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Effectiveness of Stone Concrete Block Masonry to Residential Buildings Through Numerical Simulation

Neeraj Bhandari [a], Kshitij Charana Shrestha [b], Subash Bastola [c]

[a] *Department of Civil Engineering, Pulchowk Campus, IOE, Tribhuvan University, Nepal*
[a] 078msste010.neeraj@pcampus.edu.np, [b] kshitij.shrestha@pcampus.edu.np, [c] subash.bastola@pcampus.edu.np

Abstract
This study presents an analytical approach to evaluate the seismic performance of stone concrete block masonry buildings, drawing upon data from experimental studies and applying it to case studies of building prototypes found in the Department of Urban Development and Building Construction (DUDBC) Design Catalogue I. This research assesses the efficacy of stone concrete block masonry models in contrast to both basic unreinforced and reinforced stone masonry models. Through a systematic sequence of seismic evaluations conducted using non-linear pushover analysis as well as the creation of fragility functions to depict vulnerability levels, the study emphasizes the vulnerability assessment of stone concrete masonry buildings when compared to unreinforced and reinforced stone masonry structures.

Keywords
Stone Concrete Masonry (SMC), Pre-cast stone masonry (PCSM), Stone-crete, Finite Element, Fragility

1. Introduction

“Nepal, highly earthquake-prone, faces challenges in making its buildings earthquake-resistant. Stone masonry structures, common in rural areas, are particularly vulnerable. Earthquake-induced losses are mostly due to man-made structures like buildings and dams. Reducing this threat involves adopting seismic-resistant building methods.”

Masonry structures, including those constructed with stone and brick, have a long-standing history in Nepal, with various historic architectural buildings, temples, and Dharamshalas built using these materials during different eras. Traditional construction methods for stone masonry buildings, which involve the use of cement and mud mortar, have been predominant in both urban and rural areas. However, the seismic vulnerability of such structures has become evident due to past devastating earthquakes. The 2015 Gorkha earthquake, in particular, highlighted the damage caused by low-strength masonry buildings, which constituted a significant percentage of the affected structures (NPC, 2015) [1].

Despite the provisions introduced in the National Building Code of Nepal (NBC 202:2015) for seismic strengthening, the implementation of complete design guidelines in rural reconstruction efforts has faced challenges, primarily due to limited skilled manpower and weak enforcement (Adhikari and D’Ayala, 2020) [2]. Consequently, it becomes crucial to explore more innovative and effective seismic-resistant solutions. The CBRI’s study aimed to reduce the thickness and skill required for random rubble walls, leading to the development of precast stone masonry blocks with a natural stone texture on one side using stone spalls and lean concrete mix. The use of such masonry leads to reduction in the seismic weight of the building as it makes it easier and practical to construct thinner walls as compared to the random rubble masonry. The seismic performance of such masonry can be ascertained by conducting tests such as shake table tests on the different building models and evaluating and understanding the performance till failure, however, it will be uneconomical and impractical to construct the different buildings of the stone concrete masonry, thereby arousing the need of numerical modeling and analysis which shall save both time, money and effort. Experimental studies were carried out in the Pulchowk premises to obtain the material properties of the masonry and to analyse the performance of the masonry, shock table test was performed on a half scale model building.The material properties were used to model the building in DIANA FEA software, the model was validated and further non-linear analysis was carried out on a full scale building chosen from DUDBC Design catalogue I.

