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MULTIPLE ANALYTICAL APPROACHES ON SEISMIC RETROFIT DESIGN OF UNREINFORCED MASONRY BUILDINGS IN NEPAL

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Abstract

This study presents the analytical challenges and different approaches used to overcome those challenges on seismic retrofit design of unreinforced masonry buildings in Nepal. Seismic analysis of the building has been carried out using different finite element modeling techniques. Two main techniques of numerical analysis used in the study were static analysis in SAP2000 and nonlinear pushover analysis in TREMURI. The assessment of the building was conducted based on visual observations, different non-destructive and in-situ testing. The analyzed building is two story unreinforced brick masonry with cement mortar. The material properties for finite element modeling are extracted from material testing of representative structural members. The result obtained from incremental nonlinear pushover analysis were taken as a reference to validation the static approach. The finite element modeling has been developed in SAP2000 in which wall and slab are modeled as shell element and retrofitting design is carried out. The SAP2000 obtained analysis result is validated with TREMURI program through nonlinear seismic analysis of the same building. This validated design has been implemented in retrofitting of typical unreinforced masonry building in Nepal. It is found that addition of internal walls in few location of corresponding thickness is effective to improve seismic performance of building. Reinforced concrete Splint and bandage on outer walls and inner walls were provided for this building. It is observed that time period decreases after retrofitting. The base shear increases, displacement demand capacity ratios decreases and ductility increases after retrofitting. Similarly, this building sustained 0.55g after retrofit whereas it was damaged at 0.34g before retrofit. These results conclude that the building is safe for designed peak ground acceleration. It is found that SAP2000 based linear retrofitting design gives more conservative result than TREMURI based nonlinear pushover analysis.

Keywords:Unreinforced masonry building; finite element modeling; Pushover analysis; Splint and bandage; Seismic retrofit

1. Introduction

Nepal is located within the Himalayan mountain range, a product of the continental collision of the Eurasian and Indian plates, initiated about 40–55 million years ago. The collision was followed by the introduction of the Indian plate underneath Tibet, which continues today at an estimated rate of about 3 cm/year. Numerous earthquakes have occurred in this region, including four major earthquakes of magnitude greater than M8 within the last 100 years. The April 25, 2015 Gorkha Earthquake of Magnitude 7.8 in Nepal damaged about 700,000 buildings. The main typologies of buildings in the affected area are stone masonry with mud mortar, some buildings with stone and brick masonry with cement/sand mortar and few reinforced concrete buildings with masonry infill. Fig.1 shows the building typology distribution in 31 districts, which were affected by the April 25, 2015 earthquake in Nepal. It shows that about 58% of the buildings are mud based masonry, i.e. stone in mud, adobe or brick in mud; 21% are cement based masonry either stone with cement-sand mortar or brick with cement-sand mortar and about 15% are reinforced concrete with masonry infill. There are other types of buildings that are only about 6%.

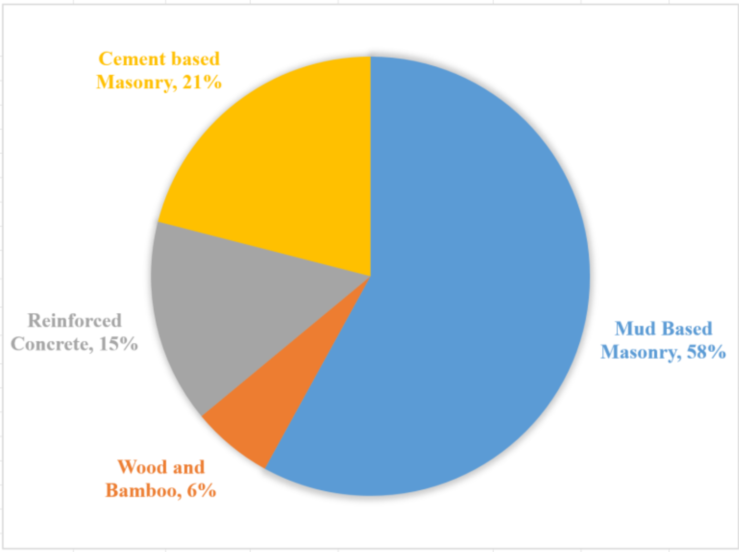


Fig. 1 – Overall building typology distribution in the affected area of April 25,Nepal earthquake (CBS 2011)

