Assessment of Infill Framed Building Constructed after 2005 Earthquake in Muzzafrabad

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SUMMARY:
Reinforced concrete framed buildings with strong infill are significant in number in Pakistan's built environment, especially in the urban areas. It is a big challenge to seismically retrofit such as they are designed without taking into account the lateral stiffness of infill. This paper is based on one of the sample building in Muzzafrabad which is constructed after 2005 earthquake and in this building after assessment as per ASCE 31 we found that the designer and the architect tried their best to take minimum risk in the planning and designing of this building. During screening phase it was observed there are few deficiencies in the building such as presence of weak storey, soft storey, torsion irregularity, captive column etc. hence there was a need to perform linear static analysis which is the evaluation phase. For evaluation a 3D model of the structure was developed on ETABS including infill. Infill panels are incorporated in this model as struts and the properties of struts such as material and geometric properties are based on ASCE 41, after evaluation phase it decided that these deficiencies are not exist in the building and the building have enough capacity against the hazard.

Keywords: infill; retrofit; stiffness; assessment; risk

1. INTRODUCTION

As per ASCI 31 the seismic assessment procedure of any building structure is a three tier procedure, tier one is the screening phase, second tier is evaluation phase and the last tier is detailed evaluation phase. Before performing seismic assessment three basic information needed regarding the structure such as level of performance, level of seismicity and building typology.
A desired level of performance shall be defined prior to conducting a seismic evaluation using the standard. The level of performance shall be determined by the owner in consultation with the design professional and by the authority having jurisdiction. Two performance levels for both structural and non-structural components are defined in section 1.3 of ASCE 31 standard one is life safety (LS) and second is immediate occupancy (IO) for both performance level, the seismic demand is based on maximum considered earthquake (MCE) spectral response acceleration values. Building complying with the criteria of this standard shall be deemed to meet the specified performance level. Level of performance of case study building is life safety (LS). The level of seismicity of the building shall be defined as low, moderate, or high in accordance with the values of spectral acceleration given in Table 1.1. S_{DS} and S_{D1} are design short-period spectral response acceleration and spectral response acceleration at one second. The level of seismicity for this case study building is high.

Table 1.1. Levels of Seismicity Definitions

Level of Seismicity	S_{DS}	S_{D1}
low	<0.167g	<0.067g
Moderate	≥ 0.167g and <0.5g	≥0.067g and <0.2g
High	≥ 0.5g	≥0.2g

The building type shall be classified as one or more of the building types listed in following table based on the lateral force resisting system(s) and the diaphragm type. Separate building types shall be used for buildings with different lateral force resisting systems in different directions, areas or levels. All possible types of building structures are define in Table 1.2.

Table 1.2. Building Types

Common Building Types	
Building Type 1	Wood Light Frames
Building Type 2	Wood Frames, Commercial and Industrial
Building Type 3	Steel Moment Frames
Building Type 4	Steel Braced Frames
Building Type 5	Steel Light Frames
Building Type 6	Steel Frames with Concrete Shear Walls
Building Type 7	Steel Frames with infill masonry Shear Walls
Building Type 8	Concrete Moment Frames
Building Type 9	Concrete Shear Walls
Building Type 10	Concrete Frames with Infill Masonry Shear Walls
Building Type 11	Precast/Tilt-up Concrete Shear Walls
Building Type 12	Precast Concrete Frames
Building Type 13	Reinforced Masonry Bearing Walls With Flexible Diaphragms
Building Type 14	Reinforced Masonry Bearing Walls With Stiff Diaphragms
Building Type 15	Unreinforced Masonry Bearing Walls

In this case study the considered building belongs to the building typology-10 which is concrete frames with infill masonry shear walls in Y-direction however in X-direction building typology correlated to building type 8 which is Concrete Moment Frames.