2. Test on a half scale building

2.1 Modeling

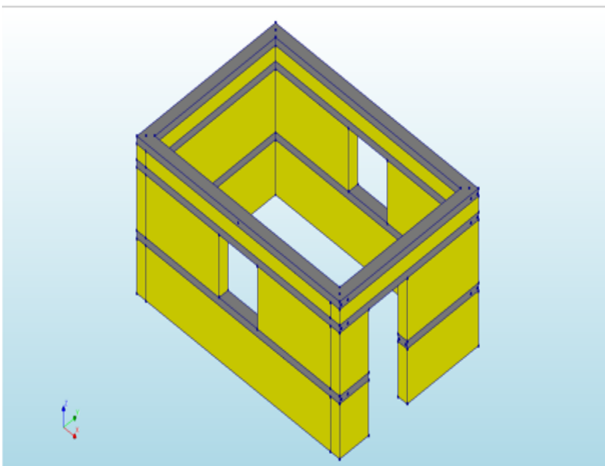


Figure 1: 3D Model of the half scale building

Using the properties of the stone concrete masonry obtained from wallet tests, a 3D model of a half scale building was created in DIANA which was the exact replica of the building priorly subjected to shock table test experimentally. The model was subjected to pushover loading and the failure pattern was observed which matched with those results obtained experimentally thereby validating the effectiveness of the model in capturing the response of the structure.

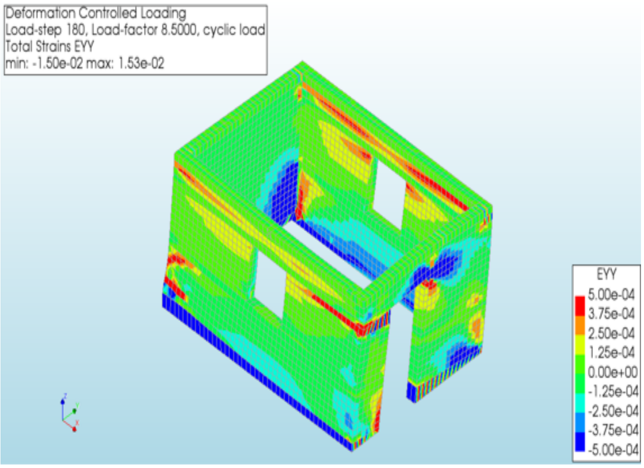


Figure 2: Failure pattern in the numerical model

Figure 3: Failure pattern observed in the experimental model

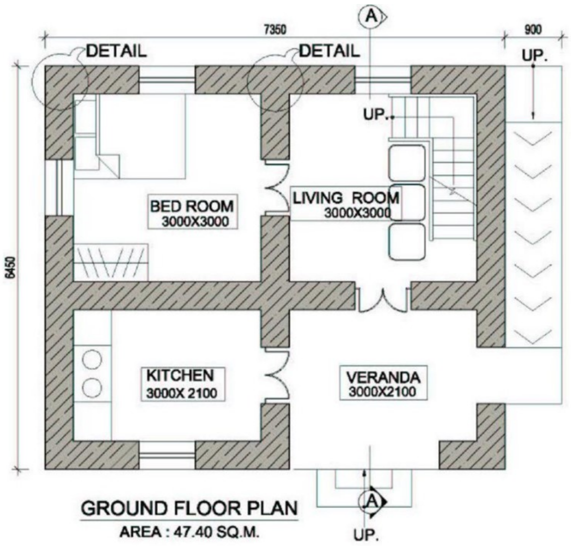
2.2 Validation of model

The model was subjected to three loadings; self weight, dead load from the roof and the displacement controlled load at a load step of 0.05 mm. The model was able to simulate the failure patterns as observed experimentally through shock table test. The compressive strains were found to be concentrated near the toe in the analytical model as well as the experimental model. Similarly the tensile strains were obtained at the interfaces between the bands and the masonry similar to that in the experimental case. Thus the numerical model was able to capture the experimental failure pattern and therefore the modeling technique could be used further to model full scale buildings.

