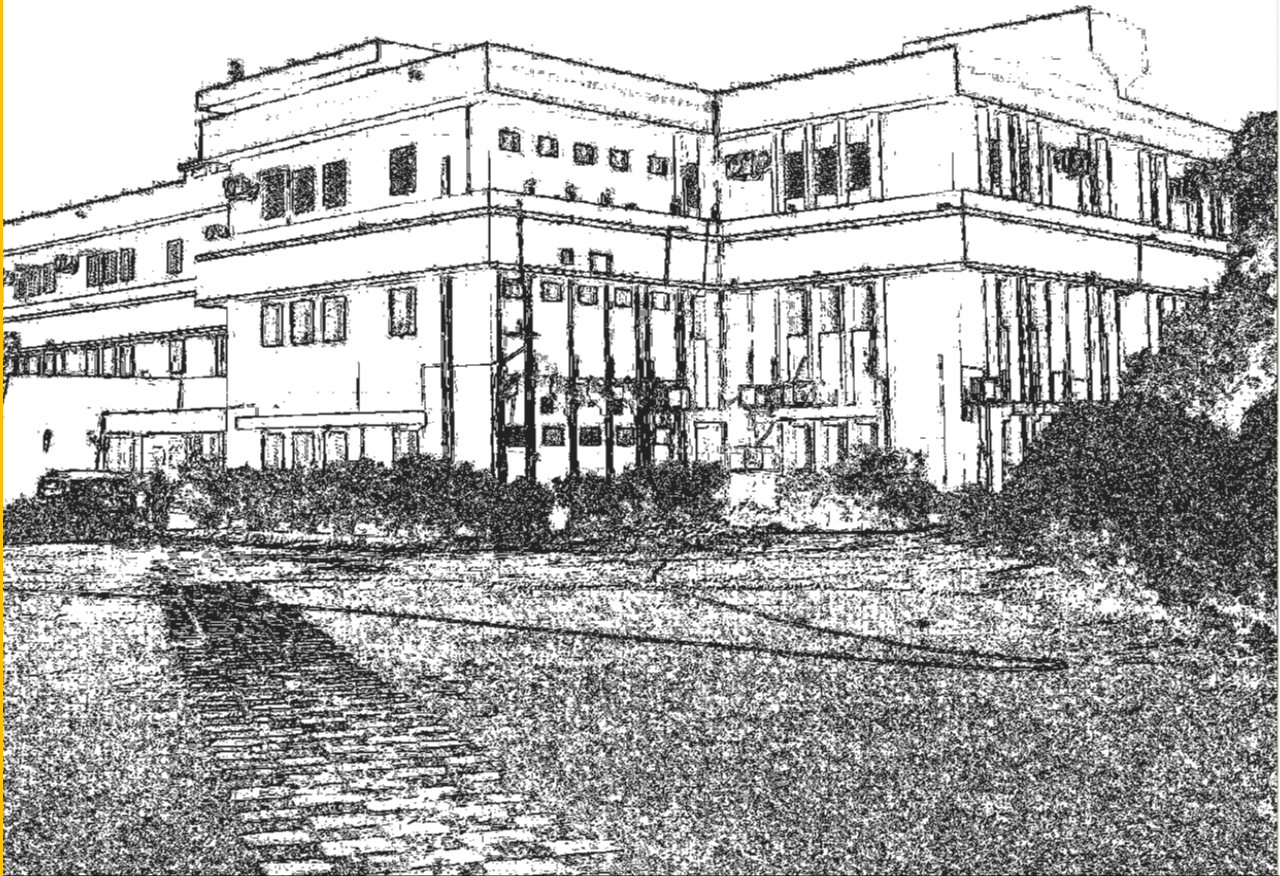

3-Storey Library Building in Karachi

A Case Study of Seismic Assessment and Retrofit Design

NED University of Engineering & Technology, Karachi

GeoHazards International
A Nonprofit Working Toward Global Earthquake Safety

Supported by the Pakistan-US Science and Technology Cooperation Program

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Summary

This case study building is a library building located on a university campus in Karachi. It is a reinforced concrete framed building initially consisting of two floors with beam-slab framing system. Later on, a small extension was built on the front of the building's ground floor, and separated from original building by expansion joints. Recently, a new floor and a detached external emergency exit stair case at rear of the building have been added. The building was constructed before the 2005 Kashmir Earthquake. Project participants selected this building as a case study because it has several seismic vulnerabilities common to low-rise buildings in Karachi: a weak story created by open working area at the ground floor, an eccentrically located stair case, a heavy rooftop water tank, and heavy, stiff unreinforced masonry infill walls that were not considered during the structural design of the building.

The case study team assessed the building's potential seismic vulnerabilities using the US Federal Emergency Management Agency (FEMA) Prestandard 310 Tier 1 Checklist modified for Pakistan conditions, as well as the American Society of Civil Engineers (ASCE) Standard 31 Tier 2 and 3 analyses and acceptance and modeling criteria from ASCE 41. The building was found to be inadequate for Seismic Zone 4 and requires retrofitting to increase the stiffness and stability of the building.

The team examined several retrofit schemes consisting of combinations of reinforced infill panels and column jacketing, and selected a retrofit solution consisting solely of reinforced infill panels.

About the Project

