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Response Reduction Factor for Mansory Buildings

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Abstract

Most of the seismic codes used today incorporate the nonlinear response of a structure by providing an appropriate response reduction factor so that a linear elastic force-based approach can be used in designs. This study focuses on evaluating the response reduction factor for masonry buildings with different mechanical properties, which are used in modern codes to scale down the elastic response of the structure. Using a similar frame-approach, a nonlinear static pushover analysis is carried out on the analytical models of masonry building in finite element analysis software SAP 2000v 20.0.0. The response reduction factor components, flexibility, and overstrength were computed from the results obtained from the nonlinear static pushover analysis. Finally, the response reduction factor is evaluated for different masonry buildings.It is concluded that the R-value given in IS: 1893-2016 for unreinforced masonry is not recommended for random rubble stone masonry buildings in mud mortar.

Keywords

Response reduction factor, masonry buildings, ductility reduction factor, overstrength factor, redundancy

Introduction

Nepal is one of the most earthquake-pronecountries, and the primary source of earthquakes in Nepal is the subduction of the Indian plate underneath the Eurasian plate. Several significant earthquakes were reported in 1255 AD, 1810 AD, 1866 AD, 1934 AD 1980 AD, and 1988 AD in Nepal (Bilhman *et al.,*1995; Chitrakar and Pandey1986). The 1993 earthquake mostly affected the Kathmandu valley and resulted in more than 8,500 deaths. About 19% of the buildings were destroyed, and 38% of the buildings were severely damaged. The earthquake of August 21st, 1988, with a magnitude 6.6 occurred in the Eastern part of Nepal, killed 721 people, injured 6,553, and damaged 66,541 buildings (22,695 destroyed, and 43,846 were severely damaged (Thapa, 1988). Moreover, the earthquake of April 25th, 2015, with a moment magnitude 8.1 occurred in the Gorkha district of Nepal. It killed nearly 9,000 people, injured 22,000, and damaged 773,378 buildings (501,201 nos. of the building were collapsed, and 272,177 buildings were destroyed (Ministry of Home Affairs, Nepal 2015). Most of the buildings damaged by the earthquake were masonry building.

As per the construction trend in Nepal, there are three significant buildings: reinforced concrete (RC) frame structures, brick masonry structures, and stone masonry structures. Cement sand mortar and mud mortar have been used to construct masonry structures. Masonry buildings are commonly practiced in rural areas, while RC frame buildings are built in urban areas.

Due to the availability of local construction materials: stone, brick, mud, lime, lime blended with black lintel, the majority of building constructed in the rural area of Nepal are of masonry. On the grounds of low strength materials and reduced construction, most of the building damaged during recent Gorkha earthquake is the masonry; nearly 800,000 buildings have been destroyed. Reconstruction of these building is going on, but there is a lack of building materials for reconstruction to full fill the demand for building materials. Alternative materials have been introduced as compressed soil earth blocks (CSEB).

In the seismic design of buildings, the nonlinear effect is incorporated by using appropriate response reduction factor (R). The concept of response reduction factor is to de-amplify the seismic force and to incorporate nonlinearity with the help of over strength, redundancy, and ductility (Keerthi and Philip 2017). Different codes and guidelines specify the response reduction factor to scale down the elastic response reduction of a structure. The same factor R is termed as "response modification coefficient" in ASCE7-2016" behavior factor" in Eurocode 8:2011, and "response reduction factor" in IS 1893:2016. The majority of the buildings in Nepal are designed using the equivalent static method, which is based on the response reduction factor. Design loads are obtained by reducing the earthquake loads by the 'R' factor. By lowering the earthquake loads, the structure enters into the inelastic range. Therefore, the structures have to experience a large deformation ratherthan dissipate the energy. While designing the structures according to IS 1893:2016, the value of the response reduction factor in masonry buildings is usually taken as 1.5, considering the unreinforced masonry. However, in IS 1893:2016, the code does not address the effect of material properties of masonry and structural configuration on the R.

