Nonlinear Static Analysis of an Infill Framed Reinforced Concrete Building

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SUMMARY:
Structural frame buildings with masonry infill panels make up a significant portion of the buildings constructed in Pakistan prior to the development of comprehensive seismic design standards. These structures may be regarded as Earthquake Risk buildings, therefore an evaluation of their level of seismic performance may be required. This paper reports on the evaluation of a reinforced concrete frame building with block masonry infill panels on the exterior and interior walls. The evaluation uses an equivalent strut approach for modelling the infill panels and nonlinear static analysis is performed for the evaluation of this building. For modelling of nonlinear hinges in beam columns and struts ASCE 41 is used. After the detailed evaluation of this structure it is observed that the building have few deficiencies which need to be retrofitted for enhancing the strength.

Keywords: infill; risk; retrofit; hinge; strut

1. INTRODUCTION

Since inelastic behavior is intended in most structures subjected to infrequent earthquake loading, the use of nonlinear analyses is essential to capture behavior of structures under seismic effects. The employment of Nonlinear Static Procedures (NSP) in the seismic assessment of existing structures (or design verification of new ones) has gained considerable popularity in the recent years, backed by a large number of extensive verification studies that have demonstrated its relatively good accuracy in estimating the seismic response of buildings.

Due to its simplicity, the structural engineering profession has been using the nonlinear static procedure (NSP) or pushover analysis. Modeling for such analysis requires the determination of the nonlinear properties of each component in the structure, quantified by strength and deformation capacities, which depend on the modeling assumptions. Pushover analysis is carried out for either user-defined nonlinear hinge properties or default-hinge properties, available in some programs based on the FEMA-356 and ATC-40 guidelines.

The NSPs may be divided into two main categories. The first category of NSPs consists on Capacity Spectrum Method (CSM), suggested by Freeman and collaborators (1975 and 1998) and implemented in ATC-40 guidelines (1996), and the equally innovative N2 method introduced by Fajfar and co-workers (1988 and 2000) and later included in Eurocode 8 (CEN, 2005). These proposals are characterized by their simplicity and usually consider a first mode and/or uniform load distributions in computation of the pushover/capacity curve. The second category consists on the more recent proposals of Chopra and Goel (2002 and 2004) on a Modal Pushover Analysis (MPA), of Kalkan and Kunnath (2006) who propose an Adaptive Modal Combination Procedure (AMCP) and of Casarotti et al. (2007) introducing the Adaptive Capacity Spectrum Method (ACSM). All of them present improvements with respect to their predecessors, such as the inclusion of higher modes contribution and the consideration of progressive damage.

In this study capacity spectrum method (CSM) is used because it gives a visual representation of capacity-demand equation, suggests possible remedial action if the equation is not satisfied and easily incorporates several limit states, expressed as station on the load displacement curve of the structure. The major steps of CSM are listed below,

1. Construction of General Response Spectrum
2. Transformation of General Response Spectrum into Demand Spectrum
3. Construction of Pushover Curve
4. Transformation of Pushover Curve into Capacity Spectrum
5. Determination of Performance Level on the basis of Performance Point

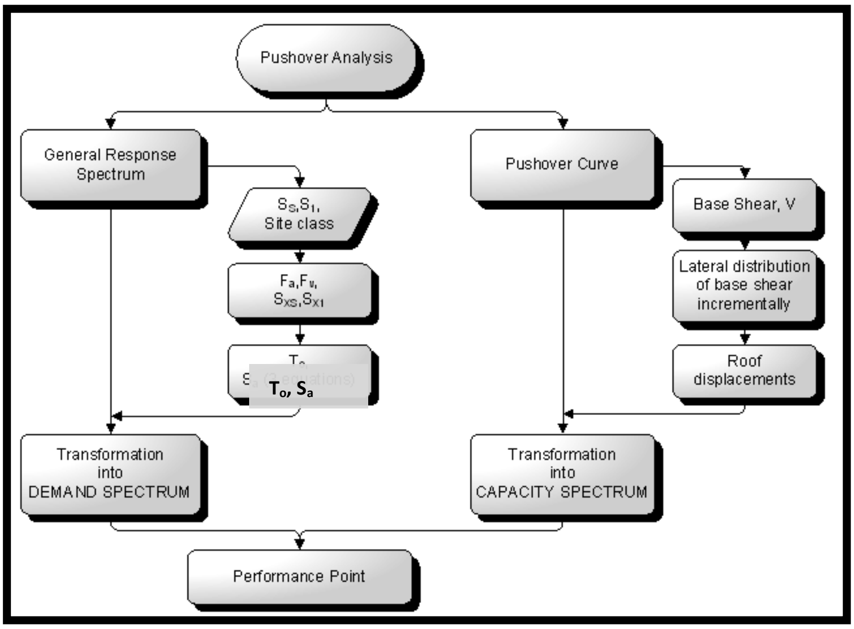

Figure 1.1. Pushover Analysis

2. BRIEF ARCHITECTURAL AND STRUCTURAL DESCRIPTION OF BUILDING

It is a reinforced concrete infill framed building with eight storeys including the ground floor. The building has shops located at the ground floor and the mezzanine floor has offices, while the above floors have residential apartments. The overall dimensions are 16.5m x 22.5m, and the total building height is approximately 27m. The foundations are mainly isolated footings. The building structure consists of beam slab system. There is a reinforced concrete lift core, which is not centrally located. The sizes and other details of various structural elements are shown in figures below from Figure 2.1 to Figure 2.4.