1.1. Screening Phase or Tier-1 Analysis

The prime objective of tier-1 phase is to quickly identify buildings that comply with the provisions of ASCE-31. It is also familiarizes the design professional with the building, its potential deficiencies and the structure response during earthquakes.
In screening phase few structural checklists are used those checklists are listed in ASCE-31 as per level of performance, level of seismicity and building typology. According to those checklists one can easily evaluate what kinds of vulnerabilities are present in a building just by site supervision and also observing the architectural/structural drawings of structures. Checklists are divided into three categories one is related to building system second is related to lateral force system and the third one is geological site hazard and foundation checklist.

1.2. Evaluation Phase or Tier-2 Analysis

In Evaluation phase or Tire-2 analysis detailed linear static analysis of building is performed as per relevant seismic codes (UBC-97, IBC200, EC8, etc) and the performance of structural elements (Beams and Columns) is checked as per code requirement. In this phase one of the following linear analysis method can be used and the application of these methods depend upon the type of structure.

- Linear static method
- Linear dynamic method
- Special method

The linear static method is applicable to all buildings in which building height is less than 30 meter or no irregularities such as mass, stiffness or other geometrical irregularities present in the structure. However special method is used for those structures which have unreinforced masonry bearing wall system with flexible diaphragm.

In Tier-2 analysis a 3-D model of structure including infill panels is required now the question is that how to considered these infill panels in our model, for that as per ASCE-41 “a concrete frame with masonry infill resisting lateral forces within its plane can be model two ways depend upon the response of infill panel, modelling of the response using a linear elastic model shall be permitted provided that the infill will not crack when subjected to design lateral forces. If the infill will not crack when subjected to design lateral forces, modelling the assemblage of frame and infill as a homogeneous medium shall be permitted. However for a concrete frame with masonry infills that will crack when subjected to design lateral forces, modelling of the response using a diagonally braced frame model, in which the columns act as vertical chords, the beams act as horizontal ties, and the infill acts as an equivalent compression strut, shall be permitted. In this case study building the infill panels are model as struts because these infill panels are not design for lateral forces acting on it.

1.2.1. Properties of struts

The in-plane behaviour of the infill panel is modelled by a diagonal strut, following the procedures given in Section 7.5 of FEMA 356 (FEMA 2000). The strut is given a thickness (normal to the wall), t_{inf} , equal to the actual infill thickness [in]. The width of the strut “a” is given by Equation 7-14 in FEMA 356 (FEMA 2000), and reproduced below:

$$a = 0.175\left(\lambda_1 h_{col}\right)^{-0.4} r_{\inf} \tag{1.1}$$

$$\lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4 E_{fe} I_{col} h_{\inf}}\right]^{\frac{1}{4}} \tag{1.2}$$

where h_{col} is the column height between centerlines of beams [in], h_{inf} is the height of infill panel [in], E_{fe} and E_{me} are the expected moduli of elasticity of the frame and infill materials, respectively [ksi]. I_{col} is the moment of inertia about the out-of-plane axis of the column cross section [in^4]. L_{inf} is the length of the infill panel [in], r_{inf} is the diagonal length of the infill panel [in], θ is the angle whose tangent is the infill height-to-length aspect ratio [radians], i.e., tan $\theta = h_{inf}/L_{inf}$, and λ_1 is a coefficient used to determine the equivalent width(a) of the infill strut .

1.3. Detailed Evaluation Phase or Tier-3 Analysis

The final evaluation phase is the detailed evaluation phase or tier-3 analysis in this phase nonlinear static or nonlinear dynamic analysis of same model (which we used in tier-2) with few minor modifications according to the type of analysis is used and the performance of individual component as well as the global performance of structure is checked against the demand. In detailed evaluation phase basically the local ductility of component and global ductility of structure is evaluated against the hazard demand.

2. BRIEF ARCHITECTURAL AND STRUCTURAL DESCRIPTION OF BUILDING

It is an infill framed structure however the infill walls are only present in shorter direction, total number of floors in the structure are four. The framing system used in the building is beam slab system. The building is located in zone 3 as per uniform building code (1997). Typical storey height is 3.0m, typical beam sizes are 450x438 mm^2, all columns are 450x450 mm^2 and slabs are 138mm thick. Concrete strength used for all structural elements is 21MPa and strength of steel is 414MPa.