NED University of Engineering (NED) and Technology and GeoHazards International (GHI), a California based non-profit organization that improves global earthquake safety, are working to build capacity in Pakistan's academic, public, and private sectors to assess and reduce the seismic vulnerability of existing buildings, and to construct new buildings better. The project is part of the Pakistan-US Science and Technology Cooperation Program, which is funded by the Pakistan Higher Education Commission (HEC) and the National Academies through a grant from the United States Agency for International Development (USAID). Together, the NED and GHI project teams are assessing and designing seismic retrofits for existing buildings typical of the local building stock, such as the one described in this report, in order to provide case studies for use in teaching students and professionals how to address the earthquake risks posed by existing building. The teams are also improving the earthquake engineering curriculum, providing professional training for Pakistani engineers, and strengthening cooperative research and professional relationships between Pakistani and American researchers.

Case Study Participants

This report was compiled by Dr. Rashid Khan, Associate Professor, Department of Civil Engineering, NED University of Engineering and Technology, and Dr. Janise Rodgers, Project Manager, GeoHazards International.

This case study building was investigated by Mr. Aslam Faqeer Mohammad, Assistant Professor, Department of Civil Engineering, NED University of Engineering and Technology, Ms. Najmus Sahar Zafar, Assistant Professor, Department of Civil Engineering, NED University of Engineering and Technology, and Ms. Nighat Fatima, Senior Structural Engineer, NESPAK.

The case study team and authors wish to express their gratitude for the technical guidance provided by Dr. Gregory G. Deierlein, Professor, Department of Civil and Environmental Engineering, Stanford University; Dr. S.F.A. Rafeeqi, Pro Vice Chancellor, NED University of Engineering and Technology; Dr. Khalid M. Mosalam, Professor and Vice-Chair, Department of Civil and Environmental Engineering, University of California, Berkeley; Dr. Sarosh H. Lodi, Professor and Dean, Faculty of Engineering and Architecture, NED University Engineering and Technology; Dr. Selim Gunay, Post-doctoral Researcher, Department of Civil and Environmental Engineering, University of California, Berkeley; Mr. David Mar, Principal and Lead Designer, Tipping Mar, and Mr. L. Thomas Tobin, Senior Advisor, GeoHazards International.