Figure 2.1. Front View of Building

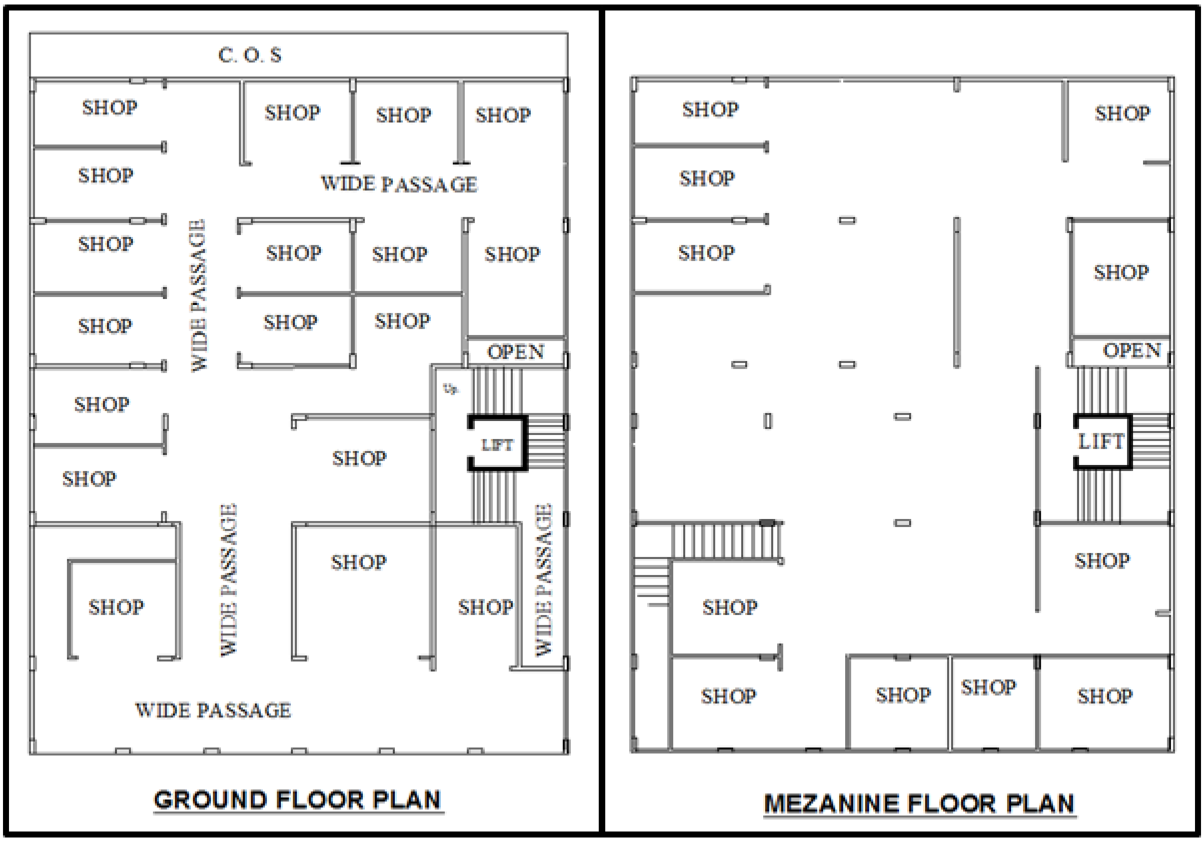

Figure 2.2. Ground Floor Plan/ Mezzanine Floor Plan

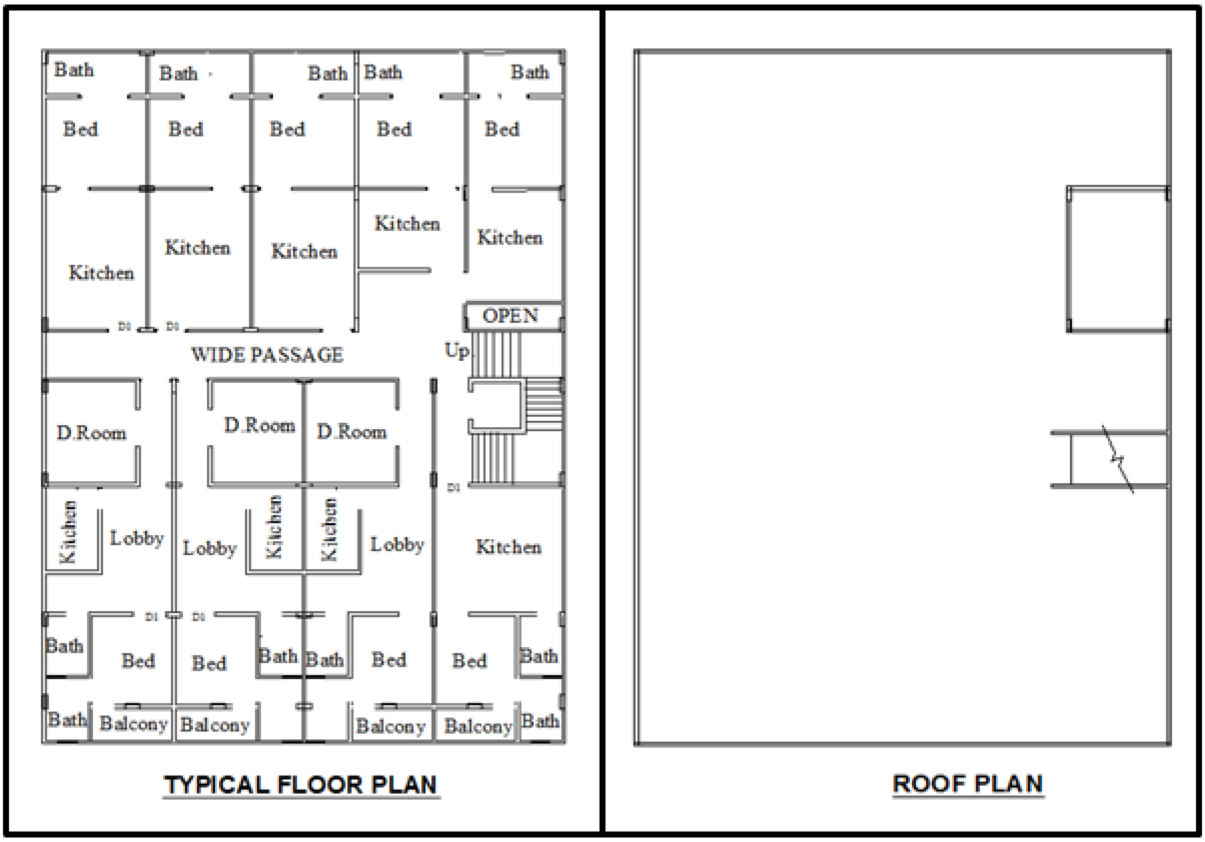

Figure 2.3. Typical Floor Plan/Roof Plan

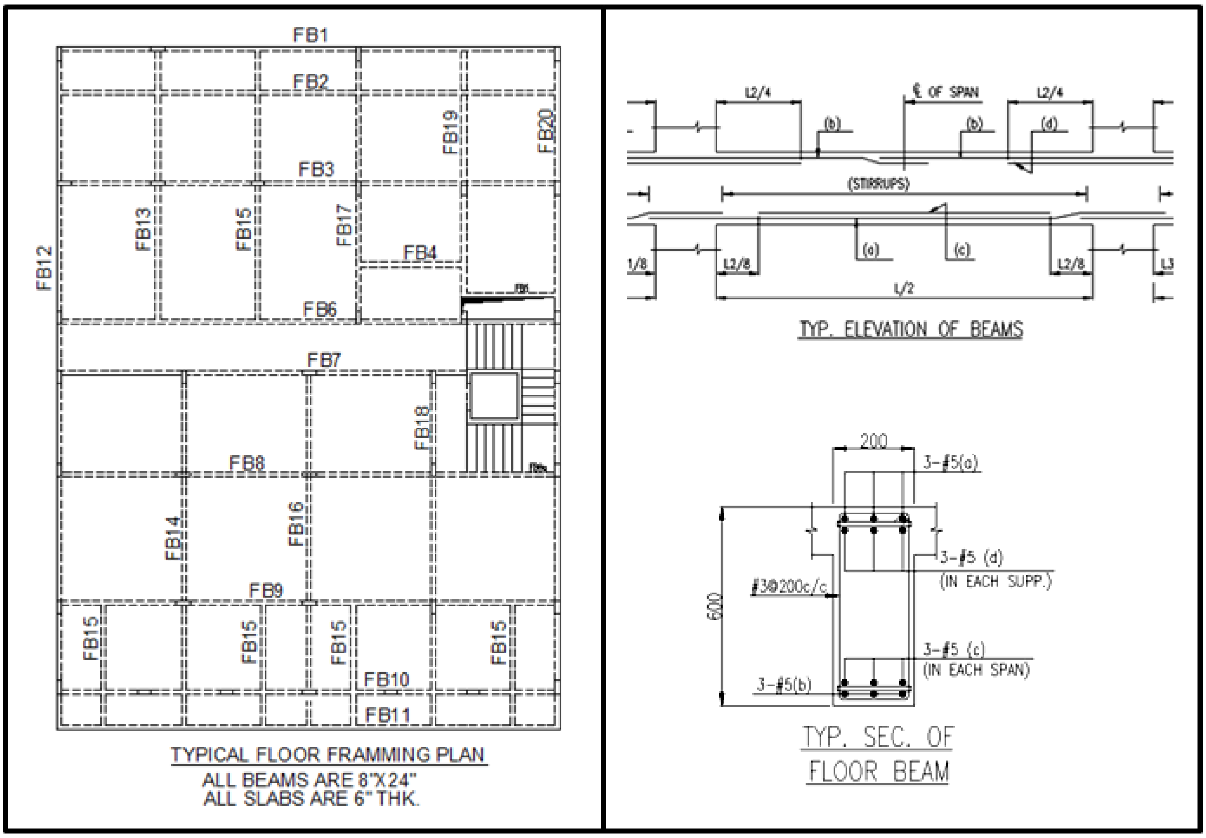

Figure 2.4. Typical Floor Framing Plan/Roof Framing Plan

3. MODELLING APPROACH

Analyses have been performed using ETABS, which is a structural analysis program used for static and dynamic analyses of building structures. In this study, ETABS Nonlinear Version 9 has been used. A description of the modelling details is provided in the following.

