Experiences on Assessment and Retrofitting of Reinforced Concrete Buildings with Masonry Infill in Kathmandu Valley

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SUMMARY:
Reinforced Concrete moment resisting frame with masonry infill is the most common building typology nowadays in the urban areas of Nepal. Various qualitative and quantitative assessment and experiences of recent past earthquakes indicate increased risk to this type of building whether non engineered or engineered. These structures lack strength and ductility and are deficient in configuration and structural system. This paper highlights on the seismic vulnerability assessment, different retrofitting methods implemented and lesson learnt. This study includes quantitative evaluation of seismic risk of this particular type of building and proposes practical methods to reduce it. Out of various available methods, wall jacketing and introduction of new shear walls are considered to be the most practically feasible and economically viable methods in Nepal. This paper also highlights the challenges in implementation of retrofitting scheme due to poor socio-economic condition of the country, lack of awareness among the public.

Keywords: Seismic vulnerability assessment, Reinforced concrete frame, infill, retrofitting, pushover analysis

1. INTRODUCTION

Reinforced concrete moment resisting frame with masonry infill is the most common building typology nowadays in the urban areas of Nepal. In Kathmandu Valley, there are many non-engineered and only few engineered reinforced concrete (RC) buildings. In most of the cases, these buildings are constructed by unskilled craftsman/masons and without involvement of civil or structural engineer. Most of these RC buildings are initially built up to three stories. However, afterwards floors are added without strengthening the frame structure. Furthermore, there is a tendency to extend the floors of upper stories beyond the floors of lower stories by constructing cantilever and then build the walls on that cantilever (JICA, 2002).

Reinforced concrete non-engineered frame buildings comprises of about 25 % of the existing building stock in the urban areas of Nepal. These category of the building are unsafe against large impending earthquake (Shrestha and Bothara, 2007).

2. SEISMIC VULNERABILITY ASSESSMENT

Studies on seismic vulnerability assessment of buildings in the Kathmandu valley revealed that most of the buildings are unsafe against large earthquakes (JICA, 2002). National Society for Earthquake Technology-Nepal (NSET) is actively involved in seismic assessment of buildings in Nepal. NSET has also developed a guideline for seismic vulnerability assessment of both public and private buildings (NSET, 2009). Seismic vulnerability assessment involves both qualitative and quantitative methods of assessment. Seismic vulnerability assessment is carried out using FEMA 310 and guidelines developed by NSET (NSET, 2009) and Indian Institute of Technology Kanpur (Rai, 2005).

2.1. Typical Characteristics of Reinforced Concrete Buildings in Kathmandu Valley

Major weaknesses revealed in this typology of buildings in Kathmandu valley are as follows (JICA, 2002):
(1) Insufficient concrete cover to re-bar
(2) Incorrect positions of re-bar's joints in columns and beams
(3) Inadequate lapping length of re-bars
(4) Incorrect detailing of hooks of hoops in columns and stirrups in beams; the angles of hooks are less than 135° and hook lengths are short
(5) Insufficient anchorage lengths of beam re-bars
(6) Poor quality of concrete construction

2.2. Case Study: Seismic vulnerability assessment of a Typical Building

NSET conducted seismic vulnerability assessment of this building on November 2010. The results of the assessment is described in the subsequent paragraphs.

2.2.1. Building Description
It is a three storey reinforced concrete frame building with masonry infill walls. The building is located in northern side of the Kathmandu Metropolitan city. This is a newly constructed engineered building. The plan is square in shape, however, the structural system is irregular. There are in total 16 numbers of columns. Sizes of columns varies from 225 mm X 300 mm to 225 mm X 450 mm. External periphery walls are constructed of full brick thickness (225 mm) and internal walls are either half brick thick (115 mm) or full brick thick (230 mm). The floor plans showing structural grids is shown in figure 1.