Contents

Introduction

The team used the US Federal Emergency Management Agency (FEMA) Prestandard 310 Tier 1 Checklist modified for Pakistan conditions, as well as the American Society of Civil Engineers (ASCE) Standard 31-03 Tier 2 and 3 analyses and acceptance and modeling criteria from ASCE 41-06 and other documents. The Tier 1 vulnerability assessment exercise carried out provided an opportunity to evaluate a real building in the field. On the basis of the vulnerabilities found through the Tier 1 assessment, Tier 2 (linear static structural analysis) and Tier 3 (nonlinear static structural analysis) assessments were carried out to assess the vulnerabilities and potential solutions in more detail. Case study team members used structural analysis software ETABS from Computers and Structures, Inc. of Berkeley, California to perform the linear and nonlinear analyses.

Building Information

The building, shown in Figure 1, is a three storey (ground plus two) library building. The building's overall dimensions are 109'-6" by 141'-0", and it is approximately 42 feet tall. The building has a reinforced concrete moment frame structural system with unreinforced concrete block infill walls. The concrete block infill walls are 6 inches thick and located primarily at the periphery. The foundations are reinforced concrete spread footings. The building is relatively new and is in reasonably good condition. No condition assessments or repairs have been made.

Figure 1. Front elevation view of the building

The architectural and structural drawings are shown in Figure 2 to Figure 4. Concrete of fc'=3000psi and steel of fy=60000 psi are used. The typical beam size is 8"x24" and column size is 20"x20". The slabs are 6" thick. Original design calculations are not available but ACI-99 was used to design the frame elements and earthquake analysis may have been carried out using UBC-97.