A three-dimensional model of building structure is created in ETABS to carry out nonlinear static analysis. Beam and column elements are modelled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns. Infill panels are modelled as struts with lumped plasticity and plastic hinge is assigned at the centre of the strut. ETABS implements the plastic hinge properties described in FEMA-356 (or ATC-40). As shown in Figure 3.1, five points labelled A, B, C, D, and E defines the force–deformation behaviour of a plastic hinge. The values assigned to each of these points vary depending on the type of element, material properties, longitudinal and transverse steel content, and the axial load level on the element.

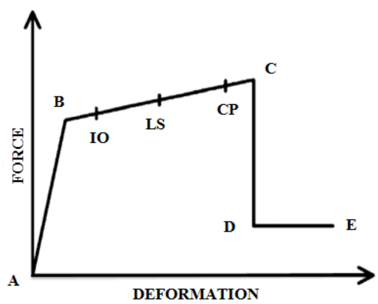

Figure 3.1. Force Deformation Relationship of a Plastic Hinge

3.1. Model Description

It is a 3D finite element model in which all beams and columns are model as per structural drawings, however infill panels are modelled as struts, for simplicity slabs are not modelled but the rigid diaphragm action is to be considered by applying diaphragm constraints on nodes located on floor levels. Material properties, geometrical properties and applied loads are listed below

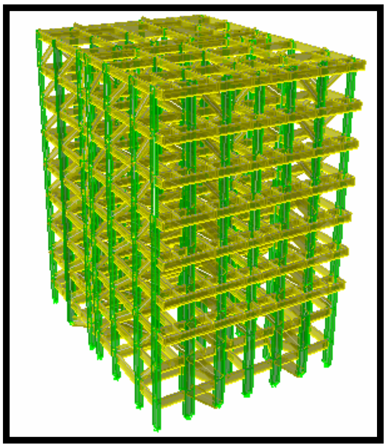

Figure 3.2. 3-D Model on ETABS

Table 3.1. Material and Geometrical Properties

Material Properties			Geometrical Properties		
Element	Modulus of Elasticity	Compressive Strength	Element	Height	Width
Beam/Slabs	21.7GPa	20.7 MPa	Beam	450/750mm	150mm
Column	25.0GPa	27.6 MPa	Column	750/600mm	300/200mm
Infill	1.5GPa	2.1 MPa	Infill	Variable	100mm

Table 3.2. Applied Loads

Applied Loads					
Dead Load	Self weight	Wall loads		Partition load	Finishes load
	Calculated by software	100mm thick wall load		1.0 KN/m^2	1.5 KN/m2
Live load	2.0 KN/m^2 for residential area and 2.5 KN/m^2 for shops area				
Earthquake load	Soil S_B	Z = 0.4g	Ca = 0.4		Cv = 0.4

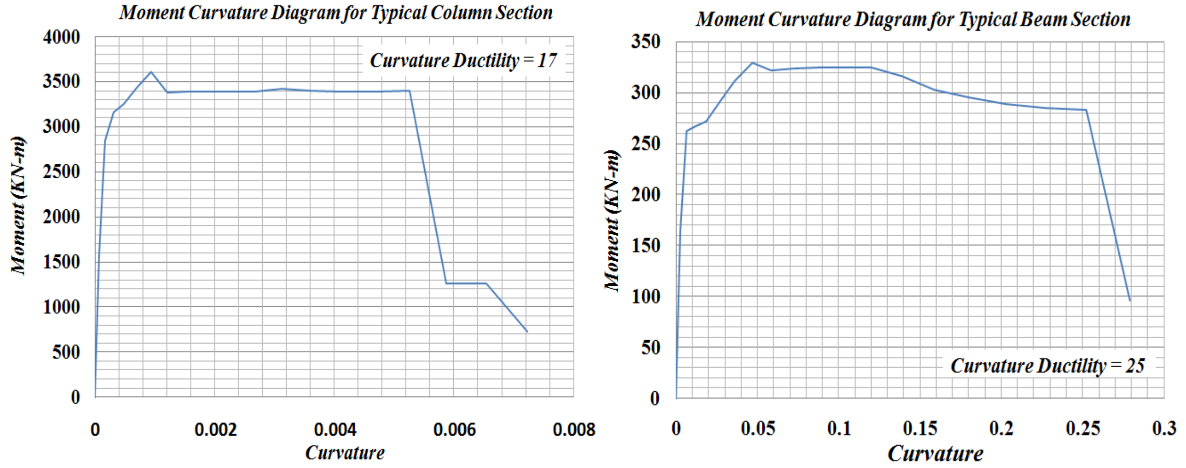

Figure 3.3. Curvature Ductility for Typical sections of Column and Beam

3.2. Non-Linear Static Analysis

Before doing nonlinear analysis, linear static analysis of structure was performed and it was observed that many columns had demand capacity ratio (DCR) > 1 but less than 2. This required further non linear static analysis. The pushover static analysis based on performance-based seismic design was adopted and hinge properties according to ATC-40 and ASCE 41-06 criteria were evaluated and manually assigned to beams, columns, and struts in the 3-D model. ASCE/SEI 41-06 standard (Seismic Rehabilitation of Existing Buildings) was adopted to compute the plastic hinge values for compressive struts, beams and columns. The hinge properties for struts were computed using lower bound unreinforced masonry properties given in table 7-1 (ASCE/SEI 41-06). For evaluation of plastic hinges for beams and columns, values given in table 6-7 and table 6-8 (Supplement 1 for ASCE/SEI 41-06) were respectively used. Pushover analysis procedure is automated in ETABS. For pushover loading patterns, restart using secant stiffness for member unloading method with P-Delta effects for geometric nonlinearity was considered. A life safety performance criterion was selected for the building.