Foundation type used in the building is raft and placed on hard rock.

Other details are mentioned in figures below,

Figure 2.1. Front View of Building

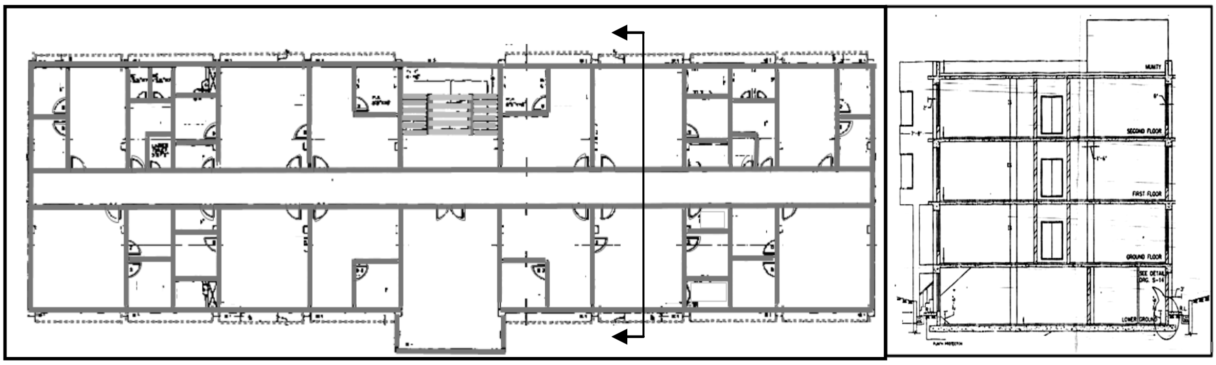

Figure 2.2. Typical Plan and Section of Building

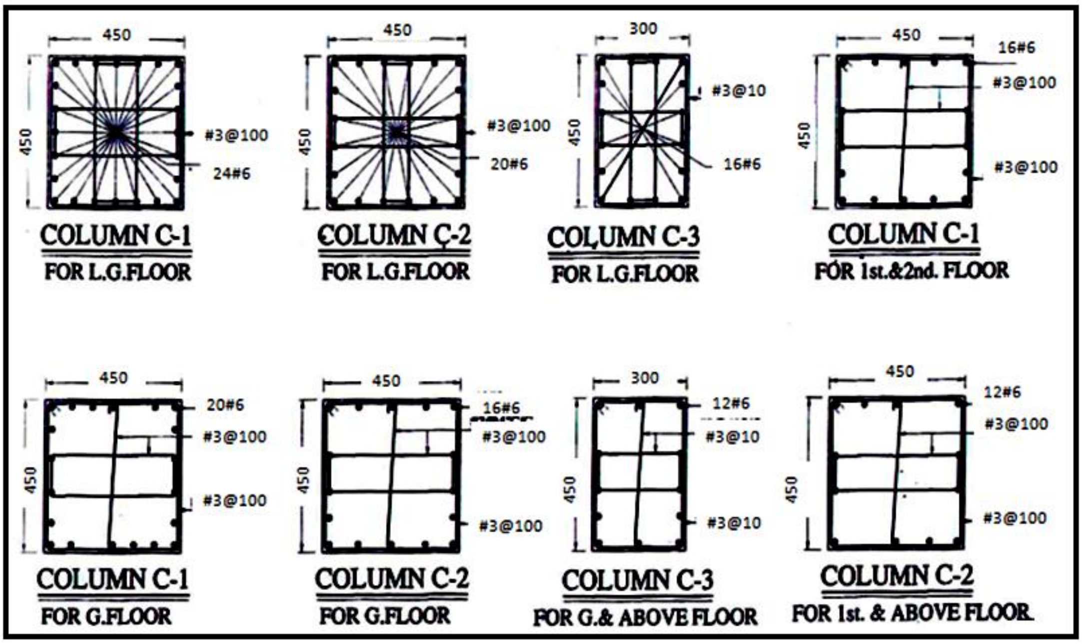

Figure 2.3. Structural Detail of Columns

2.2. Structure Response

Structure response can be easily investigated on the basis of modal analysis, the results of modal analysis are summarized in following figures.