Most of the buildings constructed in Nepal are unreinforced masonry. Since Nepal lies in the seismically active zone, it has a long history of earthquakes, and people living here have suffered a lot. It is necessary to build an earthquake-resistant structure.Hence, design plays a significant role in the performance of such masonry buildings taking an appropriate R in the seismic coefficient method. For example, IS 1893: 2016 gives the value of response reduction factor, R, for lateral load resisting system. For unreinforced masonry response reduction factor is 1.5, reinforced with horizontal RC band is two and reinforced with horizontal RC band, and vertical bars at corners of the rooms and jambs of the opening is 2.5. However, the code does not categories the response reduction factor for masonry structure having different mechanical properties. It is essential to determine the Rfor masonry buildings having different mechanical properties, which can be finally used for the design of the masonry

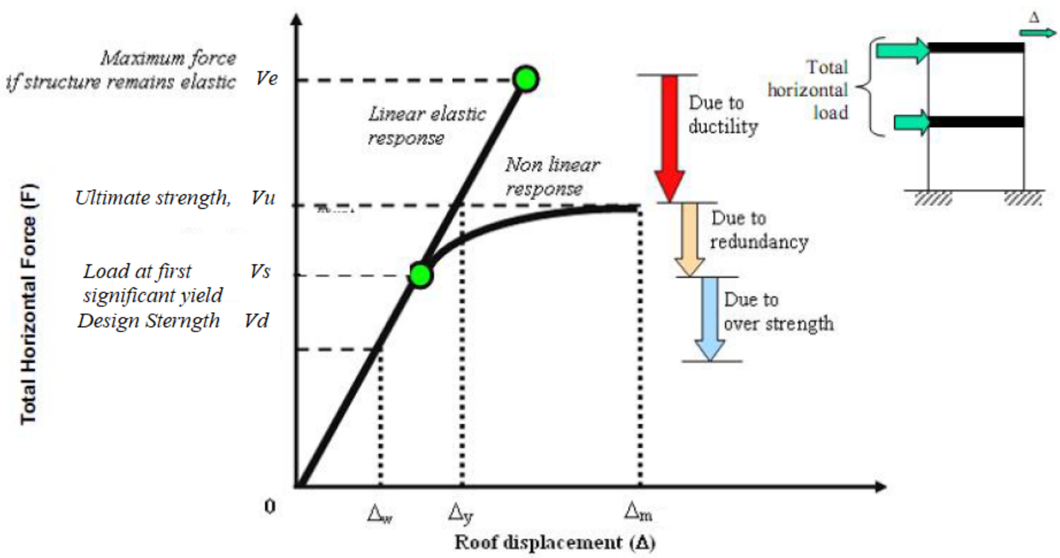

Fig.1. Concept of the response reduction factor (**Tamboliand Amin, 2015**)

structures. Hence, this paper aims to investigate the response reduction factor, 'R' for the different types of masonry buildings.

Development of a model

The primary aim of this work is todetermine the R for masonry buildings of different construction materials. An analytical model is prepared, material properties are assigned, analysis of the model is carried out, and results are interpreted carefully to achieve the study's objectives.

Response Reduction Factor (R)

The response reduction factor represents the ratio of the maximum lateral force, Ve, which would develop in a structure, responding entirely linear elastic under the specified ground motion, to the lateral force, Vd, which has been designed to withstand. Response reduction factor R is expressed by Equation 1.

$$R = Ve/ Vd \qquad (1)$$

The concept of R is shown in Figure 1.

According to ATC-19 (ATC, 1995), R is the product of three factors that accounted for reserve strength, ductility, and redundancy factor, as shown in Equation 2.

$$R= Rs.R\mu.R_R \qquad (2)$$

Where,

Rs stands for overstrength and calculated to be equal to the load at ultimate strength (Vu) divided by the design strength (Vd).

Rμ stands for the ductility reduction factor and calculated as a maximum force if the structure remains elastic (Ve) divided by the ultimate strength (Vu).

R_R stands for the redundancy factor calculated to be equal to the ultimate strength (Vu) divided by load at the first significant yield (Vs).