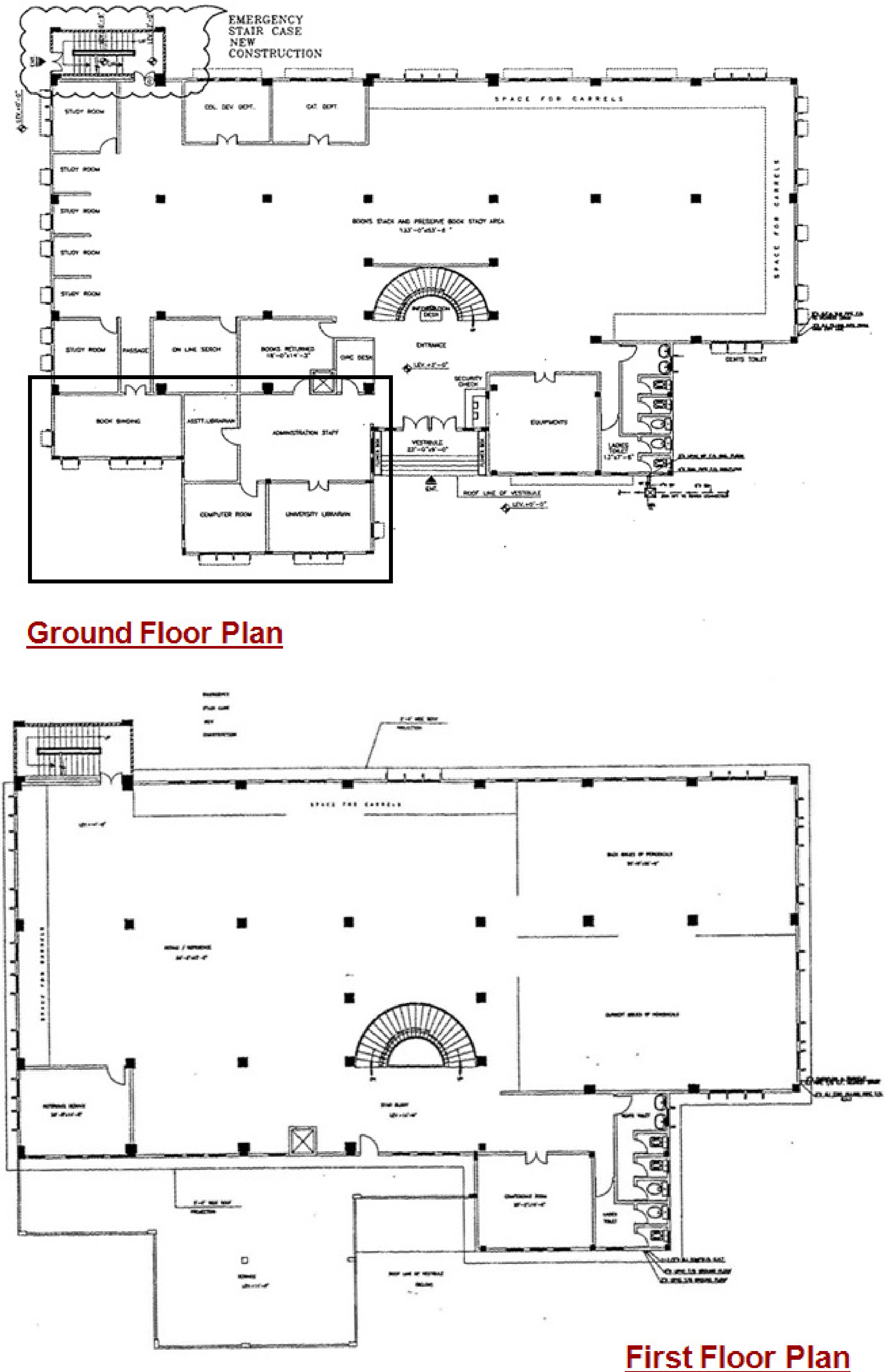


Figure 2. Architectural plans of ground floor and first floor

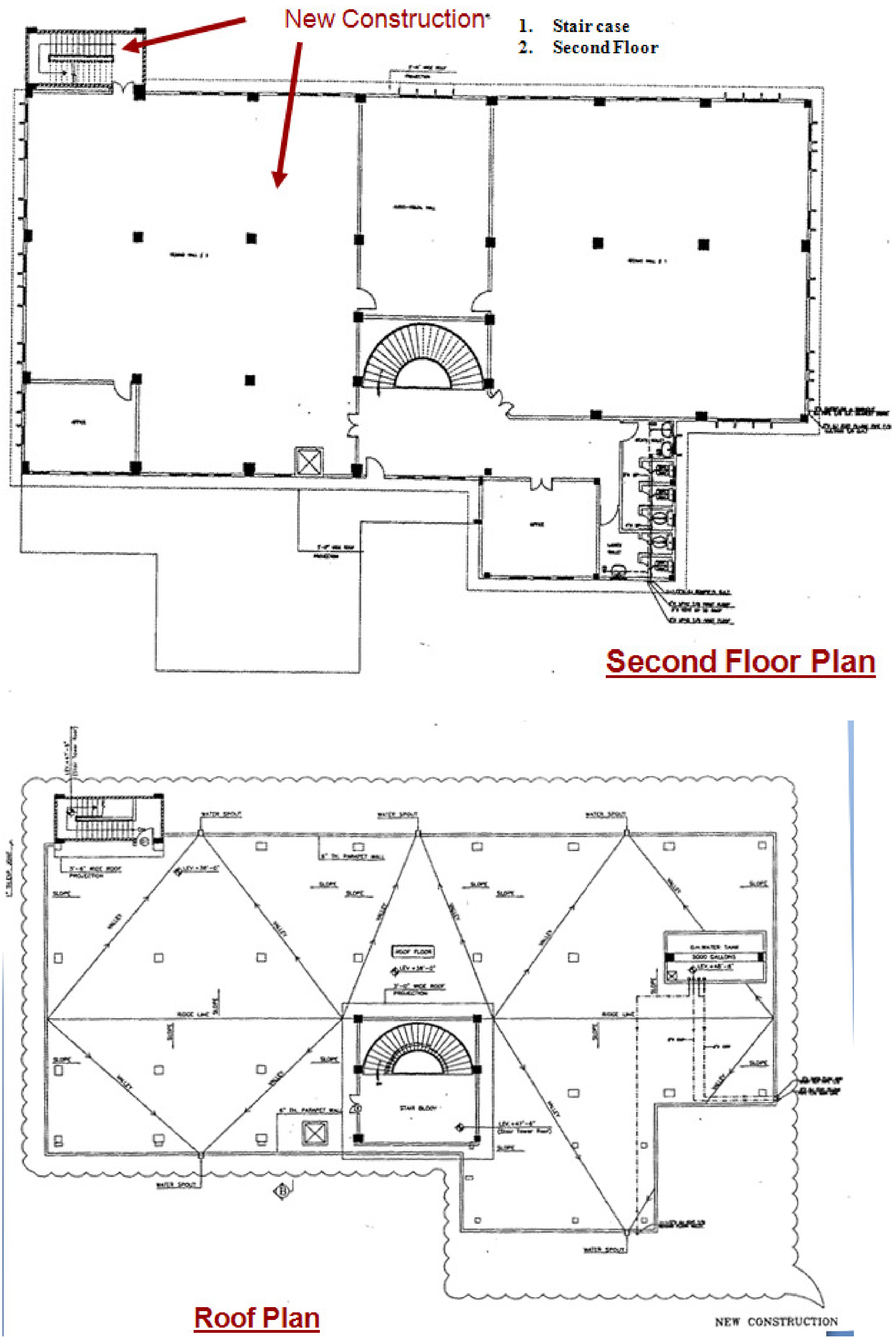


Figure 3. Architectural plans of second floor and roof

After the main building was constructed, the second storey was added, an emergency exit stair was constructed at the rear corner, and a small single storey additional block was built near the main entrance as indicated by the black outlines in the architectural plans above. The emergency exit stair and single storey additional block are separated from the main building by a small expansion joint that was not designed to accommodate seismic deformations.