Analytical modeling of masonry structures is a complicated task. Gambarotta and Lagomarsino [2] developed a finite element approach for the masonry where brick units and mortar joints are considered separately. A model has been defined in which the brick units are modeled through four or eight isoparametric elements with four-nodes while the mortar joints are modeled by interface element with four-nodes. This model seems to be too burdensome in analyzing full-scale masonry walls. In the work of Buton and Mayes [3], an analytical model of the masonry structure is developed, which predicts the out-of-plane seismic behavior of reinforced masonry walls. The model consists of a number of inelastic beam column elements arranged vertically to represent a particular wall. Each element is a series combination of an elastic flexure line element with an inelastic hinge at each end. In general, two failure modes of masonry wall elements are reported in the literatures [4], which are flexure failure and shear failure. In the flexural failure, crushing of the compressed zone occurs after tensile cracks on the tension side have reduced the effective cross-sectional area of walls. In the case of shear failure, diagonal cracks occur in the wall.

Recently, different numerical modelling approaches are developed for analysis of unreinforced masonry walls and buildings by application of different numerical tools such as Finite Element Method or Discrete Element method. Modelling approaches that are recommended for unreinforced masonry walls could be categorized into three main groups; micro and simplified micro modelling, macro modelling and equivalent frame modelling. In micro modelling approach the masonry units, mortar joints and the unit-mortar bond are modelled separately with appropriate constitutive models for each part. In simplified micro modelling approach, each masonry unit and half of the mortar joints around it are considered as equivalent blocks that are jointed together with appropriate interface models. Various researchers have proposed micro models for analysis of masonry walls [5-6]. Although the results of micro modelling approaches have high accuracy, because of complexity and major computational cost, they are not suitable for large scale modelling and analysis of masonry buildings with several walls and piers. In this regard, macro modelling approach with less computational cost has been introduced. In this method, masonry is assumed as a homogenized isotropic or anisotropic material and the equivalent mechanical properties of masonry assemblage are used in analyses. The results obtained by this method have less precision than the results of micro modelling approaches. Models developed by Lourenco et al. (1997) and Lourenco (2000) are some outstanding examples of this modelling approach [7-8].

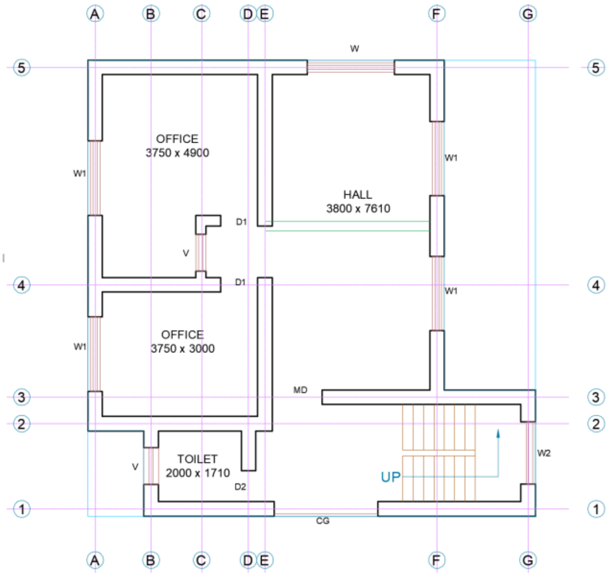
The other modelling method is known as equivalent frame method. In this method, one dimensional elements such as frame or truss elements are utilized for simulation and analysis of URM Buildings. Because of simplicity and low computational cost, this method is attractive for nonlinear static and dynamic analysis of unreinforced masonry buildings. Therefore, considerable investigations have been carried out for development of simple equivalent frame or truss models for analysis of URM buildings. Roca et al. presented an equivalent frame model for nonlinear analysis of URM buildings and historical monuments. [9]. Pasticier et al.[10] utilized software SAP2000 for nonlinear analysis and vulnerability assessment of masonry buildings. In another study, Chen et al. introduced an equivalent frame model with appropriate plastic hinges for nonlinear analysis of URM buildings [11]. M.A. Najafgholipour [12]uses simple equivalent truss model for nonlinear static analysis of URM walls with sliding shear failure as the governing in-plane failure mode that Mohr-Coulomb constitutive model is used to determine their ultimate in-plane strength. Recently, Lagomarsino et al. presented a model in software Tremuri for nonlinear analysis of URM buildings to evaluate their seismic performance [13]. In this “Equivalent frame” analysis, the elements comprise orthogonal discs, which may represent piers or spandrel beams spanning over openings, and are actually deformable masonry panels. Each disc possesses degrees of freedom at the corner points for connectivity with adjacent elements and for derivation of stiffness properties; any nonlinearity in the response refers to these deformable components. Rigid portions are the remaining regions that connect the deformable parts together.