The overstrength factor is a measure of the additional strength a structure has beyond its design strength. The extra strength exhibited by structures is due to various reasons, including sequential yielding of critical points, a factor of safety considered for the materials, load combinations found for design, member size, and ductile detailing(Keerthi and Philip 2017). The ductility enables structures to undergo large deformations, resulting in the dissipation of a large amount of energy before the structure's collapse occurs. The ductility reduction factor is a measure of the nonlinear global response of a structure.

It is a function of the structure's characteristics, including ductility, damping, and fundamental period of vibration, as well as the features of earthquake ground motion.

More redundancy in the structure leads to an increased level of energy dissipation and strength. In a non-redundant system, the failure of a member is equivalent to the collapse of the entire structure. Thus, the system's reliability is a function of the system's redundancy, meaning that safety depends on whether the system is redundant or non-redundant. From the studies conducted by Mondal *et al.,* (2013), redundancy factors can be assumed as unity following the ASCE7 (2016) guidelines.

Modeling of Masonry Buildings

The proposed modeling of the masonry building is based on the use of the Equivalent Frame method. The SAP 2000 v. 20 package allows the user to account for the nonlinear mechanical behavior of the material by introducing the plastic hinge with lumped plasticity in the equivalent frame.

The plastic hinges were used in static push over (SPO) analyses since they allow the user to accurately follow the structural performance beyond the elastic limit at each step of the incremental analysis. The mechanical properties of these nonlinear elements were defined based on the possible failure mechanisms of masonry macro-elements. The adopted modeling is described in the following sections. 3

If the geometry of the openings (doors and windows) is sufficiently regular, it is possible to idealize the multistory masonry wall to an equivalent frame made by pier element, spandrel beam element, and joint element as shown in Figure 2.

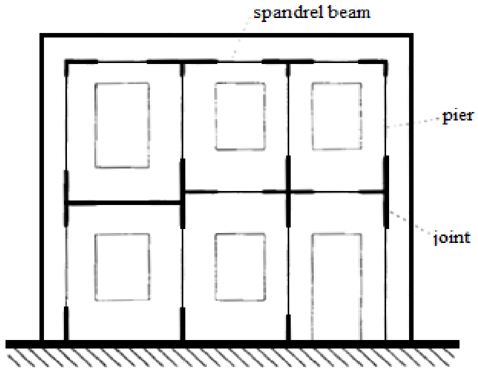

Fig. 2. Equivalent frame idealization of a multistory wall with an opening, Magenes (1998)

Modeling of nonlinear behavior for pushover analysis follows the standard force-displacement curve that can be implemented in the SAP 2000 plastic hinges.The masonry piers were modeled as elastoplastic with final brittle failure by introducing two' rocking hinges' at the end of the deformable parts and one' shear hinge' at mid-height. A rigid-perfectly plastic behavior with final brittle failure was assumed for all these plastic hinges. Plastic hinges are considered based on the works of Plastcier et al., (2007).

The force-displacement curve has been idealized as follows.The nonlinear force-displacement relationship between base shear and displacement of the control node shall be replaced with an idealized relationship to calculate the sufficient lateral stiffness, Ke, and effective yields. This relationship shall be bilinear, with initial slope Ke and post-yield slope α. Line segments on the idealized force-displacement curve shall be located using an iterative graphical procedure that approximately balances the area above and below the curve. The sufficient lateral stiffnessshall be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. The post-yield slope, α, shall be determined by a line segment that passes through the actual curve at the calculated target displacement. The effective yield strength shall not be taken as more significant than the maximum base shear force at any point along the actual curve.