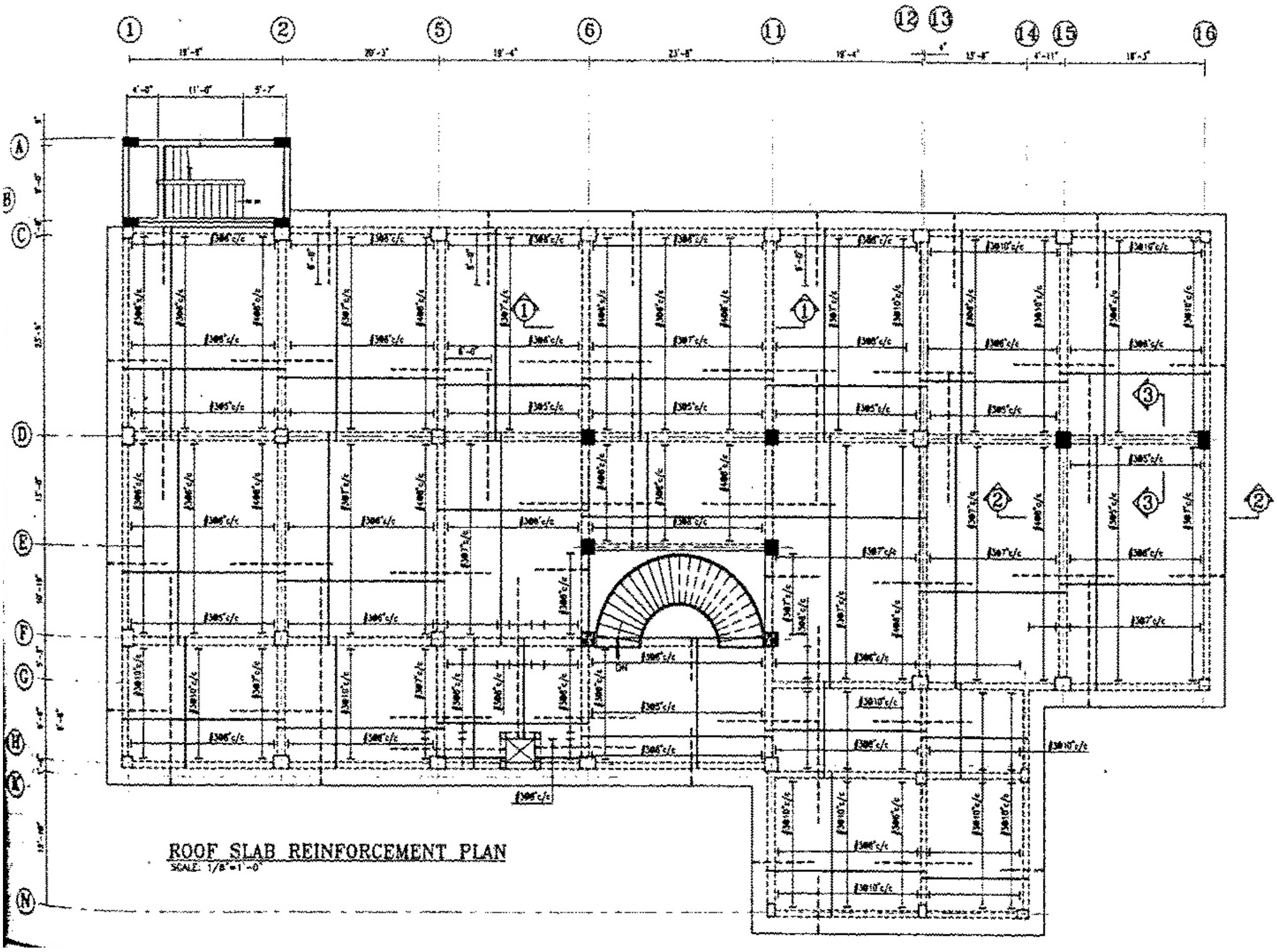


Figure 4. Structural reinforcement for roof slab in plan

Site Information

The building is located in an area with firm soil, where bedrock outcrops are often found close to the surface. No known active faults pass through or near the site. The bearing capacity of the soil is 2.0 tons per square foot (tsf).

Hazard Information

Karachi's current seismic zoning under the National Building Code of Pakistan is Zone 2B. However, there is currently significant uncertainty regarding the severity of the city's seismic hazard. For this reason, the building is being evaluated for Zone 4 of the 1997 Uniform Building Code with seismic coefficients C_a=0.4, C_v=0.4. The site is not located near any known active faults so near-source factors are not applicable.

Initial and Linear Evaluations of Existing Building

Checklist-based Evaluation

The building was assessed using a version of the FEMA 310 Tier 1 Checklist modified for Pakistan conditions. This Tier 1 assessment indicated a number of non-compliant items (i.e., deficiencies) in the building, which are summarized in the following table:

Checklist	Non-compliant Items
Building System	Soft storey Mass irregularity Drift
Lateral Force-resisting System	Interfering wall Shear stress check Axial stress check
Geologic Hazards and Foundation	None

Linear Evaluation

Figure 5 shows the 3-D model of the building generated in ETABS Nonlinear version 9.7.0. The beams and columns were modeled with linear beam-column elements, and the infill walls were modeled with single linear compression struts. The linear static analysis shows that there are a number of columns with demand/capacity ratios (DCRs) greater than one and even exceed global ductility of two, so the building is expected to respond in the nonlinear range. Please see Appendix B for linear analysis and Appendix C for non linear analysis results.