This study presents the analytical challenge of seismic retrofit design of unreinforced masonry buildings. The finding is based on the case study of building located in Surkhet district of Nepal. Seismic analysis of the building has been carried out using a different finite element modeling techniques. Two main techniques of numerical analysis used in the study were static analysis in SAP2000 and nonlinear push over analysis in TREMURI. The assessment of the building was conducted based on visual observations, different non-destructive and in-situ testing were carried out at site and the detailed analysis of the building was performed. The analyzed building is two story unreinforced brick masonry with cement mortar. Field investigation shows that there is no tie beam at plinth level of building. The lateral load resisting system is the 14 inch thick masonry wall with cement mortar in first floor and 9 inch thick masonry wall with cement mortar in second story and stair case cover. There are no vertical reinforcements at junctions of walls and jambs of door/window opening, horizontal bands at sill and lintel and corner stitches which are key elements to resist lateral load of earthquakes. The material properties for finite element modeling are extracted from material testing of representative structural members. The result obtained from incremental nonlinear pushover analysis were taken as a reference and validation solution for the static approach. The finite element modeling has been developed in SAP2000 in which wall and slab are modeled as shell element and retrofitting design is carried out. This design parameter is validated with TREMURI program for the nonlinear seismic analysis of masonry building. This validated design has been implemented in retrofitting of typical unreinforced masonry building in Nepal.

2. Structural System of unreinforced masonry building

The case study building is two story unreinforced brick masonry with cement mortar that was constructed in 1996. The building configuration in both plan and vertical is regular. The storey height is 2.99m and plinth area is 98.615 m^2. The floor and roof structure is RCC 5 inch slab. There is no possibility of landslide and rock fall in the site. The site investigation shows that there is no tie beam at plinth level. The lateral load resisting system is the 14 inch thick masonry wall with cement mortar in ground floor and 9 inch thick masonry wall with cement mortar in first story and stair case cover. There are no vertical reinforcements at junctions of walls and jambs of door/window opening, horizontal bands at sill and lintel and corner stitches, which are key elements to resist lateral load of earthquakes. The photograph and ground floor plan of building is shown in Fig. (2).

(a) Photograph of building



(b) Ground floor plan of building

Fig. 2 – Building configuration

3. Field Investigation of Building

Reliable information on shear resistance is needed when performing retrofits and seismic upgrades of masonry buildings. The in-situ shear test is also known as the push test that provides a direct measurement of the shear resistance of mortar joints in masonry. The test is suitable for masonry that has relatively strong units and weak mortar so that shear cracks form in the typical stair step pattern along mortar joints and the units remain un-cracked. Two test locations were selected based on external locations. The test locations were prepared by removing the brick, including the mortar on one side of the brick to be tested. The head joint on the opposite side of the brick to be tested was also removed. This was done with caution that the mortar joint above or below the brick to be tested is not damaged. The hydraulic ram was inserted in the space where the brick was removed. A steel loading block was placed between the ram and the brick to be tested so that the ram will distribute its load over the end face of the brick. The dial gauge was inserted in the space. The test is shown in Fig.(3). The brick was then loaded with the ram until the first indication of cracking or movement of the brick. The ram force and associated deflection on the dial gage were recorded. From the observation, final corrected shear strength of brick masonry is obtained as 0.26 N/mm2.

Fig. 3- Conducting In-situ Shear Test on Wall

Fig.4- Foundation Investigation

To explore the foundation details of the building, excavation was carried out. The details of the foundations is shown in Fig. 4. The building has footings made of brick masonry. Total depth of footing is 1325 mm and the width is around 900 mm. There is no plinth beam in the walls.