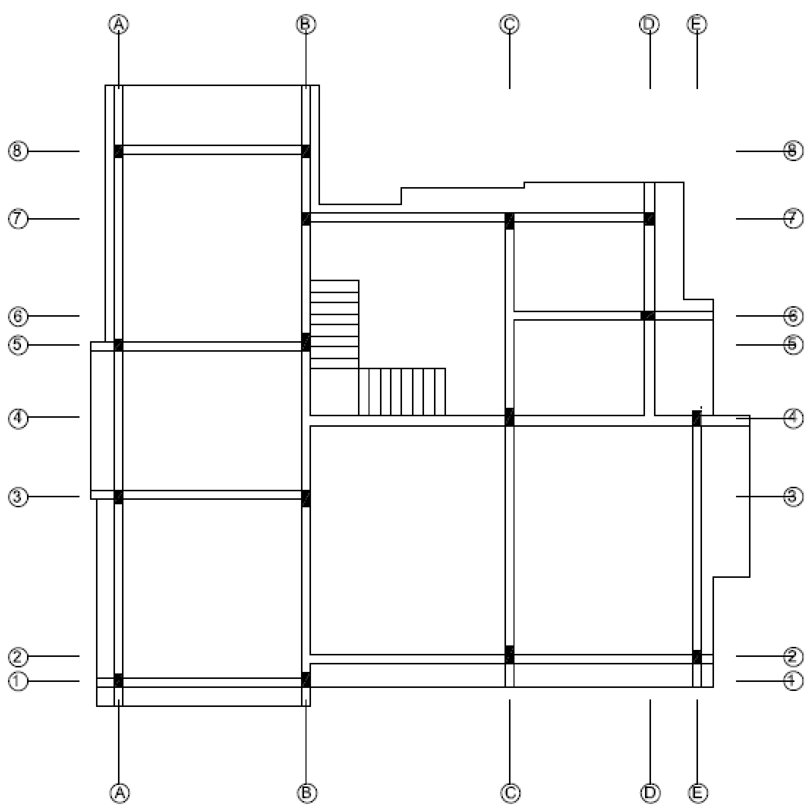

Figure 1. Plan of the building showing the structural grids

2.2.2. Deficiencies Identified

Major deficiencies identified in this building are as follows:

(1) There are three complete moment resisting frames in north-south direction whereas no complete moment resisting frames in east-west direction. This created load path discontinuity.

(2) All columns except D6 are oriented length in north-south direction. This creates larger stiffness in north-south direction only.

(3) Walls on the external periphery of the building are not placed on the frame centreline and are projected outwards. This also leads to load path discontinuity in the building.

(4) Spacing of ties in columns varies from 100 mm c/c to 150 mm c/c. This leads to non ductile detailing of reinforcements.

(5) There is a lack of wall to frame connection leading to possibility of out of plane collapse of walls.

2.2.3. Pushover Analysis of the existing Building

Pushover analysis is a process of evaluating the expected performance of the structure by estimating its strength and deformation demands in design earthquake and comparing these parameters with the available capacities at the performance levels of interest (Krawinkler, H. and Seneviratna, G. D. P. K., 1998). Pushover analysis is performed by applying incremental monotonic load to the structure. Nonlinear behaviour is assumed to the ends of frame elements. The nonlinear behaviour at the ends of the frame element is restricted to nonlinear hinges or plastic hinges (Kadid and Boumrkik, 2008).

The structural model of the building is prepared in SAP 2000 version 14. All the beams and columns are modelled as frame element and slab is modelled as shell elements. Infill walls are not considered in the structural model. Plastic hinges are developed according to FEMA 356. SAP 2000 has built-in default hinge properties according to FEMA 356. Moment and shear hinges are applied to the beam elements. P-M-M hinges and shear hinges are applied to the column elements. The structure is subjected to both gravity and lateral load. Lateral load as per IS 1893:2002 is applied to the structure. Control node is used to monitor the displacement of the structure. The plot of this control node displacement versus the base shear is the capacity (or pushover) curve (Kadid and Boumrkik, 2008). Pushover analysis was performed in two orthogonal directions (north-south and east-west direction). Next step is the estimation of displacement demand. Displacement demand can be constructed using capacity spectrum method (ATC-40) or Displacement coefficient method (FEMA 356). Once the pushover curve and displacement demand are determined, performance check is done. In this case study capacity spectrum method is used. If the displacement demand curve and pushover curve intersects performance point exists. Using this performance point global response of the structure and individual component deformations are compared with the specific performance limits of the building. If the demand curve and capacity curve do not intersect, the building will not be able to meet the seismic demand (ATC-40).