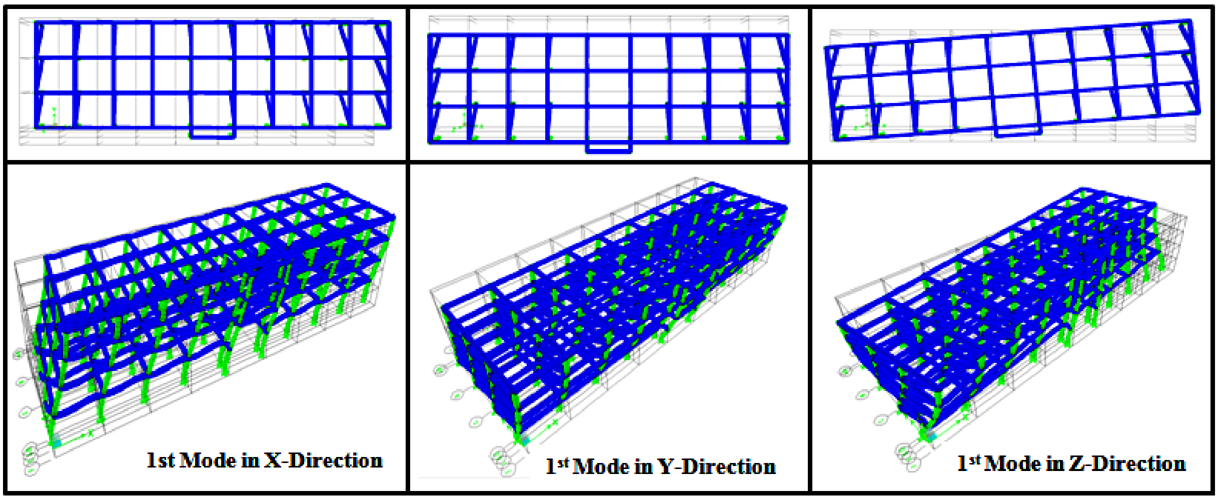

Figure 2.4. Structure Response in X, Y & Z Directions

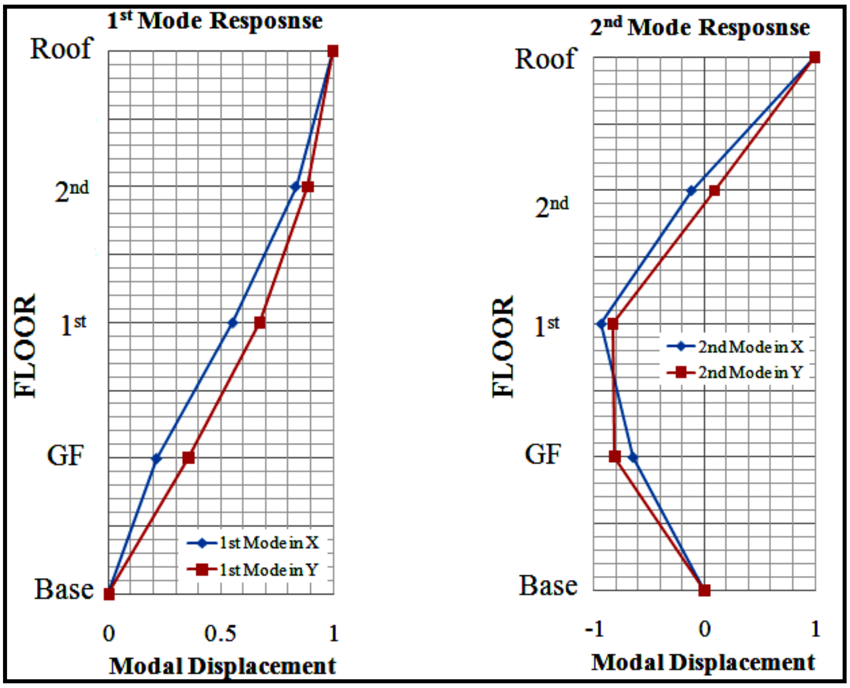

Figure 2.5. Modal Displacements in X and Y Directions

2.2. Effect of Infill

Presence of infill panels changed the response of structure drastically. In this case study building have infill panels in Y-direction, in the following diagram structure response shown with and without infill panels this diagram clearly shows that how infill alter the structure response in Y-direction (translational) mode and Z-direction (rotational) mode however (translation) mode in X-direction is not affected because there are no infill panels in X-direction.