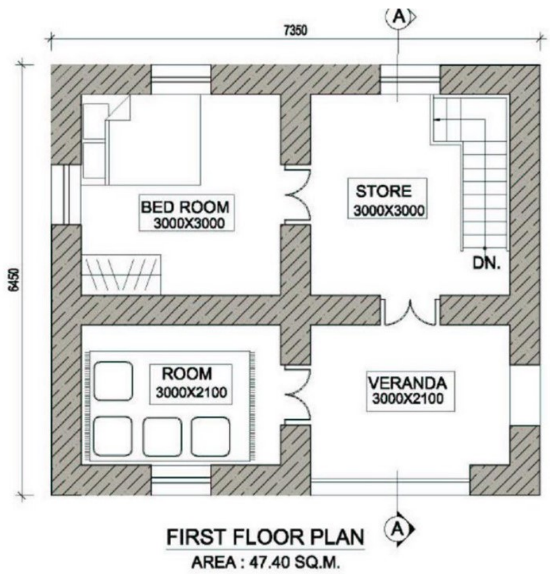
3. Case Study on a full scale Building

3.1 Building attributes

House model resembling SMC-2.2 from DUDBC Design Catalogue Volume I [3] were selected for the case study. A macro finite element modeling approach was employed. The concrete, masonry and steel are modeled as continuum elements and interface was assigned between the timber beams and the masonry. The wooden floor and floor beams are represented with linear models. For modeling simplicity, only the timber floors are included, and the weight of the mud filling is applied to the timber floor as a distributed load. The interaction between the timber floor beams and the walls is simulated using a structural plane interface that accounts for friction. Vertical reinforcement is also included at the corners and intersections of the building's walls. Figure-4 displays the building's layout.



(a)



(b)

Figure 4: Plan of Case Study Building a) Ground Floor Plan b) First Floor Plan (Source: DUDBC Catalogue)

The building has a wall thickness of 450 mm and a height of 5.3 m. The table in Table-1 lists the thickness of various horizontal bands and the provided reinforcement. Both the horizontal bands and vertical reinforcement are 12 mm in size.

Table 1: Band Detail

Band	Thickness (mm)	No. of Reinforcement
Sill	75	2
Lintel	150	4
Roof	100	2

3.2 FE Modeling

The case study building was simulated using the previously acquired and verified properties. It incorporates vertical reinforcements at the corners and wall intersections, as well as horizontal bands, including a sill band and lintel band on both floors, and a roof band at the top level of the structure.

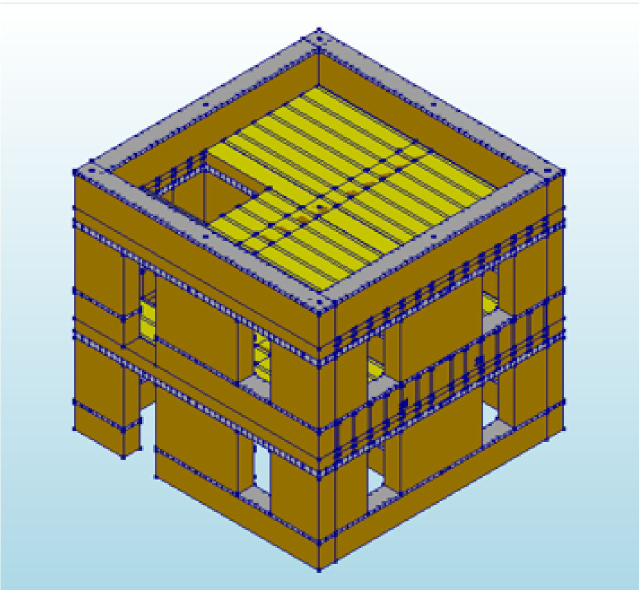


Figure 5: FE model of stone concrete masonry Model PCSM_2.5

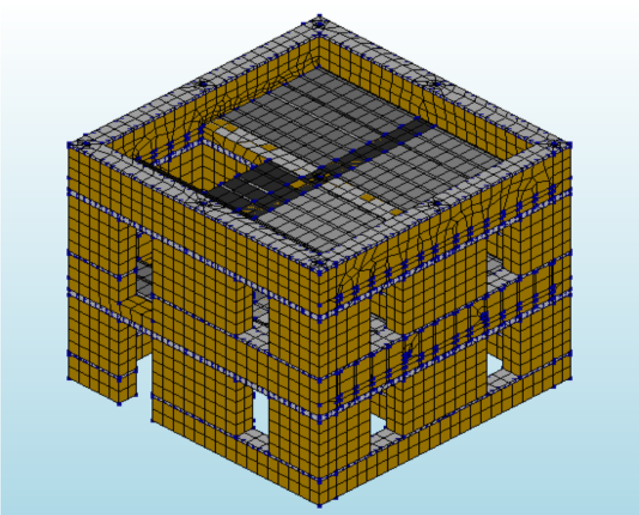


Figure 6: Meshing of stone concrete masonry Model PCSM_2.5 (300 mm mesh size)

A 3D macro-modeling approach is employed to replicate the building in the simulation. Solid elements are used to represent wall components and band elements, while line elements are utilized to model floor beams and reinforcements. Thin shell area elements represent timber floors. The roof is not included in the model, but its dead load is applied to the walls. A uniform area load of $2kN/m^2$ is applied at floor level, and the foundation is assumed to be securely anchored at ground level.

