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Seismic Performance Evaluation of Stone Masonry Building

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Abstract

Unreinforced stone masonry has been popular mode of construction in Nepal. Past earthquakes in Nepal had shown evidence of large damage in stone masonry buildings. The study is focused on typical residential unreinforced stone masonry buildings. For the analysis, two dimensional modeling is done in thick shell modeling of wall; timbers in both directions were provided assuming only one direction timber acts one at a time. From the research it is found that the existing forms of the buildings are highly vulnerable for future earthquake. The piers of stone masonry buildings fail in tension. The insufficient floor rigidity, improper connection between the walls, ongoing deteriorated structure elements reduces the performance of the building.

Keywords

Stone Masonry, Seismic Coefficient, Seismic Performance, Response Spectrum

1. Introduction

1.1 Background

Nepal lies on seismically active zone between Indian plate and Eurassian plate. Nepal has encountered many earthquakes in the past. Including the recent Gorkha earthquake Nepal has suffered 16 major earthquakes since 1310.Some of the devastating earthquakes in Himalaya are Pakistan(Kasmir 2005, Mw 7.6), India (Assam 1950, Mw 8.5) , Kangara 1905,Mw 8, Nepal-Bihar 1934 Mw 8.3 , Shillong 1897 Mw 8.3 , Gorkha earthquake 2015 Mw 7.8 . In the recent Gorkha earthquake of 25 April 2015, almost 81% of the residential houses were totally damaged among which most of the buildings were stone masonry buildings with mud mortar.
The performance of the stone-masonry structures during earthquakes is dictated by the quality of construction and the structural adequacy of the components. Quality of construction includes method of construction, quality of materials and workmanship. Traditional stone masonry buildings are mainly constructed of random rubble stone masonry, in which rough stones are piled up without any mortar or with mud mortar. Such buildings are constructed based on traditional techniques using locally available material and Roofing Technique. Stone masonry buildings are common in most of the villages in Nepal, due to easy availability of construction material and associated low cost. Seismic forces are ignored while constructing these buildings.Mostly slate is used as roofing material. The stone masonry buildings are characterized by heavy mass, very low strength compared to the mass density and brittle nature of failure. The absence of reinforcements and poor quality of mud mortar or no mortar, leads to buildings that fail in brittle manner, without allowing for energy dissipation and providing no warning to the occupants before collapse during seismic events. The unreinforced stone masonry structures constructed in Bajura district of Nepal are non-engineered. Most of the structures are traditionally constructed with mud mortar. Most of the unreinforced stone masonry structures are two storey houses with additional top small gable walled portion at the top.

The failure of unreinforced stone masonry is mainly due to out-of-plane, in-plane failure mechanism, delaminating of wall and connection failure between roof covering and wall/timber. This is due to construction and material deficiency during and after construction of the masonry structure. Construction deficiencies are due to poor workmanship, poor construction technique, improper connection between member of structures, large size of opening, lack of interlocking stones in wall, filling of small stone chips between two layer of stone wall and material deficiencies are due to poor construction material , old aged structure. Traditionally constructed stone masonry buildings are over 50 years with no maintenance in the existence buildings. So the study of seismic performance of such building is necessary to define the vulnerability of the buildings.

2. Objectives of Study

The general objective of this thesis is to assess the seismic performance of non-engineered residential stone masonry building.

3. Methodology

For masonry buildings, the stiffness depends upon wall geometry, opening and thickness of the wall. So for the real building was taken into account for the analysis, modeled to determine the distribution of seismic forces between masonry and timber framed structure.