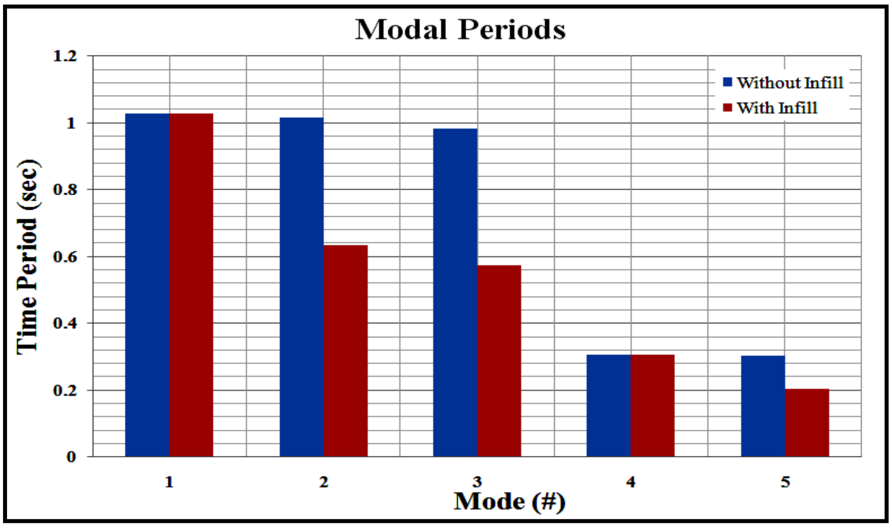

Figure 2.6. Modal Periods with and without Infill Panel

3. SEISMIC ASSESSMENT OF CASE STUDY BUILDING

3.1 Tier-1 Analysis/Screening Phase

Screening phase of this case study building is summarized as under in tabular form, one thing to be noted all these checks which are considered to be vulnerable for this structure are listed as NC which means non complient, all these checks which are listed as NA stands for not applicable for this structure however all others are listed as C means that this structure is safe against listed vulnerability.

Table 3.1. Screening Phase or Tier Analysis

Building System		Lateral Force Resisting System		Geological Site Hazard	
Load Path	C	Redundancy	C	Liquefaction	NA
Adjacent Building	NA	Wall Connections	C	Slope Failure	NA
Mezzanine	NA	Shear Stress Check	C	Surface Fault rupture	NA
Weak Story	NC	Axial Stress Check	C	Foundation Performance	C
Soft Story	NC	Flat Slab Frames	NA	Deterioration	C
Geometry	C	Pre-Stressed Frames	NA	Pole Foundation	NA
Vertical Discontinuities	C	Captive Column	NC	Overturning	C
Mass Irregularity	NC	No Shear Failure	C	Ties Between Foundation Element	NA
Torsion Irregularity	NC	Strong Columns/Weak Beams	NC	Deep Foundation	NA
Deterioration	C	Beam Bars	C	Sloping Sites	NA
Post Tension Anchors	NA	Column Bar Splices	C		

From above table one can easily conclude that for this building it is necessary to perform tier-2 analysis because of presence of significant number of vulnerabilities in Tier-1 analysis.

3.2 Tier-2 Analysis/Evaluation Phase

In tier-2 analysis 3-D model of structure is required with suitable method of analysis in this case study building linear static method is used because the structure height is less than 30 meter and structure is more or less symmetrical in both axes.