For the analysis of brittle materials like masonry and concrete, simulations use the total strain crack model. This model considers cracking as a distributed effect with directionality, employing exponential softening for tensile behavior and parabolic softening for compressive behavior. Crack width calculations use Rot's element-based method to compute the crack width. Reinforcements are fully embedded within their respective elements and exhibit no relative slip. The reinforcement bars are modeled using Von Mises Plasticity with strain hardening. Linear material models represent the wooden floor and floor beams.

Table-2 lists the material properties used in the model.

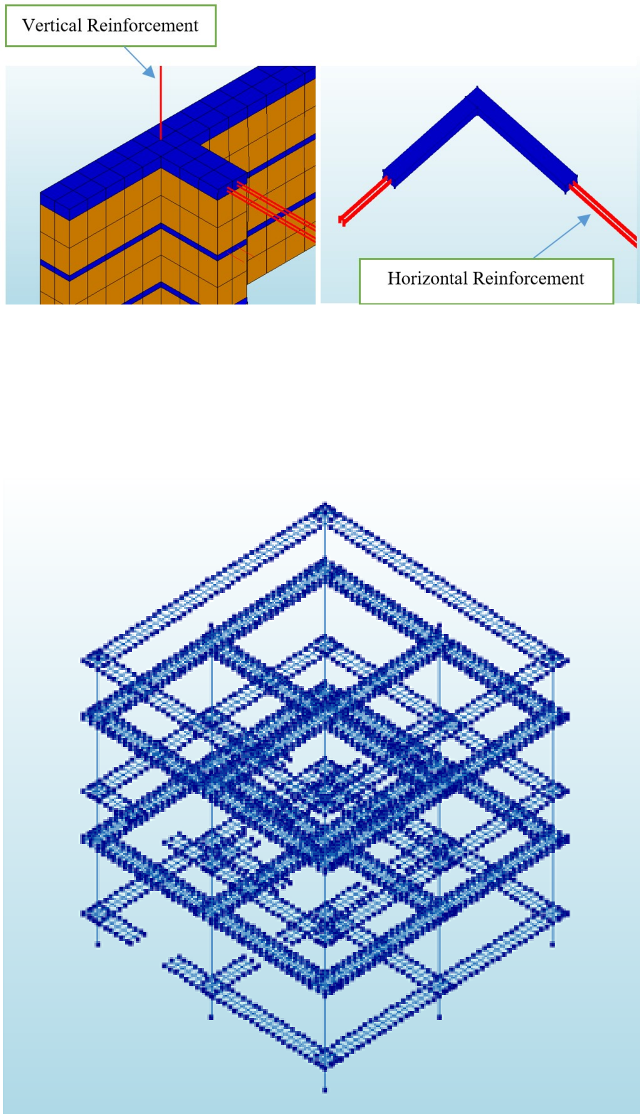


Figure 7: Reinforcement Details

Table 2: Material properties used for modeling

Material Properties	Value	Unit
Masonry		
- Unit Weight	23.22	kN/m^3
- Elasticity	965	N/mm^2
- Poisson's Ratio	0.2	
- Tensile Strength	0.12	N/mm^2
- Compressive Strength	3.5	N/mm^2
Concrete-M20		
- Unit Weight	25	kN/m^3
- Elasticity	22360.7	N/mm^2
- Poisson's Ratio	0.2	
- Tensile Strength	2.71	N/mm^2
- Compressive Strength	20	N/mm^2
Timber (Sal wood)		
- Unit Weight	7.9	kN/m^3
- Elasticity	12670	N/mm^2
- Poisson's Ratio	0.3	
Steel		
- Modulus of Elasticity	200000	N/mm^2