3.1 Analysis Methods

Analysis for the design earthquake actions shall be in accordance with one of the following methods:

1. The Seismic Coefficient Method
   The seismic coefficient method is one of the static procedures for earthquake resistant design of structures. In this method, the dynamic seismic force is transmitted into an equivalent static force on the building and is distributed throughout the height of the building.
   As per the Indian Seismic Code IS:1893 (Part 1) , horizontal seismic coefficient Ah can be determined by the following expression:

$$A_h = \frac{Z}{2} * \frac{I}{R} * \frac{S_a}{g} \tag{1}$$

   The total design lateral force or design seismic base shear (VB) along any principal direction shall be determined by the following expression:

$$V_B = A_h * W \tag{2}$$

2. The Modal Response Spectrum method
   The dynamic response of a structure against an earthquake ground motion is governed by the natural period and the damping coefficient of the structure, and the predominant components of the ground motion. Response Spectrum Method is used here for the model analysis to understand the dynamic behavior of structure.
   Two types of modal analysis; Eigen vector and Ritz vector analysis are generally used to understand the dynamic response of a structure. Eigen vector analysis determines the undamped free vibration mode shapes and frequency of the system.
   The undamped Eigen values and Eigen vectors of the MDOF system are found form the characteristic equation:

$$\{[k] - \omega_i{}^2[m]\}\phi_i = 0 fori = 1,2,3...n \tag{3}$$

$$det(\{[k] - \omega i^2[m]\}) = 0 \tag{4}$$

where, ωi^2 = Eigen values of the ith mode
ϕi = Eigen vector or mode shape of the ith mode
ωi = natural frequency in the ith mode.
Buildings with regular ,or nominally irregular plan configuration may be modeled as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction of consideration.

(a) Design lateral force at each floor in each Mode: The peak lateral force at floor i in k th mode is given by:

$$Q_{ik} = A_k * \phi_{ik} * W_i \tag{5}$$

where, Ak = Design horizontal acceleration spectrum values using the natural period of vibration

(b) Storey shear force in each mode: The storey peak shear force at i th storey in mode k is given by:

$$V_{ik} = \sum_{j=i+1}^{n} Q_k \tag{6}$$

3.2 Seismic assessment criteria

The performance criteria have been developed for most stone masonry's structural and non-structural components. Only the acceptance criteria for walls and piers are given in this paper. According to Canadian Guidelines for seismic Evaluation of Existing Buildings (CGSEEB) (NRC1992), pier rocking is given by:

$$V_r = 0.9 * P_D * \frac{D}{H} \tag{7}$$

where, P_D is the axial load on the pier and D and H are the pier's width and height.
Shear resistance is given by:

$$V_d = v_m * \frac{D_t}{1.5} \tag{8}$$

where, t is the thickness of the wall and v_m is the masonry shear strength given by:

$$v_m = 0.56 v_t + 0.75 * \frac{P_D}{A} \tag{9}$$

where v_t is the mortar shear strength (not less than 0.2MPa).
For the uncracked pier, the load required to initiate cracking at the pier's top and bottom based on strength of the materials principles, is given by:

$$V_c r = f_t \frac{tD^2}{3H} + \frac{P_D D}{3H} \tag{10}$$

where, f_t is the tensile strength of the masonry.
Deformation criteria: The deformation acceptance criterion for walls and piers is:

$$\Delta_D \leq k \cdot \Delta_C \tag{11}$$

Where, Δ_D and Δ_C are, respectively, the demand deformation obtained from the seismic analysis and the deformation capacity. The latter for rubble stone-masonry is further defined as:

$$\Delta_C = 0.0004h \tag{12}$$

The assessment of the structure's seismic response is based on deformation and strength criteria. For stone-masonry structures, the following responses are needed for the assessment of seismic performance of the subsystems:

1. Maximum lateral displacements
2. Maximum inter-storey drift
3. Maximum resultant forces
4. Maximum resultant stresses

3.3 Numerical Modelling

Two dimensional modeling is done in Thick shell modeling of wall. In this thesis, timber in both directions were provided assuming only one direction timber acts one at a time i.e. for a direction of seismic force, only lateral direction timber provides stiffness. For modeling the timber floor, a three dimensional linear beam element is used to model the timber joist. The connection of the timber floor/roof with the masonry wall is neglected and assumed that it simply rest on walls.by considering the fact that the timber nails. So simply supported connection is used for modeling the joint between the timber joist and masonry wall. The equivalent timber is assigned in both directions assuming the response of building in both directions. Equivalent timber is obtained as a timber of depth 0.1 and width 0.15m at spacing of 30cm C/C spacing. In such models, masonry wall is modeled as thick shell element of thickness 0.45m for external and internal wall. Shear walls were designed such that all piers would successively develop a pier-rocking behavior during seismic response. The initial portion of the curve is the linear elastic stiffness of the shear wall, while the second portion has a zero slope representing the rocking behavior of the piers in the shear wall.