3.2.1 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 considered by applying diaphragm constraints on nodes located on floor levels. Material, geometrical properties and applied loads are listed below

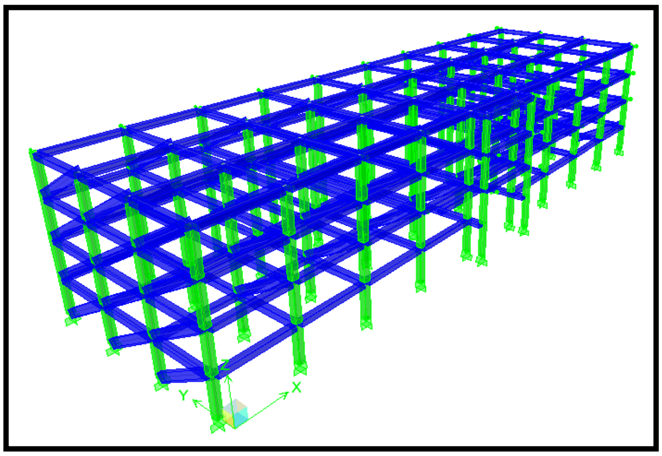

Figure 3.1. 3D Model

Table 3.2. 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	438mm	450mm
Column	21.7GPa	20.7 MPa	Column	450mm	450mm
Infill	1.5GPa	2.1 MPa	Infill	690mm	150mm

Table 3.3. Applied Loads

Applied Loads					
Dead Load	Self weight	Wall loads		Partition load	Finishes load
	Calculated by software	150mm thick wall load		1.0 KN/m^2	1.5 KN/m2
Live load	3.0 KN/m^2				
Earthquake load	Soil S_B	Z = 0.4g	Ca = 0.4		Cv = 0.4

3.2 Tier-2 analysis results

On the basis of above model those checks which are non-compliant in tier-1 analysis investigated further more in detail and confirmed these checks are actually vulnerable or not, the results of these checks are listed below.

Table 3.4. Soft Storey Check in X Direction

Story	**Load**	**storey force**	**Total Displacement**	**Stiffness**	**% diff in K (30% allow)**	
					% difference compare to	
		KN	**m**	**KN/m**	**Above storey**	**Below storey**
ROOF	EX	1335	0.0476325	28027.08	----	17.3
2ND	EX	1340	0.039555	33876.88	20.9	0.6
1ST	EX	895	0.026255	34088.75	0.6	16.9
GROUND	EX	418	0.010195	41000.49	20.3	----

Table 3.5. Soft Storey Check in Y Direction

Story	**Load**	**storey force**	**Total Displacement**	**Stiffness**	**% diff in K (30% allow)**	
					% difference compare to	
		KN	**m**	**KN/m**	**Above storey**	**Below storey**
ROOF	EY	1993.6	0.0251575	79244.76	----	13.5
2ND	EY	2015.85	0.022015	91567.11	15.5	12.7
1ST	EY	1339.45	0.01649	81228.02	11.3	12.1
GROUND	EY	623	0.0086	72441.86	10.8	----

As per code the allowable percentage difference in stiffness between adjacent stories not more than 30% so in this building there is no soft storey because all floors have stiffness difference within permissible limit.

Table 3.6. Torsion Irregularity Check

Story	Diaphragm	XCM (m)	YCM (m)	XCR (m)	YCR (m)	Allowable % diff 20	
						% diff X	% diff Y
ROOF	D1	26.759	9.306	26.7625	9.265	0.0	0.3
2ND	D1	26.763	9.239	26.7625	9.318	0.0	0.5
1ST	D1	26.763	9.239	26.7625	9.387	0.0	0.9
GROUND	D1	26.755	9.472	26.7625	9.364	0.0	0.7

As per code the allowable percentage difference between centre of mass and centre of rigidity not more than 20% so in this case there is no torsion irregularity.

Table 3.7. Mass Irregularity Check

Story	Mass	% diff in Mass (50% allow)	
		Above storey	Below storey
ROOF	699.569	---	26
2ND	940.99	35	0
1ST	940.99	0	7
GROUND	876.679	7	---

As per code the allowable percentage difference between storey masses not more than 50% so in this case there is no mass irregularity.