Figure 3.4 and 3.5 shows the load-deformation curve, or *pushover* curve and the *performance point*, the point at which the demand spectra and capacity spectra intersect each other and where it is necessary to see the condition of the structure, and whether it is fulfilling the demand or not.

3.2.1 Observation in X-direction and Y-direction

The deformed shapes and state of the nonlinear hinges at the performance point (Figure 3.4 to Figure 3.5) shows that the building will be heavily damaged during the maximum considered earthquake, but that it is not likely to collapse in X-direction, however in case of Y-direction two columns exceed the limit of collapse prevention as shown in Figure 3.8. Structure response in term of floor displacements and frame resistance to base shear also shown in Figure 3.6 and Figure 3.9 during maximum considered earthquake

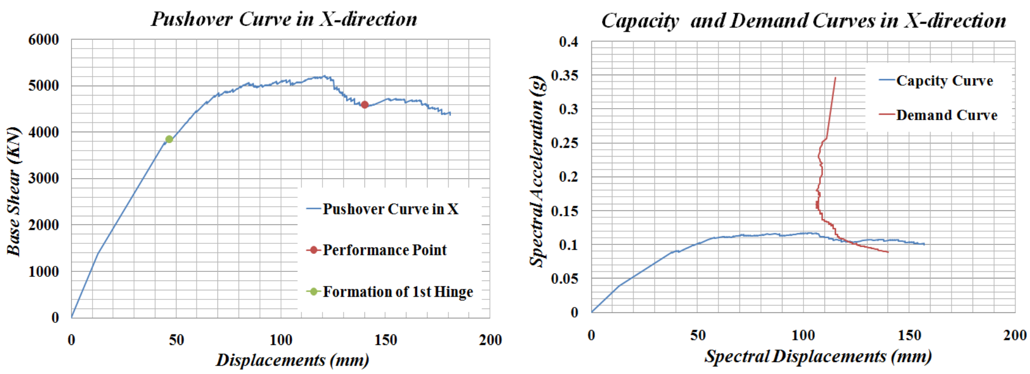

Figure 3.4. Pushover Curve, Capacity Curve and Demand Curve for Seismic Forces in X-Direction

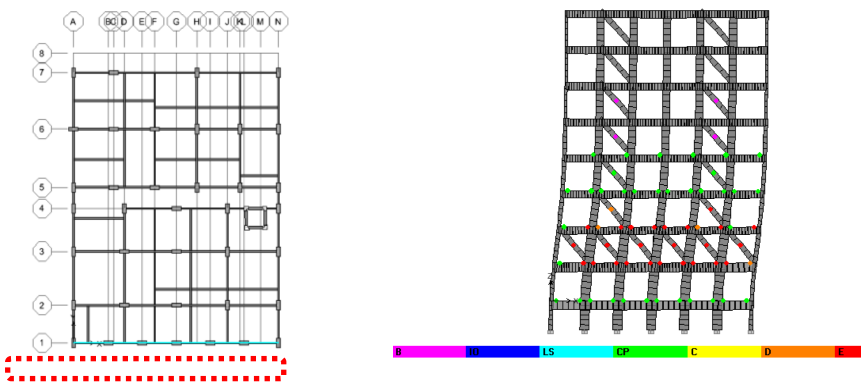

Figure 3.5. Deformed Shape at Performance Point for Frame at Grid Line 1,
With Frame Location Shown at Left

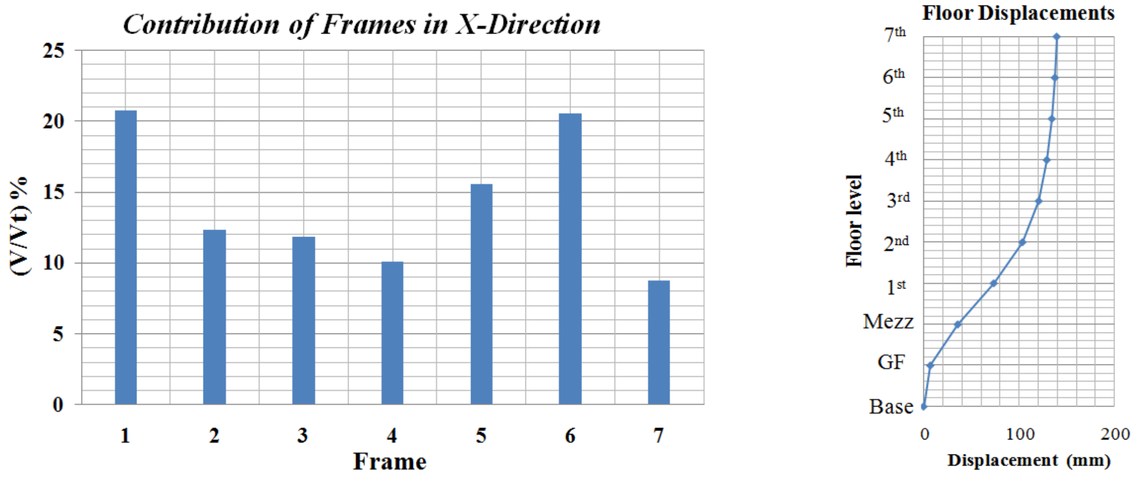

Figure 3.6. Frame Participation in X-Direction (Left) and Floor Displacements in X-Direction (Right)