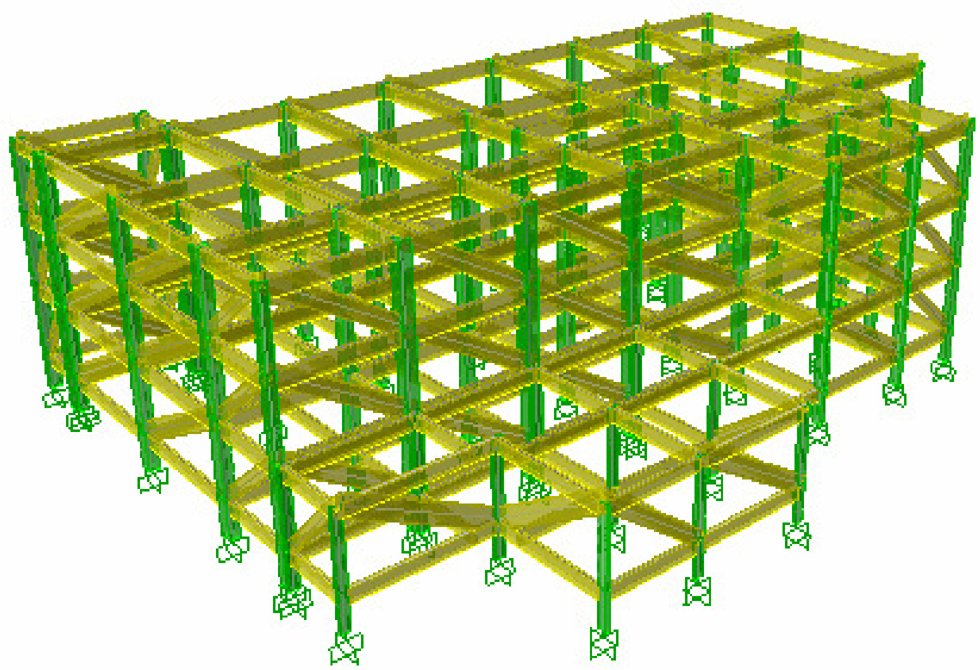


Figure 5. Rendering of linear ETABS model of the building

The team also conducted the other checks mandated in ASCE 31 for Tier 2 analysis based on the Tier 1 Checklist results. Despite using a modified FEMA 310 Tier 1 Checklist there was enough correspondence between items in the ASCE 31 Tier 1 Checklist and the modified FEMA 310 checklist to use ASCE 31's Tier 2 checks directly. For this building, the required Tier 2 checks were for torsion irregularity (shown in Table 1), mass irregularity (shown in Table 2), storey drift (shown in Table 3) and soft storey (shown in Table 4).

Table 1. Torsion irregularity check

Story	Diaphragm	XCM	YCM	XCR	YCR	% diff X (allow 20%)	% diff Y (allow 20%)
RF	D1	832.658	762.646	747.504	768.862	5.1	0.7
2F	D1	814.612	765.612	734.399	754.523	4.8	1.3
1F	D1	745.342	655.473	712.054	729.157	2.0	8.4
GF	D1	695.087	611.771	727.197	747.252	1.9	15.5

XCM = centre of mass in X direction, YCM = centre of mass in Y direction, XCR = centre of rigidity in X direction, YCR = centre of rigidity in Y direction

Table 1 shows that there is no torsion irregularity per ASCE 31, because the difference between centre of mass and centre of rigidity is less than 20% for each storey.

Table 2. Mass Irregularity

		% diff in Mass (50% allow)	
		% difference compare to	
Story	MassX	Above storey	Below storey
RF	6.1926	---	8
2F	6.7341	9	21
1F	8.4826	26	149
GF	3.4121	60	---

Table 2 shows that the building has mass irregularity at ground floor level.

Table 3. Storey drift check

Story	Etab Drift X	Code Modified Drift	Etab Drift Y	Code Modified Drift
	Δ_S	Δ_M	Δ_S	Δ_M
RF	0.001877	0.00723	0.002926	0.01127
2F	0.00311	0.01197	0.004783	0.01841
1F	0.003803	0.01464	0.005791	0.02230
GF	0.001957	0.00753	0.002967	0.01142

Table 3 shows that the building exceeds the allowable storey drift value of 0.02 in the y direction at first floor level.