To verify the reliability of the proposed modeling, a validation of the model wall of stone masonry buildings with mud mortar that was previously analyzed in the 'Catania Project' ismodeled with SAP 2000 v. 20, which was modeled by Plasticier *et al.*, 2007 in SAP 2000 v10. The 'Catania Project'in Italy was an extensive nationwide research project focused on the seismic performance of existing masonry buildings. In the project, some laboratory and in-situ tests were performed to characterize the masonry's mechanical properties. The University of Pavia used the SAM (Simplified Analysis of Masonry buildings) code, which is considered as an essential reference for this work. The code is based on the equivalent frame modeling and was previously validated on several experimental tests providing satisfactory results as conducted by Manganese *et al.,* (1997). The wall used for validation of the model is shown in Figure 3. The model of the wall in SAP 2000 V 20.0.0 is shown in Figure 4.

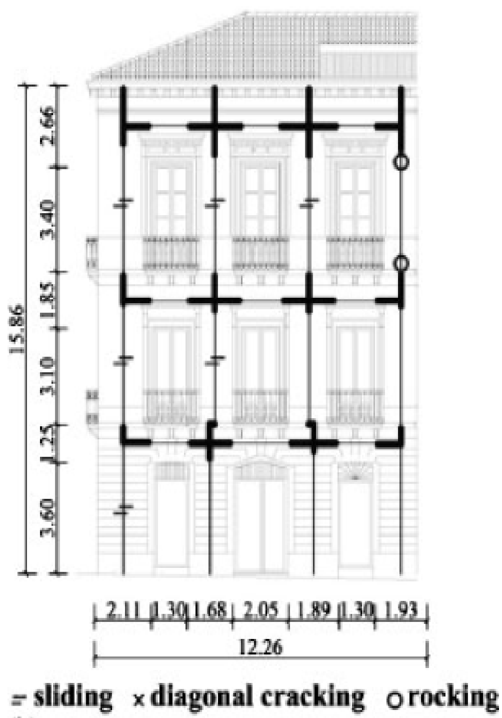

Fig. 3. Wall used for validation of the model (Plasticier et al., (2007)

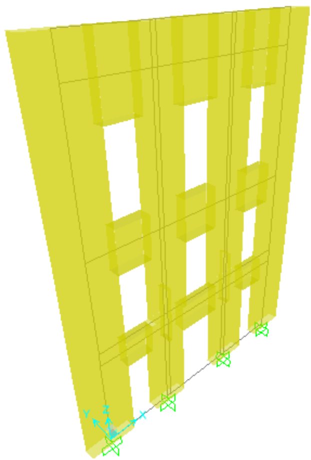

Fig. 4. Modeling of the wall in SAP 2000 v 20.0.0

The mechanical properties used in the analyses were Young's modulus=1500 N/mm^2, shear modulus =250 N/mm^2, unit weight=1900 kg/m^3, design compression strength=2.4 N/mm^2, design shear strength with no vertical stress =0.2 N/mm^2, and friction coefficient =0.5.

The hinge strengths, for mean vertical stress ($\boldsymbol{\sigma}_o$), of the wall are calculated using considering only gravity loads and no lateral loads. The outcomes are compared with results obtained from different researchers, especially Plasticier *et al.,* (2007), as shown in Figure 5. Table 1 shows the comparison of results with previous works. The pushover curve and failure mechanism are shown in Figure 6 and Figure 7.