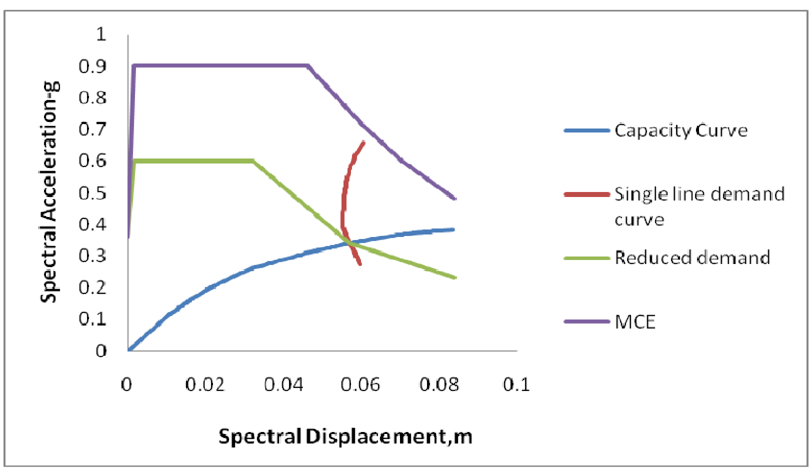

Figure 2. Results of pushover analysis along north-south direction

Pushover analysis along north-south direction illustrated that performance point exist at spectral acceleration of 0.34g and spectral displacement of 57 mm at 16.9 % effective damping (see figure 2). However, performance point does not exist in east-west direction, implying that the structure does not have adequate lateral strength and stiffness in this direction (see figure 3). Therefore, the building needs to be retrofitted to enhance the performance of the building.

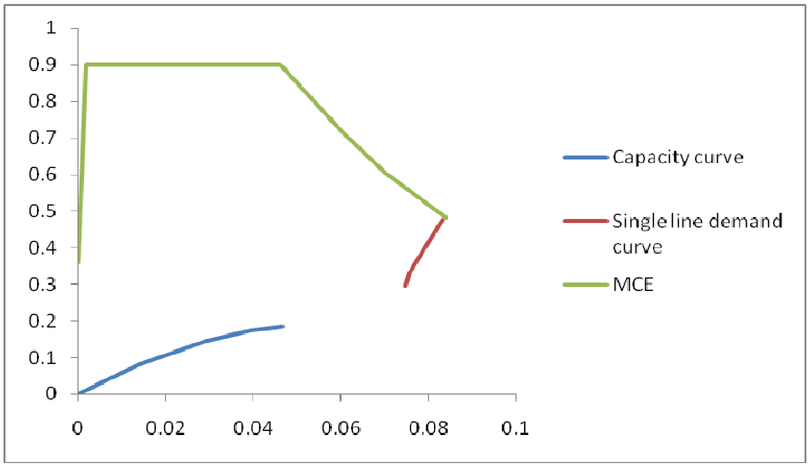

Figure 3. Results of pushover analysis along east-west direction

2.2.4. Probable performance of building at different intensities of earthquake

On the basis of the available information about the building, architectural and structural details obtained from field visit and conducting limited number of non-destructive tests, it is identified that the building under study is likely to suffer moderate to heavy structural damage at large earthquake of intensity IX on MMI scale. The probable performance of the building at different intensities of the earthquake is given in table 1.

Table 1. Probable performance of the building

Description	Probable performance of the building		
	MMI = VII	MMI = VIII	MMI = IX
Structural Damage	Slight	Moderate	Moderate-Heavy
Non-Structural Damage	Moderate	Heavy	Heavy-Very Heavy

2.2.5. Summary and Recommendation

It is suggested to retrofit the existing building for improving the performance during large earthquakes. There is a need to relocated some of the walls for adjustment of vertical alignment. These walls are to be used as structural walls for a supplementing the incomplete moment resisting frames in east-west direction. Brick walls which are not built on the column centrelines are to be tied for preventing out of plane collapse of these walls.

4. CASE STUDY: RETROFIT OF THE ASSESSED BUILDING

The seismic vulnerability assessment of the existing building showed that it could suffer moderate-heavy structural damage and very heavy non-structural damage at an intensity of IX on MMI scale. Therefore, the building needs to be retrofitted in order to improve its performance at large earthquakes.

