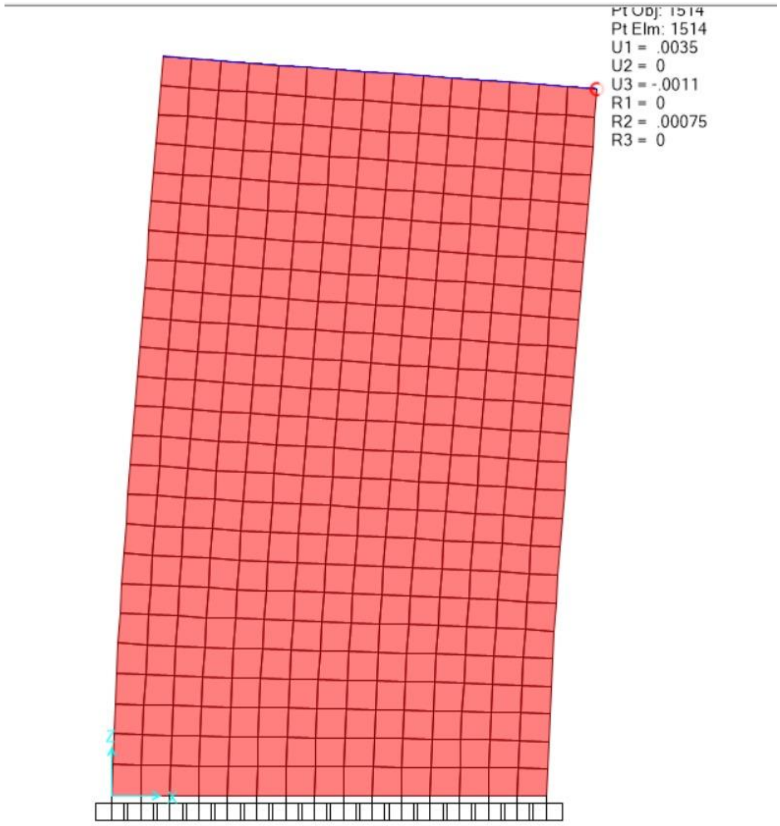

Figure 12. Deflection of wall 2 due to Eqx ultimate

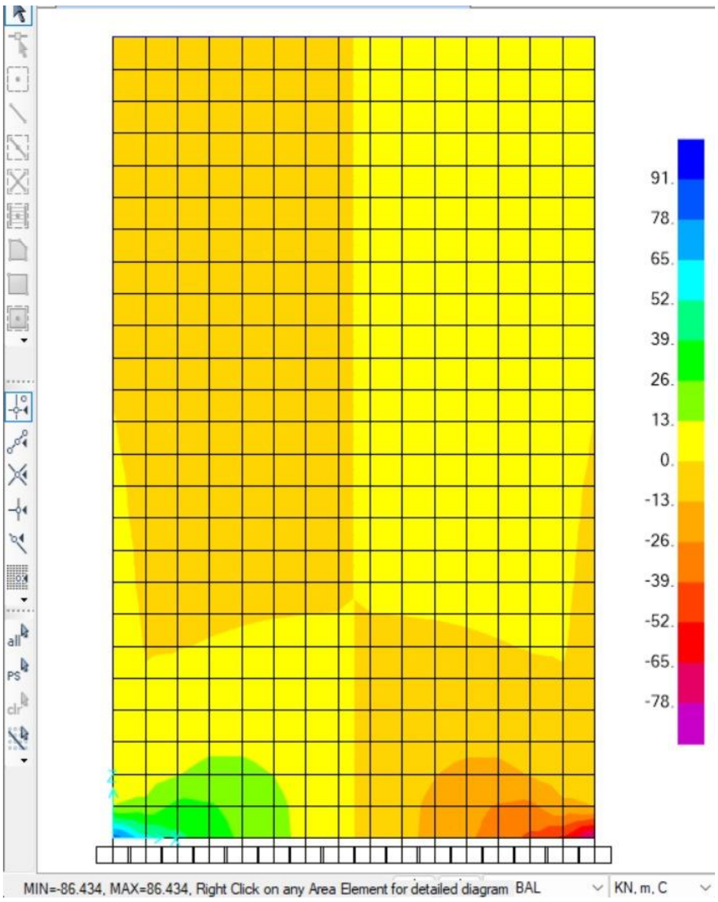

Figure 13. Normal Stress on Wall 2 due to Eqx ultimate

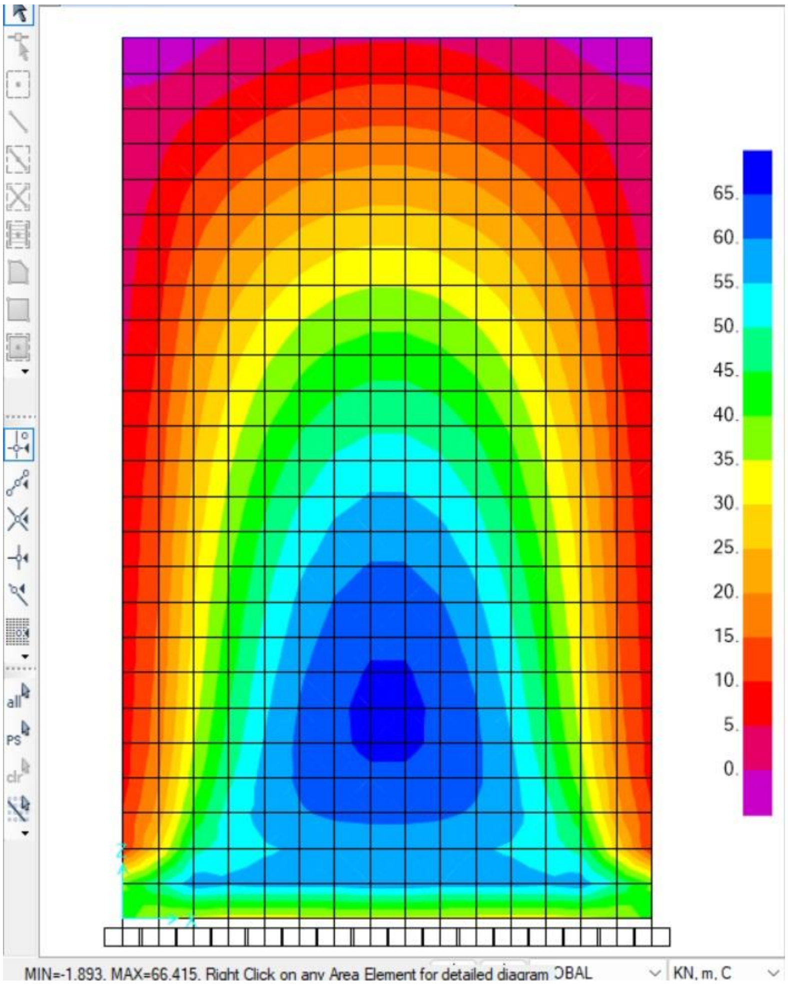

Figure 14. Shear stress on wall 2 due to Eqx ultimate

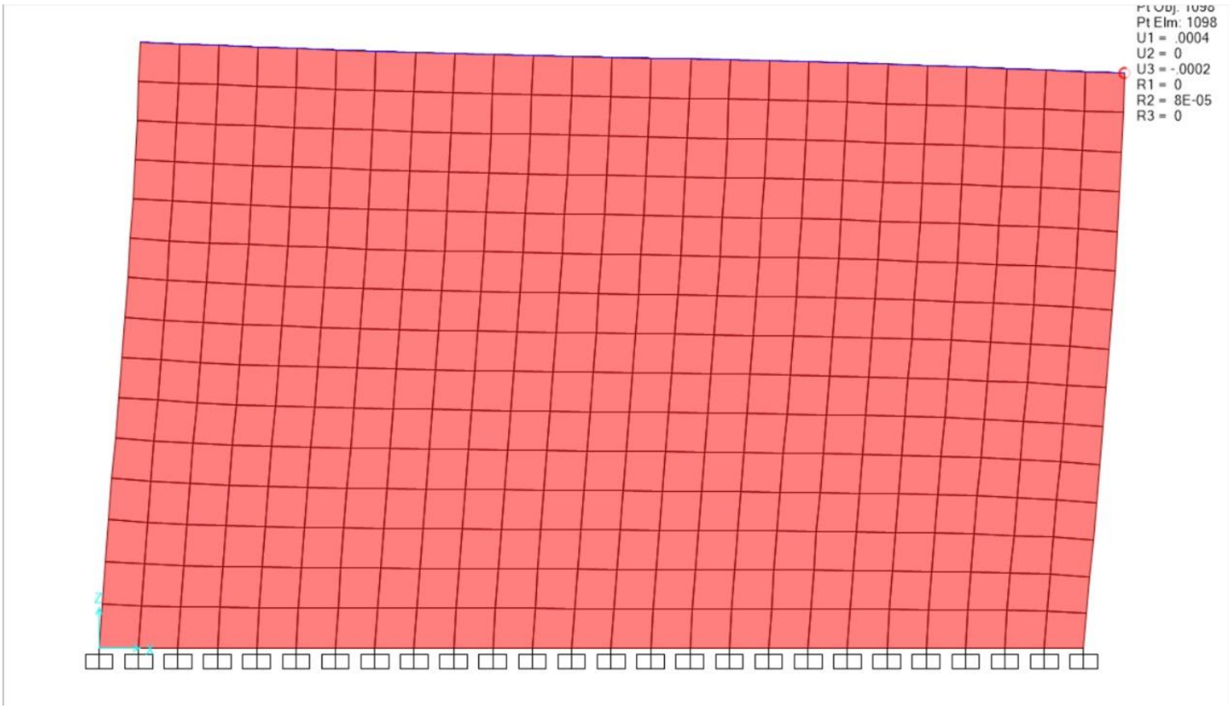

Figure 15. Deflection of wall 3 due to Eqx ultimate

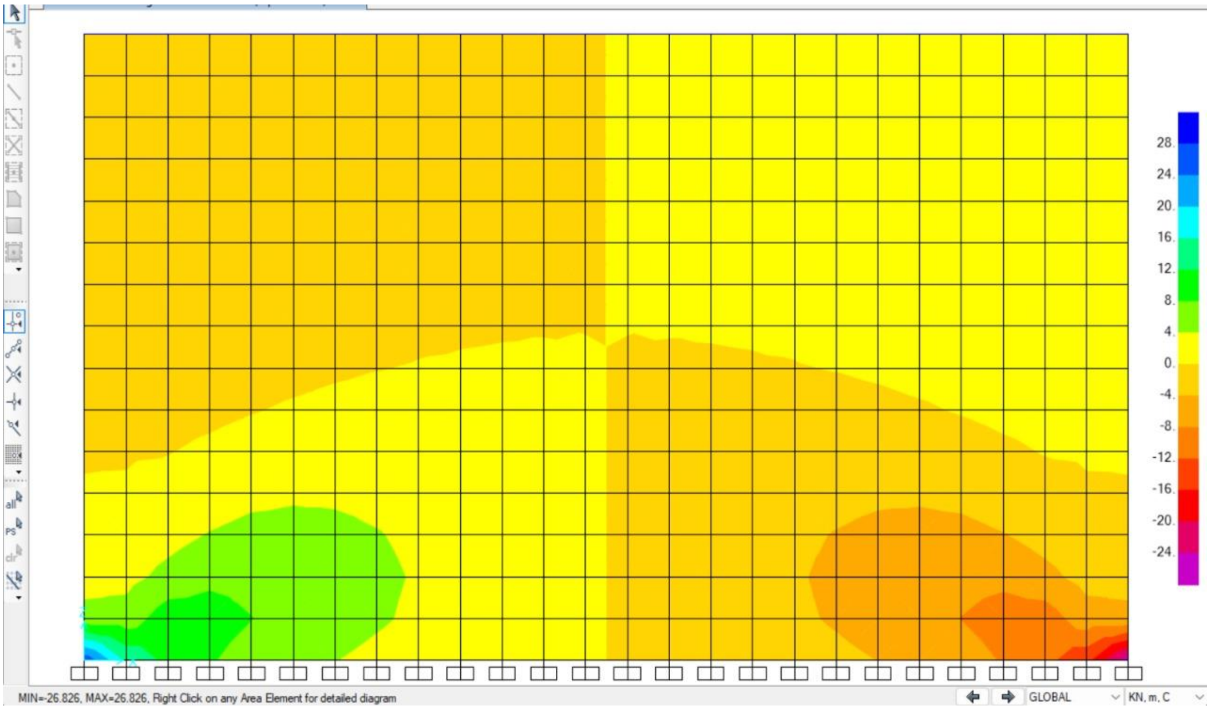

Figure 16. Normal Stress on Wall 3 due to Eqx ultimate

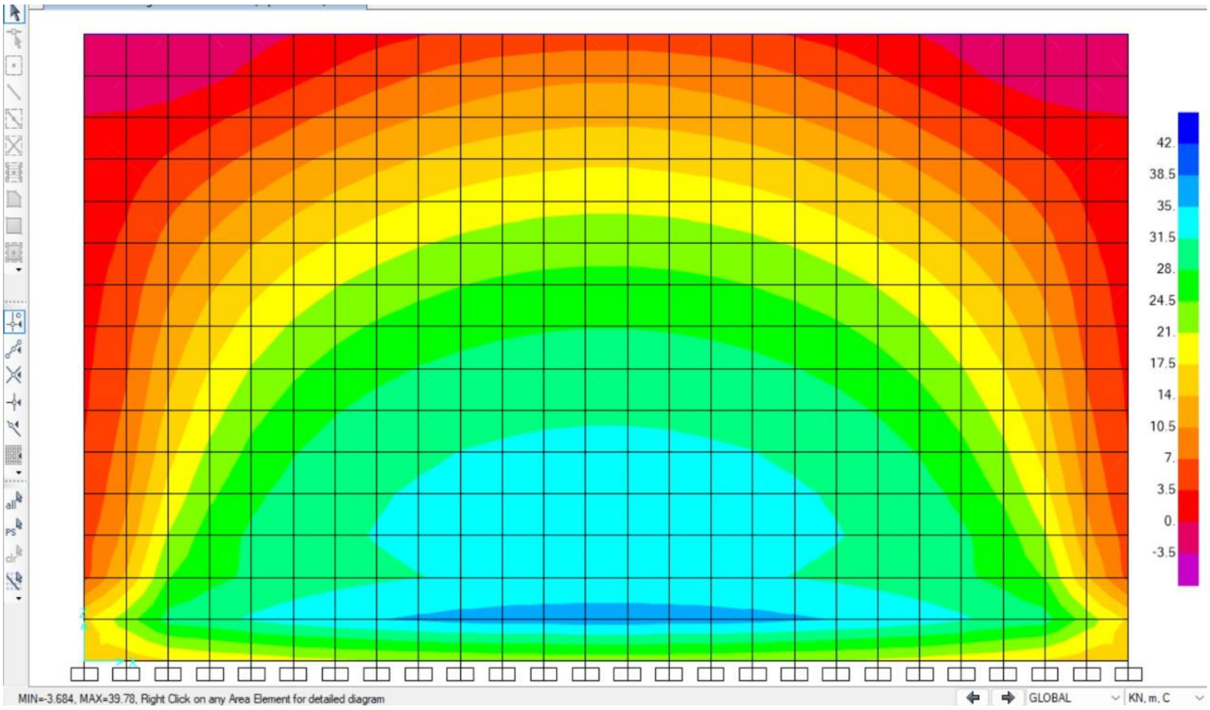