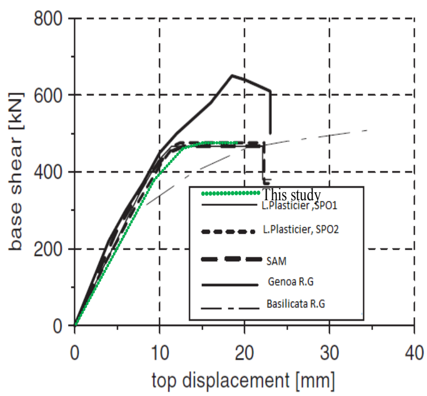

Fig. 5. Pushover curve of wall obtained by the various researchers

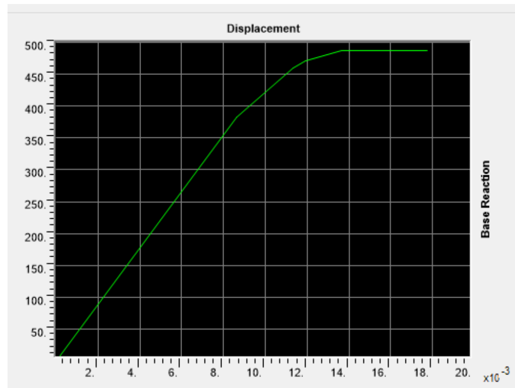

Fig. 6. Push over curve from SAP 2000 v20.0.0

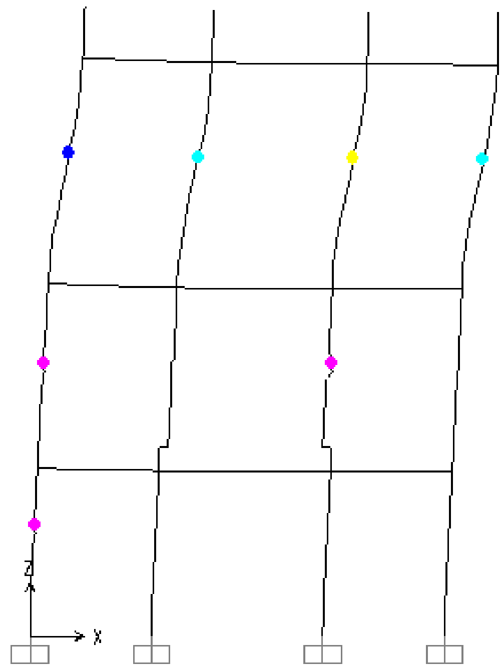

Fig. 7. The failure mechanism of the wall

It shows that the analysis out come is close to the results obtained by Plasticier *et al.,* (2007), having only a 4.1% deviation from SPO 2. The obtained pushover curve (Figure 6) and base shear (486 kN) is in quite an agreement with the pushover curve obtained by other researchers. It implies that this modeling technique can be used to study other cases as well.

Table 1. Comparison of result with the resultobtained by the differentresearcher

Research Group	Base Shear(KN)	Error(%)
Genoa research group	617	+32.1
Basilicata research group	503	+7.7
L.Plasticier, SPO 1	476	+1.9
L.Plasticier SPO 2	467	+0
This study	486	+4.1%

Application of the model in analysis of buildings

A typical one story one bay in both directions masonry building is selected as case study building. The story height of the building is kept 2.85 m and plan, and elevation of the building is given in Figure 8 and Figure 9.

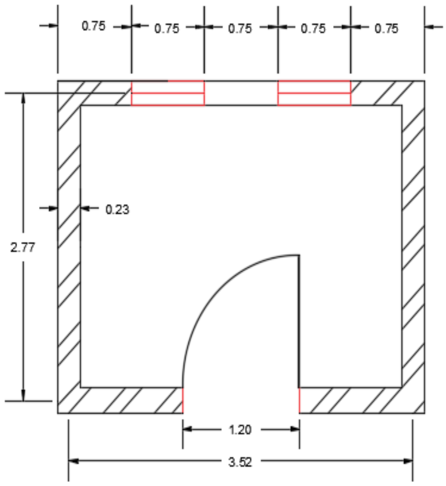

Fig.8. Plan ofthe building (Dimensions are in m)

The definition of the mechanical properties of a masonry building is according to Euro code 6. The characteristic compressive strength of the masonry can be obtained from the normalized mean compressive strength of the unit and the compressive strength of the mortar.

The estimation of the modulus of elasticity of the masonry is based on the approximation presented in the Swiss Society of Engineers and Architects (SIA) structural design code SIA 266. Mechanical Properties of masonry building are taken from previous studies, as presented in Table 2.

Following the procedures explained above, pushover analysis of one bay one-storey building is carried out for material brick in cement, brick in mud, CSEB in cement, and stone in mud mortar.