4.1. Suggested method of retrofitting of the assessed building

Different strategies are adopted for seismic strengthening of the existing reinforced concrete frame buildings. Improving the probable seismic performance of the existing building and reducing the existing risk to an acceptable level are two key strategies adopted for seismic strengthening of the existing building (ATC-40, 1996).

The study considered the structural system of the building, its major structural problems and different available options of retrofitting. Composite jacketing of columns and selected infill walls can be one of the options of retrofitting (Griffith and Pinto, 2000). Previous studies have shown that wall jacketing and addition of new shear walls are considered the most practically feasible and economically viable method in Nepal (Shrestha and Bothara, 2007). On this background, retrofitting by addition of brick walls at few strategic locations, composite jacketing of portions of selected peripheral and internal walls adjacent to the columns and the column itself, and tie up of vulnerable walls on cantilevers are selected as the most appropriate retrofitting technique for this building. Similarly, slender wall at staircase in third floor is suggested to demolish where the height of the wall is more than 3 feet above staircase flight to reduce the extent of damage from out-plane failure of wall. These options are chosen with due consideration of architectural and socio-economic impact of the existing building structure. These methods are suggested only for life safety requirement. The building may suffer moderate structural damage at large earthquake even if this retrofitting option is implemented. The retrofit scheme is shown in figure 4.

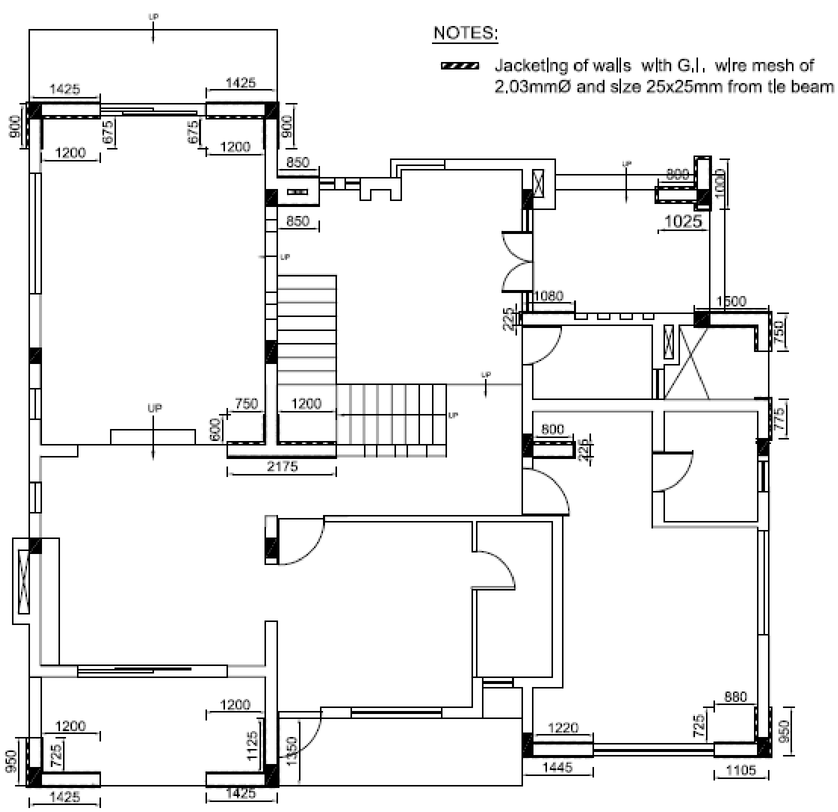

Figure 4. Retrofit scheme with addition of walls and applying wall jacketing

4.2. Pushover Analysis with the retrofitting scheme

The structural model of the building is prepared in SAP 2000 version 14. All the beams and columns are modelled as frame element and slab is modelled as shell elements. Infill walls are not considered in the structural model. The jacketed portions of brick walls adjacent to the columns are accounted for in the structural model by increasing the sizes of the columns so that the stiffness of the columns and adjacent walls combined is equal to the stiffness of the idealized columns of increased sizes. Now, pushover analysis is performed on this retrofitted structure as per the methodologies outlined in section 2.2.3.

Pushover analysis along north-south direction of the retrofitted building illustrated that performance point exist at spectral acceleration of 0.47g and spectral displacement of 41 mm at 17.3 % effective damping (see figure 5). Similarly, Pushover analysis along east-west direction of the retrofitted building illustrated that performance point exist at spectral acceleration of 0.38g and spectral displacement of 51 mm at 16.7 % effective damping (see figure 6).