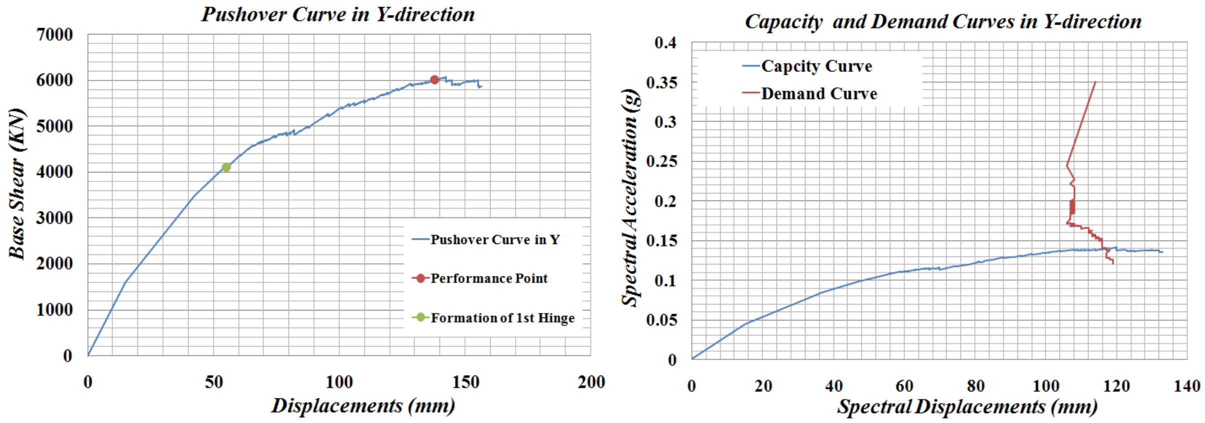

Figure 3.7. Pushover Curve, Capacity Curve and Demand Curve for Seismic Forces in Y-Direction

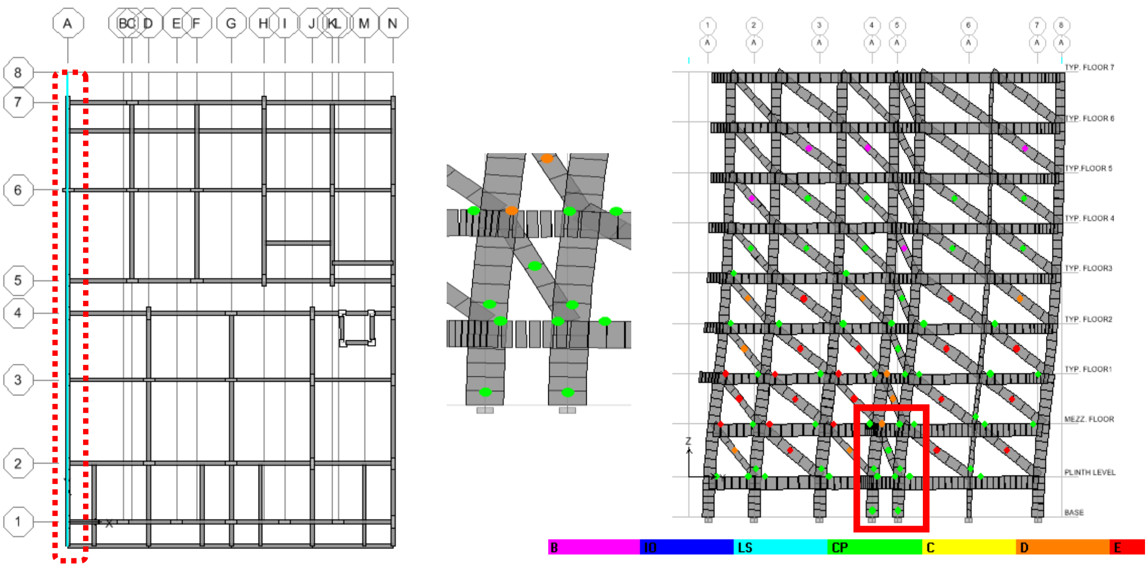

Figure 3.8. Deformed shape at performance point for frame at grid line A, with frame location shown at left

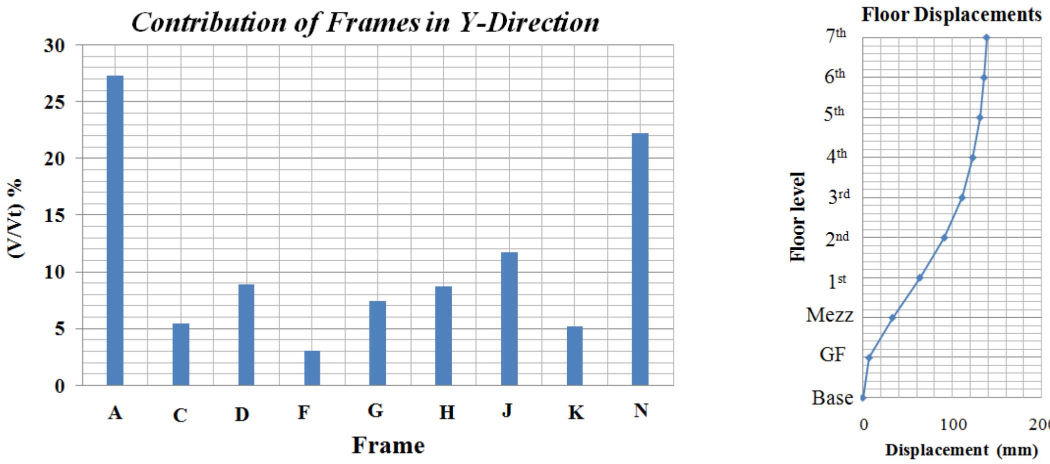

Figure 3.9. Frame Participation in Y-Direction (Left) and Floor Displacements in Y-Direction (Right)