Results And Discussion

The representative masonry building models have been analyzed using SAP 2000 v20. As a result, pushover curves, ultimate displacement, and ultimate load capacity are obtained from static nonlinear analysis (pushover analysis) for the different mechanical properties of the masonry

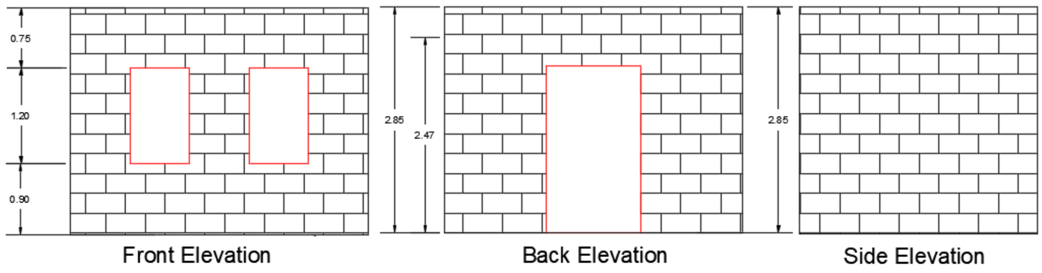

Fig. 9. Elevation of the building (Dimensionsarein m)

Table 2. Mechanical Properties of Masonry

Description	Brick in cement (Shrestha, 2005)	Brick in mud (Parajuli, 2012)	CSEB in Cement (Neupane, 2014)	Stone in mud (Fernando, 2008)
Modulus of Elasticity (MPa)	899.62	509.00	1,213.00	305.00
Shear Modulus (MPa)	352.80	204.00	485.00	152.00
Specific weight (kN/m^3)	19.00	17.30	19.00	17.60
Compressive Strength (MPa)	1.89	1.82	2.71	0.43
Shear strength (MPa)	0.68	0.15	0.29	0.23
Frictional Coefficient	0.50	0.50	0.50	0.50

buildings.Figure 10 shows the relation between the base shear and the top displacement of the building models.

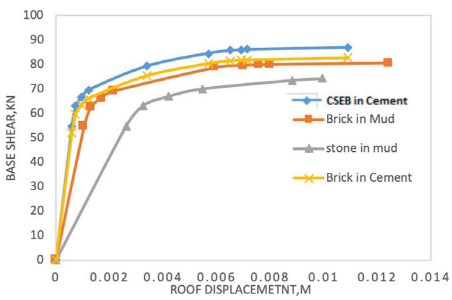

Fig. 10. Base shear vs. displacement

For the models, ultimate strength and ultimate displacement of brick in cement masonry are 71.00 kN and 1.69 mm, brick in mud masonry is 68.49 kN and 2.04 mm, CSEB in cement masonry is 74.34 kN and 4.33 mm. Random rubble stone in mud masonry is 71.8 kN and 1.24 mm, respectively. Among four masonry properties, CSEB in a cement block hasthe highest seismic performance, and random rubble stone in mud has the lowest seismic performance.

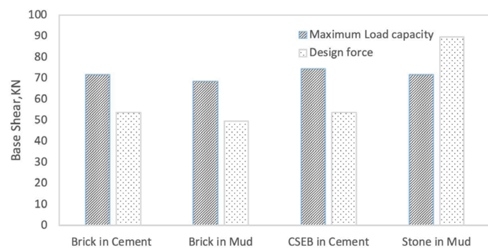

Fig. 11. Ultimate strength and design strength from nonlinear analysis

The histograms in Figure 11 show that the ultimate strength of brick in cement, brick in mud, and SEB incement masonry is higher than that of design strength. Still, random rubble stone mud has lesser ultimate strength than design strength, indicating that they have additional strength beyond design strength, but random rubble stone in mud masonry does not have added strength beyond the design strength.