4. Seismic analysis and retrofit design of Building

The analysis and design of the building is carried using prevalent design philosophy for masonry building structures. The seismic analysis is a part of the detailed evaluation of an existing building. The steps involve are developing a computational model of the building, applying the external forces, calculating the internal forces in the members of the building, identifying deformations and capacity of the members and building, and finally interpreting the results. The structural analysis is carried out with the help of the available drawings and Sap2000 a structural analysis and design software. A three dimensional model has been prepared. Slab and wall are modeled as shell element. The material properties of existing unreinforced masonry and concrete and rebar are shown in Table 1 and 2 respectively. The seismic load is calculated as shown in Table 3 using seismic coefficient method. The seismic performance assessment was based on design value of peak ground acceleration (a_g) 0.34 g for 300 years return period.

Table 1 – Existing Unreinforced Masonry (Brick in Cement Mortar)

S.N.	Properties	Value	Unit
1	Modulus of Elasticity	2569	MPa
2	Poisson's Ratio	0.25	
3	Shear Modulus	1028	MPa
4	Unit Weight	19	KN/m^3
5	Characteristic compressive strength of masonry	12.67	MPa
6	Average compressive strength of masonry	6.67	MPa
7	Shear strength under zero compressive stress	0.26	MPa
8	Upper limit value of the shear resistance	2.20	MPa
9	Tensile strength	0.04	MPa
10	Confidence factor	1.35	
11	Material factor	1	
12	Shear drift	0.004	
13	Bending drift	0.008	

Table 2 – Properties of existing concrete and Rebar

Material	Properties	Value	Unit
Concrete	Characteristic compressive strength of concrete	15	MPa
	Modulus of Elasticity	19365	MPa
	Poisson's Ratio	0.2	
	Shear Modulus	8069	MPa
	Unit Weight	25	KN/m^3
Rebar	Yield strength	415	MPa
	Modulus of Elasticity	2×10^5	MPa
	Poisson's Ratio	0.3	
	Shear Modulus	76920	MPa
	Unit Weight	78.5	KN/m^3

Table 3 – Modelling parameter of Building (NBC 105, [14])

Seismic Zone factor	Z	1	
Importance factor	I	1.0	
Structural performance factor	K	2.500	
Height of the building	h	8.035	m
Dimension of the building Along X	Dx	10.800	m
Dimension of the building Along Y	Dy	11.010	m
Time period of the building along X,	$T_x = 0.09h/\sqrt{D_x}$	0.220	sec
Time period of the building along Y	$T_y = 0.09h/\sqrt{D_y}$	0.218	sec
Soil type	Medium type	Type II	
Design Horizontal Seismic Coefficient along X	Cdx = CZIK	0.227	
Design Horizontal Seismic Coefficient along Y	Cdy = CZIK	0.227	

The design load combinations including earthquake for the working stress method according to NBC 105:1994 [14] are: DL + LL, DL + LL + EQ,0.7 DL + EQ. The finite element model prepared in SAP2000 is shown in Fig.5. It is found from the analysis that time period for Transverse and longitudinal directions are 0.095s and 0.104s respectively. Similarly, the observed base shear is 748.6kN. Initially, the existing building is modeled and in-plane stresses along with out-of-plane moments are studied. The in plane stress of Compressive/Tensile stress (In plane stress (S22)) of Grid A-A is shown in Fig. 6. Retrofitting of existing structures with insufficient seismic resistance accounts for a major portion of the total cost of hazard mitigation. Thus, it is of critical importance that the structures that need seismic retrofitting are identified correctly, and an optimal retrofitting is conducted in a cost effective way. The size of walls are sufficient as per codal requirement, but there are no tensile elements such as vertical bars and bands. Therefore, to enhanced the tensile capacity of buildings minimum interventions such as RC Splint & Bandage were provided in the critical locations.