3.2.2 Conceptual retrofitting scheme

As observed from nonlinear static analysis structure have few columns which are not meeting the criteria of collapse prevention in Y-direction so to enhance the capacity of structure few reinforced concrete walls are added conceptually in it, location of RCC walls and relevant detailing shown in Figure 3.10. These RCC walls are basically replaced the existing ordinary masonry walls, so that the same 3-D model is used with strengthened infill walls, modelled with linear compression struts and tensions ties. Results of revised model are as under

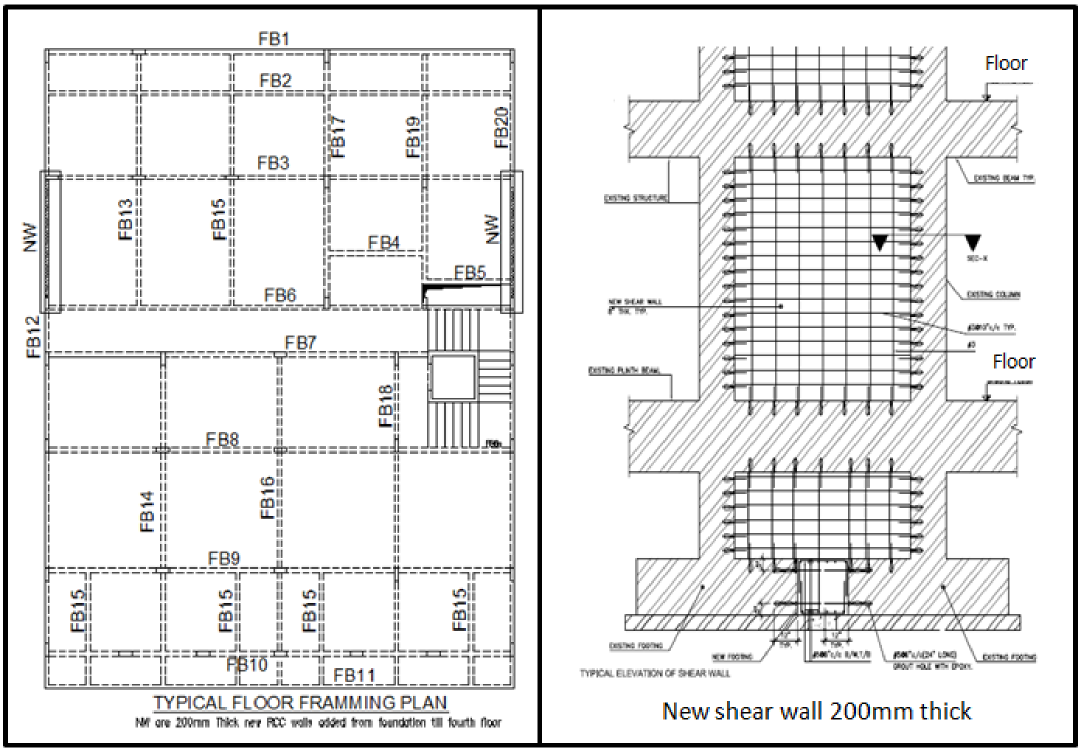

Figure 3.10. Location of Proposed RCC Walls in Plan and Typical Detail

3.2.3 Observation in Y-direction after retrofitting

After retrofitting it was observed from analysis structure satisfy the collapse prevention criteria. Improvement in structure performance clearly observed through results shown below from Figure 3.11 to Figure 3.14. In Figure 3.11 pushover curves before and after shown, in Figure 3.12 structure deformed shape shown at performance point now all columns are meeting the criteria of collapse prevention. In Figure 3.13 floor displacements are shown before and after retrofitting of structure however in Figure 3.14 contribution of existing frames and proposed RCC walls is shown.

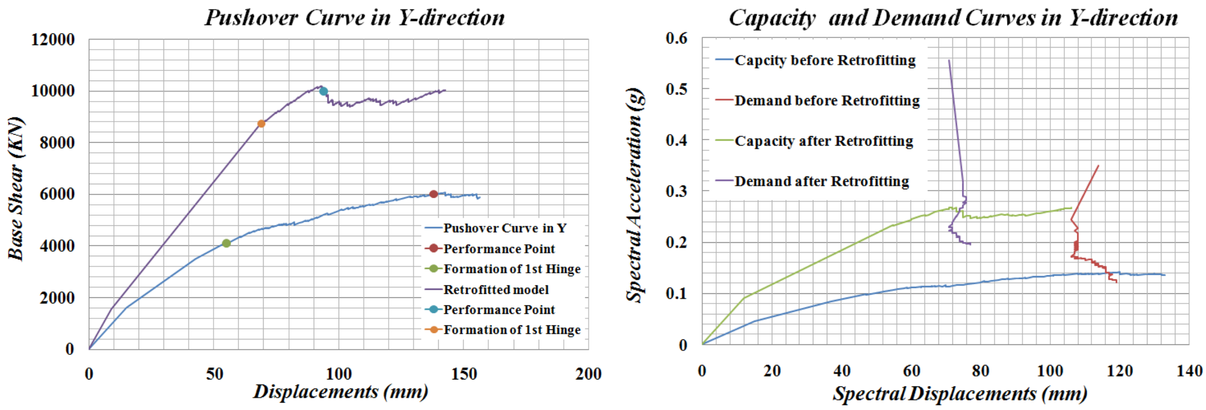

Figure 3.11. Pushover Curve, Capacity Curve and Demand Curve Before and After Retrofitting