Figure 17. Shear stress on wall 3 due to Eqx ultimate

From the Table 4 and graph the Normal stresses in wall 1, wall 2 and wall 3 are 33.898 kN/m^2, 86.434 kN/m^2 and 26.826 kN/m^2 respectively which is below the permissible compressive stress 250 kN/m^2 and shear stresses on respective wall are 38.93 kN/m^2, 66.415 kN/m^2 and 39.78 kN/m^2 which is below the permissible shear stress 100.5 kN/m^2 so retrofitting of the wall panels is not necessary. However numerical modelling of the whole building is yet to be performed.

6 Conclusion

Three walls with various aspects of ratios behaved differently on application of lateral loads. Further, Earthquake loads were defined in SAP2000, and the walls were analyzed for the ultimate earthquake loads in accordance with NBC105-2020.

The following conclusions were extracted from the current study.

a) The numerical modelling was justified based on the fundamental period, deflection.
b) The failure mode was Rocking failure for all the studied wall models.
c) The seismic coefficient from NBC105-2020 was calculated to be equal to 0.4922 for Ultimate Limit State and 0.2148 for Serviceability limit state.
d) Finally, based on numerical modelling and stress analysis it was observed that the wall panels do not need retrofit.

Afterwards a full model of the building will be developed with the same methodology adopted in the current study and suitable retrofitting options will be decided.

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