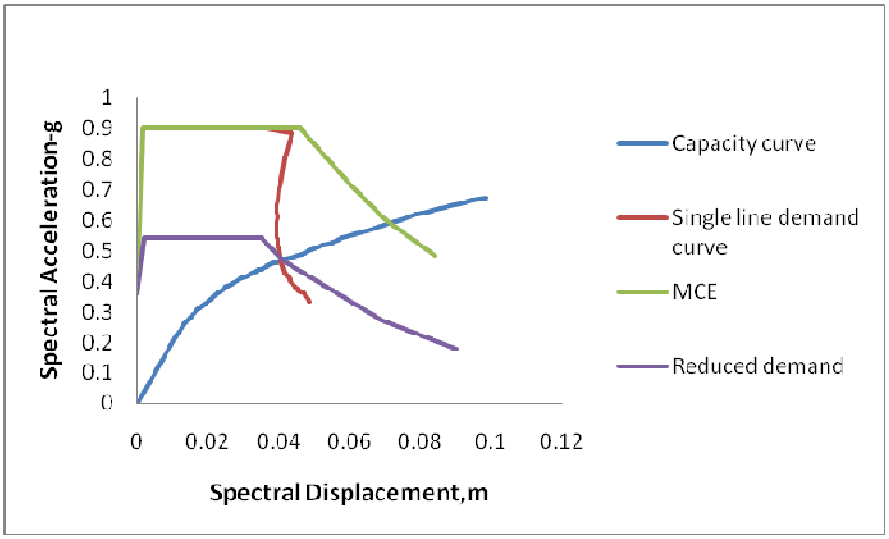

Figure 5. Results of pushover analysis of retrofitted building along north-south direction

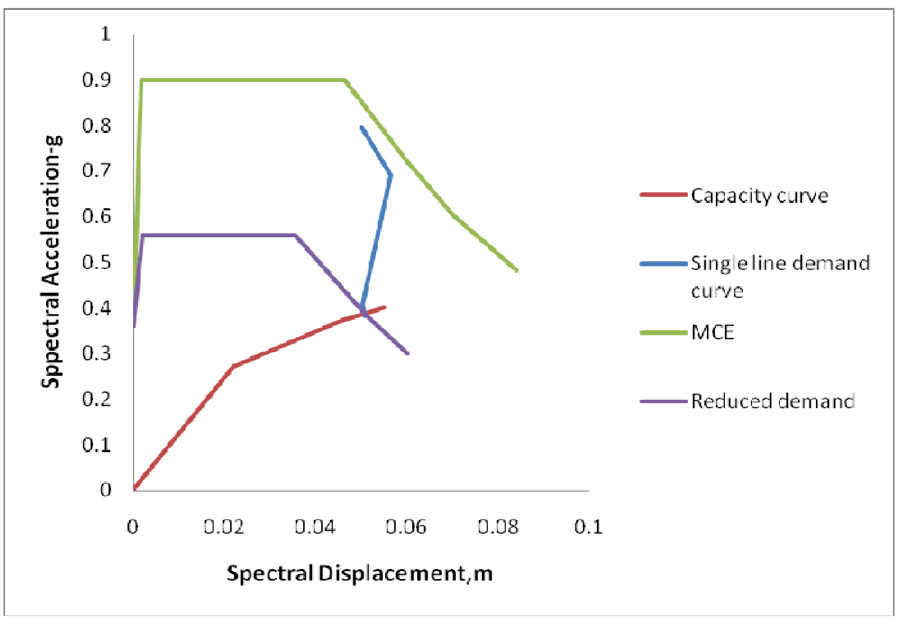

Figure 6. Results of pushover analysis of retrofitted building along east-west direction

5. CONCLUSIONS

Seismic vulnerability assessment of a typical engineered reinforced concrete frame building with masonry infill showed the increased risk during large earthquakes. The existing reinforced concrete frame building with masonry infill was retrofitted successfully by addition of wall at few strategic locations and jacketing the portions of walls adjacent of the columns. This is the most practically feasible and economically viable option of retrofitting considering the socio-economic condition of Nepal.

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