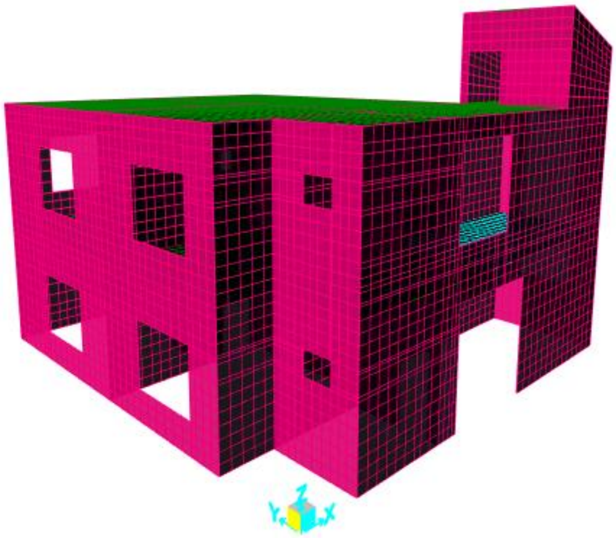


Fig.5- Finite element model in SAP2000

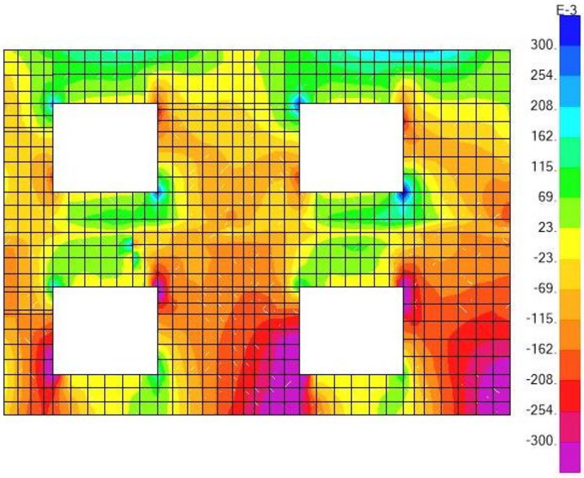


Fig.6- Compressive/Tensile stress (In plane stress (S22)) (Grid A-A)

The importance of vertical and horizontal reinforced concrete splint and bandage is namely to improve the confinement of the entire structure, to increase resistance and ductility and to reduce the risk of wall collapse. The splint and bandage has been designed from stress obtained from SAP2000 results. The designed retrofitting work is shown in Table 4. It is observed that addition of internal walls in few location of corresponding thickness is effective to improve seismic performance of building. Reinforced concrete Splint and bandage on outer walls and inner walls is worked out for this building. This approach improves the seismic response of existing structure as required by earthquake design consideration.

Table 4 – Conclusion and recommendations for retrofitting work

Modifications	Demolition and addition of new walls in few location of corresponding thickness to improve load path.
Retrofitting of walls	Splint with (3 Nos-8mm Ø bar @ 300mm width at opening and 3Nos-10 mm Ø bar+ 2Nos-8 mm Ø bar @600mm width at corner and 3Nos-10 mm Ø bar@900mm width at T junction-vertical) in outer and inners walls and Bandage with (2Nos-8 mm Ø bar @ 300mm width-Horizontal) in outer and inner walls
Foundation	Strengthening of wall footing by adding plinth beam in outer and inner wall with size 300mm x 225mm (4 Nos 12mm Ø bar) and anchoring the RC beam to the wall through 12mm Ø anchorage rod.

5. Design parameter validation with TREMURI program

TREMURI software [13] is used for the non-linear pushover analysis (NLPO) in this work. It uses the method FME- Frame by Macro Element, specially studied for masonry structures. TREMURI is capable of reproducing three in-plane failure modes: rocking, shear cracking, and shear sliding. For linear design the stresses induced in the structural elements due to the applied loads on the structure shall be within the permissible/allowable capacity of the materials, whereas, for non-linear pushover analysis, the acceptable drift limit in shear is 0.004 and that for flexure is 0.008. The retrofit design carried out using SAP2000 as explained above is validated with TREMURI software. A series of 24 analyses for each masonry building was performed. All types of horizontal floors are assumed rigid in their planes. Due to software capabilities, separate analyses by considering slide and diagonal cracks were performed. The two models performed associated with the 24 analyses takes into account the bending mechanism and shear mechanism with diagonal cracks and bending mechanism and shear mechanism with shear failure. It found that bending mechanism and shear mechanism with shear failure is critical in this case. So whole analysis was done based on the bending mechanism and shear mechanism with shear failure is critical in this case. Horizontal forces were assumed proportional to the product of storey masses and the static force.