3.3 Analysis

Eigenvalue analysis was conducted to compare the time periods. A non-linear static analysis, specifically pushover analysis, was performed to derive the capacity curve by selecting the control node at the floor level. The structural performance point was determined using the capacity spectrum method by intersecting the capacity curve with the demand curve obtained from the response spectrum of NBC 105:2020 [4], following the procedure outlined by Otani et al. (2000) [5]. The variation in ground motion parameters at the performance point is associated with different damage states, reflecting the structure's vulnerability to a user-defined intensity measure. The fragility curve was generated using the methodology proposed by Wen et al. (2004) [6], considering various threshold values for different damage states, as indicated in Table 3.

Table 3: Threshold value for damage states

Damage State	URM [7]	RM [8]
Slight Damage	1/750	1/500
Moderate Dame	1/500	3/500
Extensive Damage	1/250	3/200
Collapse	1/100	1/50

4. Results and Discussions

From the pushover analysis the maximum values of base shear was 1627 kN which occurs at drift level of 0.2 i.e.the base shear increases gradually from zero to the maximum values. The pushover curves are shown in Figure 8.

The drifts for different PGA values for SMC 2.5 Reinforced are lower than that of SMC 2.5 Unreinforced and PCSM 5_450, however the drift values for PCSM 5_450 are found to be higher than that of the SMC 2.5 Unreinforced models.

Analytical fragility curves are derived through fragility analysis. The capacity spectrum analysis for various ground motion parameters provides a range of performance point values. These outcomes can be related to damage states, illustrating the structure's vulnerability to a user-defined intensity measure, displayed as a graphical representation along the capacity curve.

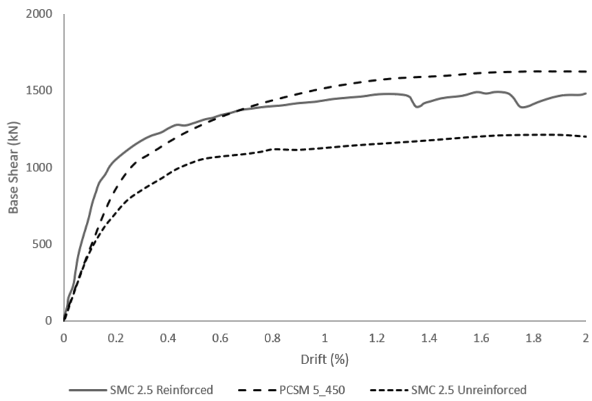


Figure 8: Pushover curve along the direction of Fundamental Mode for stone concrete model and SMC reinforced and unreinforced models

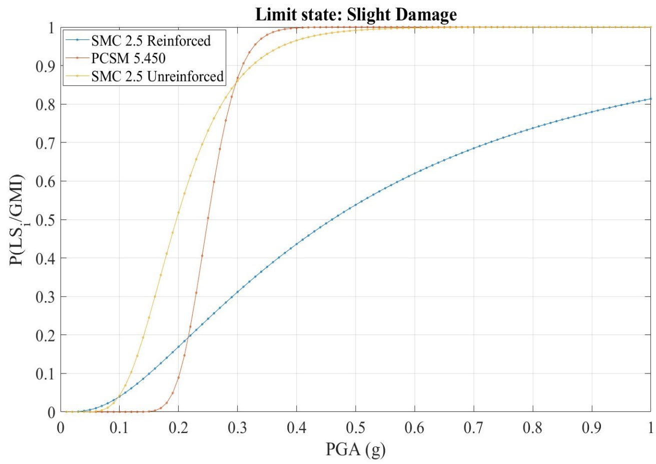


Figure 9: Fragility Curves for SMC 2.5 Reinforced, unreinforced and PCSM_450 for Slight Damage State