The mechanical properties of the masonry are taken from [1] works on Numerical study on unreinforced semi-dressed stone masonry for Build-Back-Better in Nepal.

Young's Modulus E = 850 N/mm^2
Poisson ratio μ = 0.25
Mass density ρ = 2000kg/m^3
Tensile strength f_t = 0.05 N/mm^2
Compressive Strength f_c = 1.8 N/mm^2

Mechanical properties of the timber are taken as:
Young's Modulus E = 1160 N/mm^2
Poisson ratio μ = 0.25
Mass density ρ = 450kg/m^3

The base of the model is considered to be rigid. No bands are provided in the existing buildings so unreinforced masonry modeling is done in numerical modeling. Slate roof is provided at the top.

Table 1: Dimensions of Typical Residential Houses

Residential House	Model 01	Model 02	Model 03
Height(m)	2.00	4.00	4.00
Base Length (m)	5.00	8.00	11.00
Base Width(m)	4.80	5.00	5.00
Wall Thickness(m)	0.45	0.45	0.45

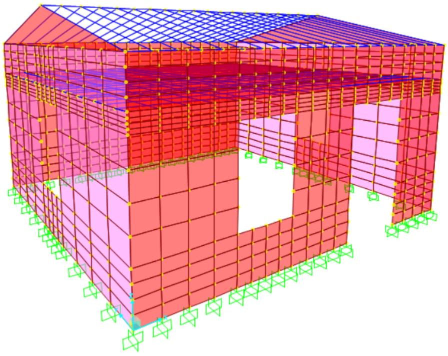

Figure 1: 3D modelling of Model 01

4. Analysis and Result

The models of the selected buildings of different storey that represent the buildings of the locality are modeled in SAP 2000 V.20. The model is analyzed by Seismic coefficient method, in which the seismic effect i.e., a horizontal force is considered as the percentage of total weight of the building. Seismic coefficient method is used as described by IS 1893(Part I):2002. Results of the analysis are presented in Tables 2 to 7.

Table 2: Drift Ratios of three models

Model	Displa-cement(m)	Height(m)	Drift Ratio
Model 01	0.67	2.00	0.335
Model 02	1.01	4.00	0.253
Model 03	0.49	4.00	0.123

Theoretical formulation is done for the Model 01. According to Canadian Guidelines for seismic Evaluation of Existing Buildings (CGSEEB) (NRC1992):
Maximum force transmitted by diaphragm to shear wall W_d =140.76kN
Tributary load of each shear wall W_{wX1}=58.44kN
Corresponding expected in-plane seismic load on shear wall F_{wX1}=0.4($W_{wX1}+W_{d/2}$) = 51.53kN

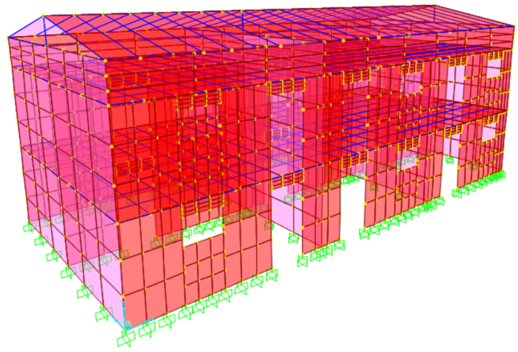

Figure 2: 3D modelling of Model 02

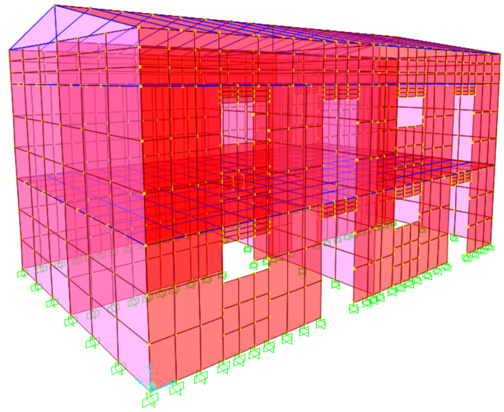

Figure 3: 3D modelling of Model 03

Table 3: Comparison of Time Period of models with Empirical Formula

Model	Empirical Formula (Sec)	Analysis Time Period (Sec)
Model 01	0.080	0.062
Model 02	0.108	0.080
Model 03	0.091	0.073