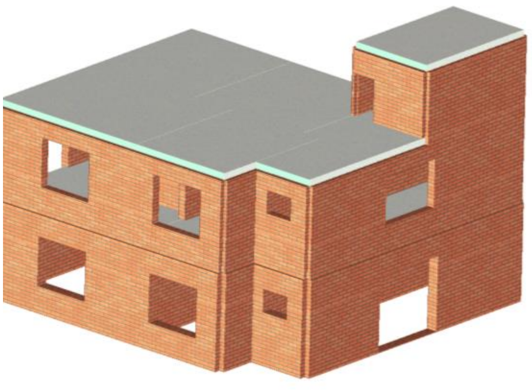
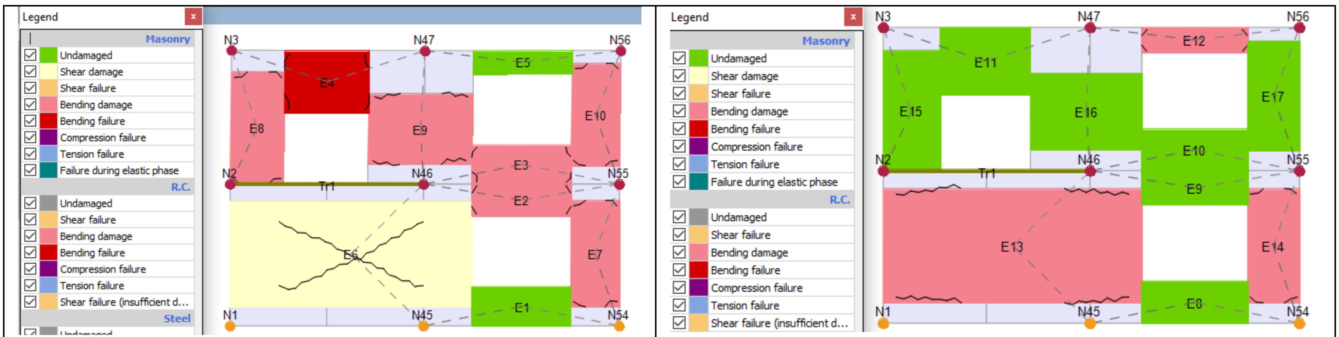


Fig.7- Model of the building in TREMURI Software

Three cases are considered; (1) As built (unreinforced) performance in plane NLPO (2) Retrofit performance in plane NLPO (3) Retrofit performance in plane NLPO. The first case deals with TREMURI results for structure before retrofitting. The same modeling parameters are used as in SAP2000. Similarly, the second case considers the retrofit demand as per TREMURI. Splint with (1 No-8mm Ø bar @ 200mm width at opening and 2Nos-8 mm Ø bar@200mm width at corner and and 4Nos-8 mm Ø bar@600mm width at T junction-vertical) in outer and inners walls and Bandage with (2Nos-8 mm Ø bar @ 300mm width-Horizontal) in outer and inner walls are considered in this analysis. Retrofitted design from SAP2000 is validated with TREMURI in third case. TREMURI based design has not dealt with out of plane bhavior. It has been studied with SAP2000. The model developed in TREMURI program is shown in Fig. 7. The program converts surface model to equivalent frame system. Based on the pushover analysis software shows pushover curves and calculates the displacement demand. Competent analyses for this building, corresponding displacement demand and maximal displacement, which the building can withstand for the Life safety (LS), were chosen. Criterion for determining competent analysis was the maximum ratio between the displacement demand and maximal displacement and also the minimum limit-state peak ground acceleration. The analysis has been carried out for each three cases.



(a) Before retrofitting (case 1)

(a) After retrofitting (case 3)

Fig.8- Progressive failure of wall (Grid 5-5) before and after retrofitting (TREMURI Software)

Progressive failure of wall (Grid 5-5) before (Case 1) and after retrofitting (Case 3) is shown in Fig.8. It shows the wall's deformation; the elements color indicates the type of the identified damage immediately through the color legend. It is found that the critical failure mode before retrofitting is shear and bending failure. It can be noticed that piers and spandrel beams are the most vulnerable sections of the building. The pushover analysis obtained after the global analysis in TREMURI software provide capacity curve which are shown in Figs. 9 and 10 for X and Y direction respectively. Similarly, sensitivity analysis result and status of building in different cases are presented in Table 5 and 6 respectively.