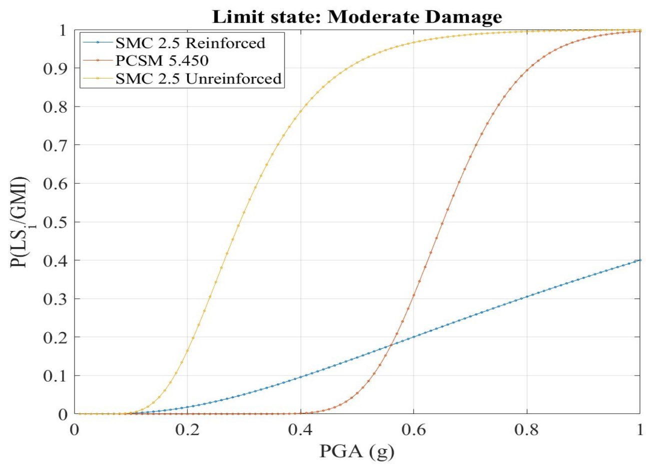


Figure 10: Fragility Curves for SMC 2.5 Reinforced, unreinforced and PCSM_450 for Moderate Damage State

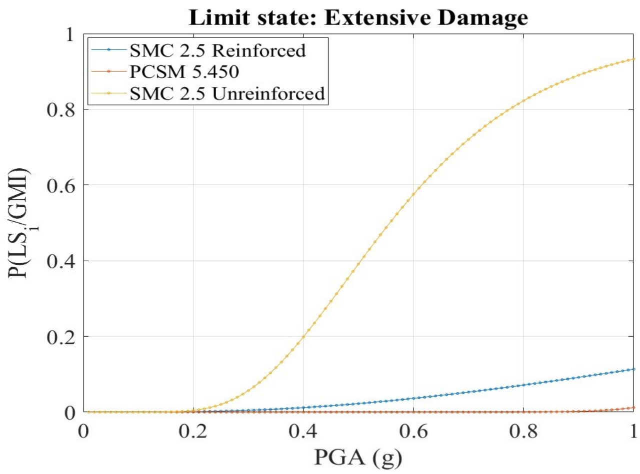


Figure 11: Fragility Curves for SMC 2.5 Reinforced, unreinforced and PCSM_450 for Extensive Damage State

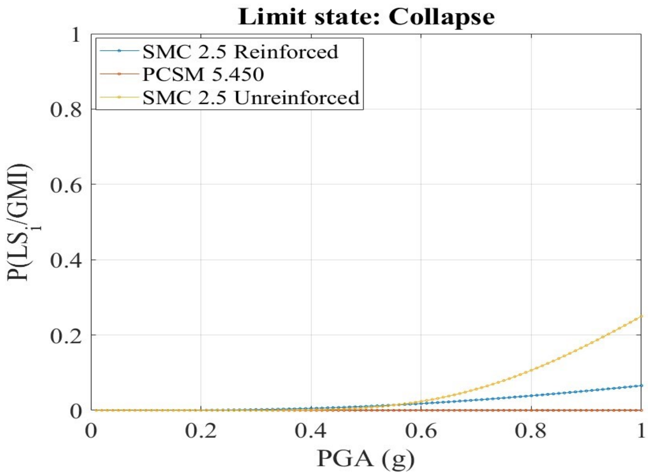


Figure 12: Fragility Curves for SMC 2.5 Reinforced, unreinforced and PCSM_450 for Collapse State

5. Conclusions

An analytical study for the seismic assessment of stone concrete masonry buildings with non-linear properties have been done using DIANA FEA and has been compared with that of conventional reinforced as well as unreinforced stone masonry buildings. The pushover analysis was done for the generation of capacity curves, which was later used to develop fragility curve through capacity spectrum method. The major objective of this research was to determine the seismic vulnerability of stone concrete masonry buildings.

In case of these model buildings, the probability of exceedance for slight damage state for SMC 2.5 was found to be higher below 0.2 PGA and lower above it as compared to PCSM 2.5. Similarly the probability of exceedance for moderate damage for SMC 2.5 was found to be higher below 0.55 PGA. The probability of exceedance for extensive damage state was found to be lower than that of both reinforced and unreinforced stone masonry for all values of PGA. Similar result was observed for the collapse state.

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