Table 4: Natural Time period and Mode shapes of Model 01

Mode Number	Period (Sec)	Description of mode shapes
1	0.062	Large relative motion in X-axis
2	0.054	Large relative motion in Y-axis
3	0.052	Rotation about Z-axis

5. Discussion and Conclusion

In this study, numerical analysis was carried out in different models. The study is focused on typical

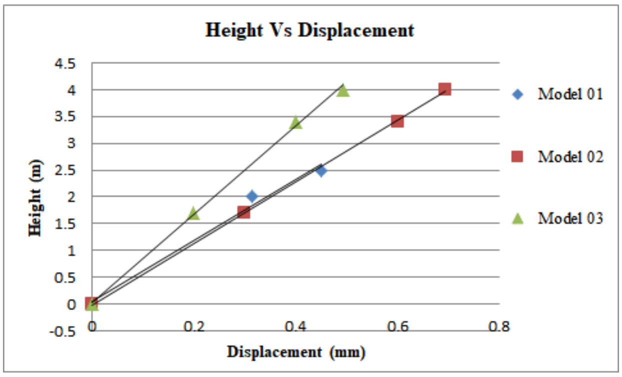

Figure 4: Height vs Displacement curve for different Models

Table 5: Max Stress developed due to EQx (MPa)

Model	Tension	Compression	Shear
Model 01	0.156	-0.146	0.067
Model 02	0.169	-0.168	0.081
Model 03	0.162	-0.177	0.066

Table 6: Max Stress due to EQy from Seismic Coefficient Method (MPa)

Model	Tension	Compression	Shear
Model 01	0.176	-0.175	0.063
Model 02	0.167	-0.164	0.092
Model 03	0.171	-0.165	0.071

Table 7: Max Stress due to EQy from Seismic Coefficient Method (MPa)

Pier	Width D (m)	Height H (m)	Axial load P_D (KN)	Rocking V_r (KN)	Cracking V_{cr} (KN)	Shear V_a (KN)
Pier 1	1.20	1.20	30.33	27.29	19.11	55.48
Pier 2	1.00	1.20	25.28	18.96	13.27	46.24
Pier 3	1.00	1.80	25.28	12.64	8.85	46.24
			$\sum V_r$=	58.89		

residential unreinforced stone masonry buildings. In order to increase the performance of the building for future earthquake, seismic evaluation of stone masonry building is required. After studied, it was found that the existing forms of the buildings are highly vulnerable for future earthquake. The insufficient floor rigidity, improper connection between the walls, ongoing deteriorated structure elements reduces the performance of the building. The walls of the masonry buildings are vulnerable and fails under tension, compression and shear developed during the earthquake. The compression and tensile strength of the masonry buildings exceeds the permissible limits while comparing with the codal provisions (permissible stress for tension 0.05Mpa, compression 0.48Mpa, Shear 0.1Mpa). The piers of stone masonry buildings fail in tension. The results obtained from the response spectrum analysis are similar to that of the seismic coefficient method. There was no significant change in stresses in both methods. The storey drift obtained from the model are within the permissible limits calculated (i.e., 0.0004h).

According to Canadian Guidelines for seismic Evaluation of Existing Buildings (CGSEEB) (NRC1992) was done. The maximum load required for initiating rocking, cracking and shear in the pier elements was calculated and compared with the stress developed in the masonry structure. For all the piers, rocking resistance is less than the shear resistance and total rocking resistance of each shear wall is sum of individual component i.e., $\sum V_r = 58.89$kN.Since, $F_{wX1} < \sum V_r$, the specimen would theoretically be able to resist the highest seismic lateral force.

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