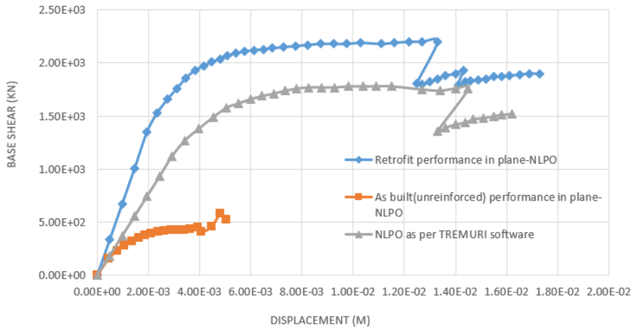


Fig.9- Pushover curve in X direction

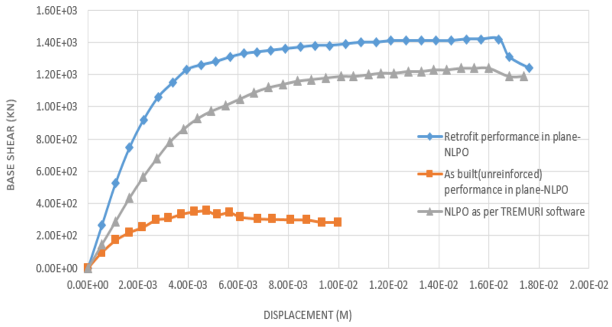


Fig.10- Pushover curve in Y direction

Table 5 – Sensitivity analysis

Analysis/ performance		Period (s)		Base Shear (kN)		Disp. Ductility (mm)		Disp. Capacity (mm)		Disp. Demand (mm)	
		Trans.	Long.	Trans.	Long.	Trans.	Long.	Trans.	Long.	Trans.	Long.
As built (unreinforced) performance in plane -NLPO	1	0.193	0.244	351.2	285.8	1.730	3.672	3.79	7.47	25.1	29.9
NLPO as per TREMURI software	2	0.133	0.177	1107.0	1047.2	5.028	4.879	16.4	19.1	8.42	14.2
Retrofit performance in plane-NLPO	3	0.101	0.134	1396.0	1216.0	5.462	5.046	12.9	13.2	3.13	8.42

Table 6 – Status of building in different cases

Analysis/performance		Peak Ground Acceleration causing limit state of significant damage (α_g)		Design value of the Peak Ground Acceleration $(\alpha_{g,300\ years})$	$\alpha_{min}=\frac{\alpha_g}{\alpha_{g,300\ year}}$		Status	
		Trans.	Long.		Trans.	Long.		
As built (unreinforced) performance in plane -NLPO	1	0.096	0.107	0.340	0.283	0.316	Unsafe	Unsafe
NLPO as per TREMURI software	2	0.470	0.410	0.340	1.382	1.206	Safe	Safe
Retrofit performance in plane-NLPO	3	0.558	0.430	0.340	1.641	1.265	Safe	Safe

It is observed from Fig. 9, Fig. 10 and Table 5 that time period decreases after retrofitting. The base shear increases, displacement demand capacity ratios decreases and ductility increases after retrofitting. Similarly, Peak Ground Acceleration causing limit state of significant damage increases in both cases 2 and 3. These results conclude that the building is safe for designed PGA. Peak Ground Acceleration value causing limit state of significant damage in case 3 is greater than that in case 2 which shows that SAP2000 based linear retrofitting design gives more conservative result than TREMURI based NLPO.

(a) Excavation of the plinth beam for Splint

(b) Fixing of reinforced bar mesh Splint and bandage on outer walls of the building

Fig.11- Photograph during construction

The retrofitting design has been implemented. The photograph during construction is shown in Fig. 11. The total construction cost of building for the proposed retrofit scheme is equal to USD 10500.

6. Conclusion

Seismic analysis of the building has been carried out using finite element modeling with static analysis in SAP2000 and nonlinear push over analysis in TREMURI. The assessment of the building was conducted based on visual observations, different non-destructive and in-situ testing. The finite element modeling has been developed in SAP2000 and this design parameter is validated with TREMURI program for the nonlinear seismic analysis of masonry building. This validated design has been implemented in retrofitting of typical unreinforced masonry building. It is found that addition of internal walls in few location of corresponding thickness is effective to improve seismic performance of building. Reinforced concrete Splint and bandage on outer walls and inner walls is adopted for this building. The base shear increases, displacement demand capacity ratios decreases and ductility increases after retrofitting. Similarly, Peak Ground Acceleration causing limit state of significant damage increases after retrofitting. These results conclude that the building is safe for designed PGA. It has been concluded that SAP2000 based linear retrofitting design gives more conservative result than TREMURI based NLPO.

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8. Copyrights

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