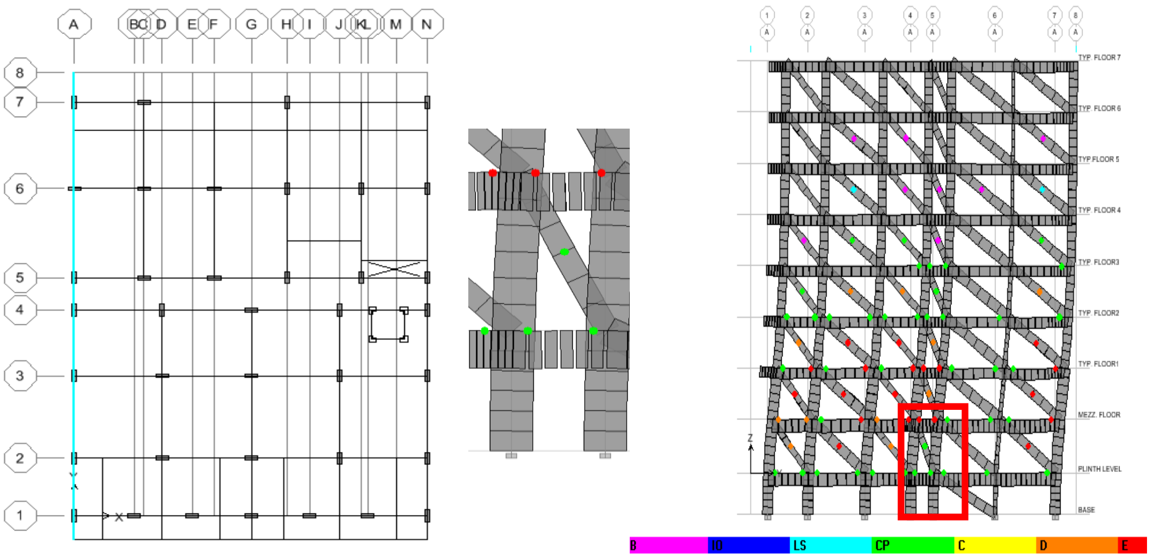

Figure 3.12. Deformed Shape at Performance Point for Frame “A” After Retrofitting

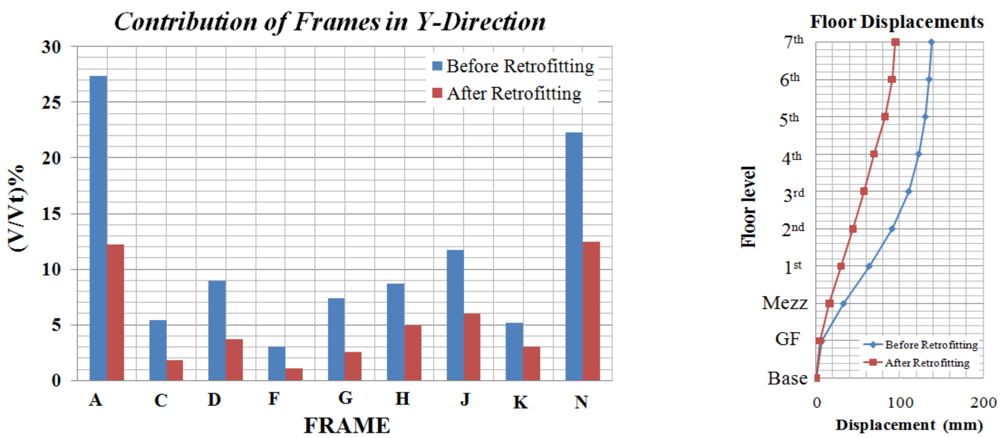

Figure 3.13. Frame Participation n Y-Direction Before/After Retrofitting (Left) and Floor Displacements (Right)

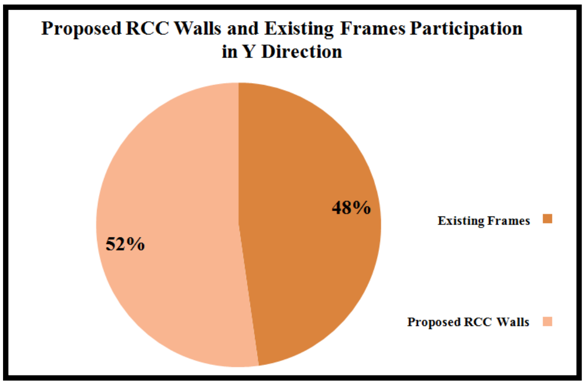

Figure 3.14. Proposed RCC Walls and Existing Frames Participation after Retrofitting

4. CONCLUSION

This case study building demonstrates the power of nonlinear analysis, by using nonlinear static analysis one can easily calculate the capacity of existing and new structure and check it against the demand. It is also used to assess the effectiveness of various kinds of innovative and lower cost retrofitting schemes, such as the rocking spine concept used in this particular structure.

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