Table 3.8. Storey Drift Check

Story	Etab Drift X	Code Modified Drift	Etab Drift Y	Code Modified Drift
	Δ_S	Δ_M	Δ_S	Δ_M
ROOF	0.002711	0.01044	0.001273	0.00490
2ND	0.004468	0.01720	0.002244	0.00864
1ST	0.0051	0.01964	0.003187	0.01227
GROUND	0.003431	0.01321	0.003398	0.01308

As per code the allowable percentage drift should be less than or equal to 0.02.
The last check which is very important it is related to ratio of the hazard demand and the capacity of prime structural components such as beams and columns, these ratios are listed below.

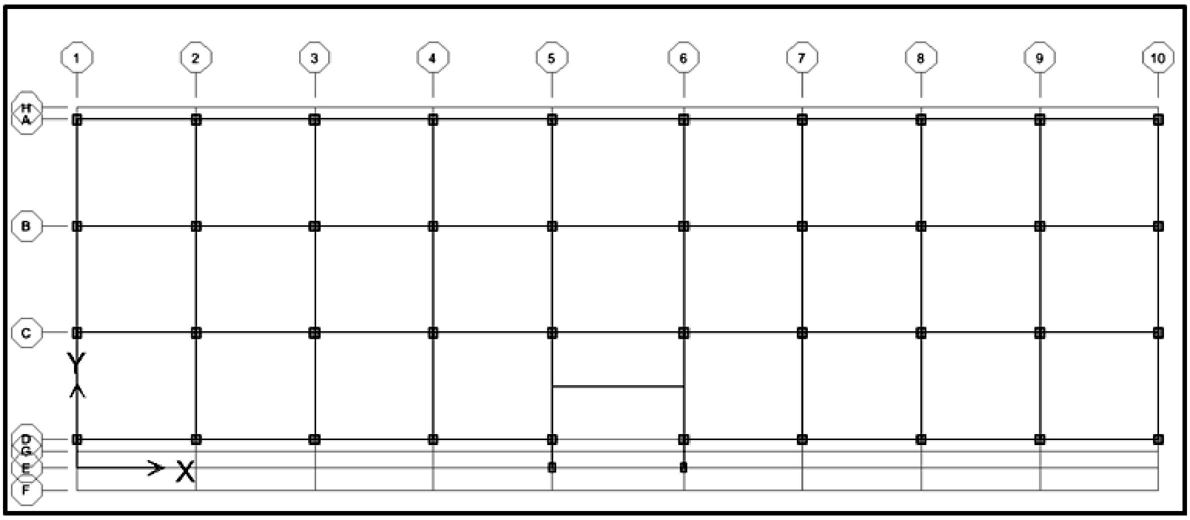

Figure 3.2. Plan View of Building Model on ETABS

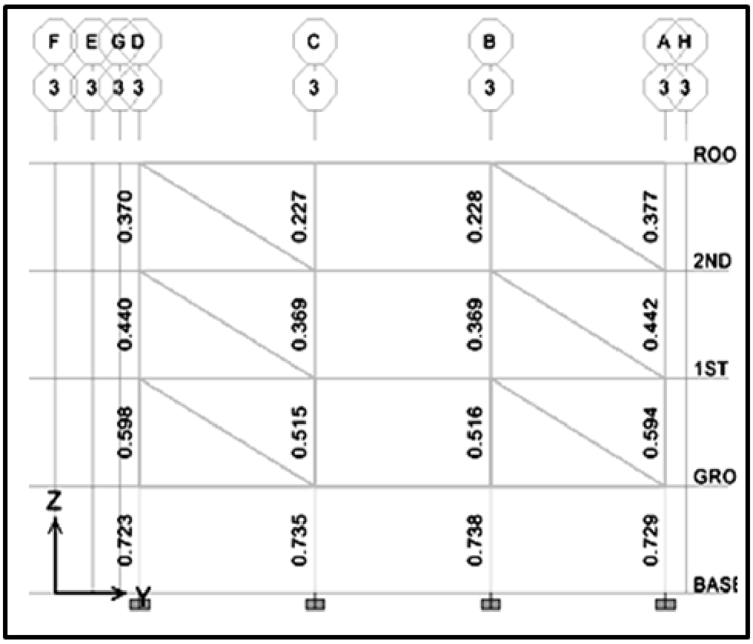

Figure 3.3. Demand Capacity Ratio of Columns at Grid 03

The software used in this study is ETABS, it is only calculate the demand capacity ratios for columns (shown above) however for beams this software provide the values of required area of steel which is shown below on the basis of that required area of steel it is very easy to calculate the demand capacity ratio just by dividing the required area of steel with provided area of steel.

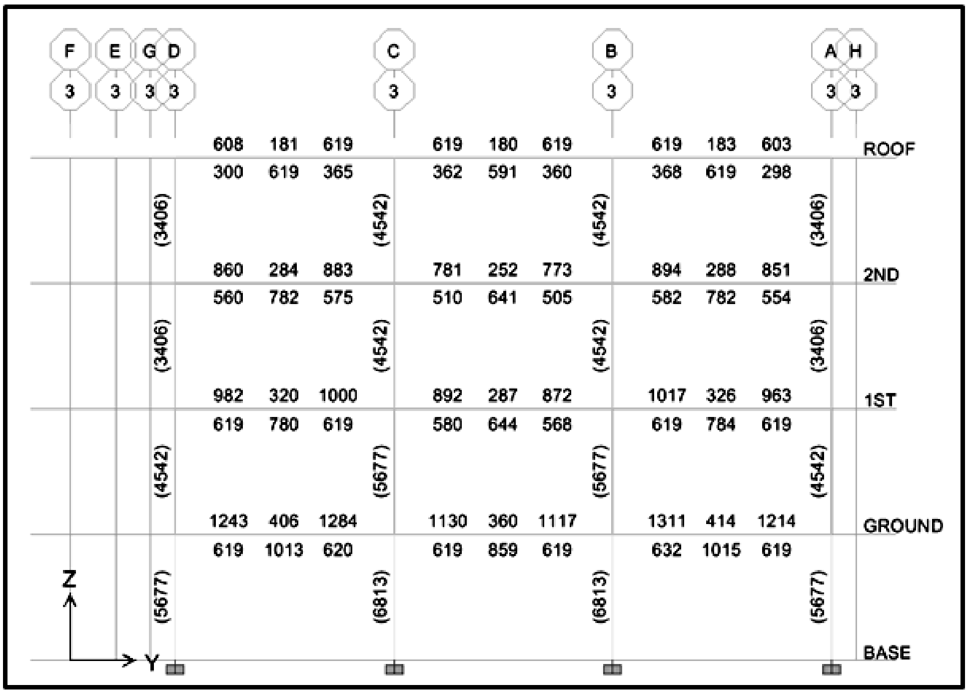

Figure 3.4. Required Reinforcement in Beams

Table 3.9. Demand Capacity Ratio for Beams

Level	Mid Span			Support		
	Required (mm2)	**Provided (mm2)**	**Ratio**	**Required (mm2)**	**Provided (mm2)**	**Ratio**
Roof	591	956	0.62	619	956	0.64
2nd	641	956	0.67	781	956	0.81
1st	644	1650	0.39	892	1925	0.46
Ground	859	1650	0.52	1130	1925	0.58

Since all vulnerabilities which are identified in tier-1 analysis or screening phase, do not exist actually and it is proved through tier-2 analysis so there is no need to perform tier-3 analysis for this case-study building.

4. CONCLUSION

The building was found to be adequately designed, but requiring removal of a small number of partial-height infill masonry walls that currently create a captive column condition at the ground storey on one side of the building.

Lesson learned from this case study, two important aspects can be concluded first one is "assessment procedure described in ASCE-31 is very well defined one can easily assess any kind of building structure by adopting simple steps which are described in ASCE-31 standards" the next one is "after 2005 earthquake in Pakistan professional are considering seriously codes detailing and designing procedure according to level of seismicity and level of performance of structure".

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