Over strength factor of three masonry buildings

Table 3. Response Reduction Factor for Different Masonry Buildings

SN.	Masonry Buildings	Overstrength factor	Ductility	Ductility reduction factor	Response reduction factor
1	Brick in cement	1.38	2.08	1.78	2.45
2	Brick in Mud	1.37	1.68	1.54	2.10
3	CSEB in Cement	1.38	3.21	2.33	3.22
4	Random rubble stone in mud	0.82	2.03	1.38	1.13

(Brick in cement, brick in mud, and CSEB in cement) are found to be similar, which indicates additional strength beyond design strength is identical, as shown in Table 3. But the overstrength factor for random rubble stone in mud masonry is found to be very low, indicating that structure does not have any additional strength beyond design strength. Also, the ductility of masonry buildings is small, showing low inelastic deformation during failure. According to IS 1893-2016, R for an unreinforced masonry building is 1.5. From this study, it is clear that the R given in the code is sufficient for unreinforced brick in cement and CSEB in cement. However, the R-value for random rubble stone in mud masonry is not enough due to the low overstrength factor and ductility reduction factor.

Conclusion

The results on the response reduction factor using nonlinear pushover analysishave been drawn as follows:

Response reduction factor for all masonry building is found more than 1.5 for unreinforced brick in cement, brick inmud, and CSEB in cement. Hence,the value of R provided by IS: 1893-2016 is sufficient for masonry structure. However, in the case of random rubble stone in mud masonry, it does not meet the requirement of over strength due to low lateral load caring capacity. The R-value given in IS: 1893-2016 for unreinforced masonry is not recommended for random rubble stone masonry buildings in mud mortar.

References

Applied Technology Council (ATC),. Seismic Response Modification Factors (ATC-19), Redwood City, California, 1995.

ASCE 7, (2016). Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers. The USA.

Bilham, R., Bodin P., and Jackson, M. (1995). Entertaining a Great Earthquake in Western Nepal: Historic Inactivity and Geodetic Tests for the Present State of Strain. *J. Nepal Geol. Soc*. **11**(1): 73–78.

Chitrakar, G., and Pandey, M. (1986). Historical Earthquakes of Nepal. Bulletin of Geological Society, 4:7-8.

Eurocode 6. Design of Masonry Structures. British Standards Institutio., European Committee for Standardization.t. 1, no. 1., 2005.

Fernando, F. S., Manuel, F. C., Válter, J. G. and Paulina, F. Experimental Research on Rubble Random rubble stone mud masonry Walls. Portugal, 2008.

IS 1893, Criteria for earthquake resistant design of structures, part 1 General provisions, and buildings. Sixth revision, New Delhi: Bureau of Indian Standards, 2016.

Magenes, G., and Calvi, G.M. (1997). "The in-plane seismic response of brick masonry walls" Earthq. Engin. And Struct. Dyn., Vol. 26, pp. 1091-1112.

Magenes, G., and Fontana, A. (1998), "Simplified nonlinear seismic analysis of masonry buildings," Proc. of the British Masonry Society, Vol. 8, pp. 190-195.

Mondal, A., Ghosh, S., and Reddy, GR (2013), Performance-based evaluation of the response reduction factor for ductile RC frames. Engineering Structures 56, 1808–1819.

Parajuli, H. R., (2012), "Determination of mechanical properties of the Kathmandu World Heritage brick masonry buildings."

Pasticier, L.; Amadio, C. & Fragiacomo, M., Non-Linear Seismic Analysis and Vulnerability Evaluation of a Masonry Building by Means of the SAP2000 ® v.10 Code, 2007.

Shrestha, L., (2005), "Seismic Capacity of Brick Masonry Residential Building," Institute of Engineering, Pulchowk campus.

SIA 266, Masonry, Swiss Society of Engineers and Architects (SIA), Zürich, Switzerland, 2003.

Thapa, N., (1988), Bhadau Panch Ko Bhu Kampa (in Nepali), Central Disaster Relief Committee, Nepal78=-=-098765YTE MJ, L';/, KMJNBGX

Keerthi, S. and Philip, V. (2017), evaluation of response reduction factor based on redundancy for high rise buildings. International Research Journal of Engineering and Technology (IRJET), Vol. 4(5), pp. 3161– 3165.

Tamboli, K and Amin, J. A. (2015), Evaluation of Response Reduction Factor and Ductility Factor of RC Braced Frame. Journal of materials and engineering structures 2, 120–129.