Table 4. Soft storey check

Story	Load	storey force kips	Total Displacement inches	Stiffness kip/in	% diff in K (30% allow) % difference compare to Above storey	Below storey
RF	EX	710	1.3315	533.23	----	3.4
2F	EX	551	1.0686	515.63	3.3	22.8
1F	EX	417	0.6245	667.73	29.5	50.8
GF	EX	56	0.1265	442.69	33.7	----

Story	Load	storey force kips	Total Displacement inches	Stiffness kip/in	% diff in K (30% allow) % difference compare to Above storey	Below storey
RF	EY	710	1.7306	410.26	----	2.6
2F	EY	551	1.3775	400.00	2.5	23.3
1F	EY	417	0.7992	521.77	30.4	43.8
GF	EY	56	0.1543	362.93	30.4	----

Table 4 shows that the building has soft storey at ground floor level both in x and y directions.

Detailed Evaluations of Existing Building

Through linear static analysis of this building, the checks for building system (mass irregularities, torsion etc.) in tier 1 analysis which were assumed non-compliant through visual inspection were confirmed by tier 2 analysis results. In addition it was also observed that many columns had DCR > 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 are evaluated and manually entered into the 3-D model.

Analytical Models

The building was modeled using discrete plastic hinge elements (i.e., a lumped plasticity model) in locations expected to experience nonlinear behavior, such as beam and column ends and the midpoint of compression struts. ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings*, was adopted to compute the plastic hinge values for compressive struts, beams and columns. Infill walls were modeled using equivalent compression struts defined using procedure in Section 7.5.2 of FEMA 356. The hinge properties for compression 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 used, respectively. ETABS Nonlinear (version 9.7.0) was used to create the models and perform the pushover analysis. Table 5 gives the geometric and material properties used in the model.

Loading and Performance Criteria

Table 5 shows the ETABS input values for gravity and earthquake loading, as well as key assumptions. The UBC-97 was used for the seismic demands. As mentioned in the Seismic Hazard section, the building was evaluated for Zone 4 seismic loads due to the current uncertainty in the seismic hazard. For the pushover analysis, the team used restart using secant stiffness for member unloading method with P-Delta effects for geometric nonlinearity. A life safety performance criterion was selected for the study building.

Table 5. Loads and modelling parameters

Loads:	Slab loads transferred to beam were manually calculated and applied to each of the beams in the 3-D model.	
Dead load	Self wt of frame + 6" thick slab + 2" thick finishes + 50psf wall load	
Live load	100psf on floor and 30psf on roof	
Earthquake load:		
Z	0.4g	
R	5.5	
C_a	$0.4N_a$ (Ref: Table 16-Q (UBC 97)) $N_a = 1.0$	
C_v	$0.4N_v$ (Ref: Table 16-R (UBC 97) $N_v = 1.0$	
Soil type	S_B (Ref: Table 16-J UBC-97)	
Geometric properties	Typical Beam size	Width = 12 Depth = 24
	Typical Column size	Width = 20 in Depth = 20 in Height = 12 ft
	Ordinary Strut (for modeling infill)	Width = 6in Depth = 30 in
Material properties	f'c = 3000 psi for beam and column fc' = 300 psi for concrete block infill ordinary strut E_{con} = 3144 ksi for beam and column E_{mas} = 214.5 ksi	

Analysis Results

Figure 6 shows the pushover load-deformation curve. The curve bends and becomes jagged as the various structural members begin to yield and undergo plastic deformation. In Figure 7, the pushover curve, a measure of the building's capacity, is converted into a capacity spectrum and compared with the estimated demand using the capacity spectrum method. This figure shows the performance level where demand and capacity spectra intersect each other at that point where it is necessary to see the condition of the structure, and whether it is fulfilling the demand or not. This point is called the *performance point*.

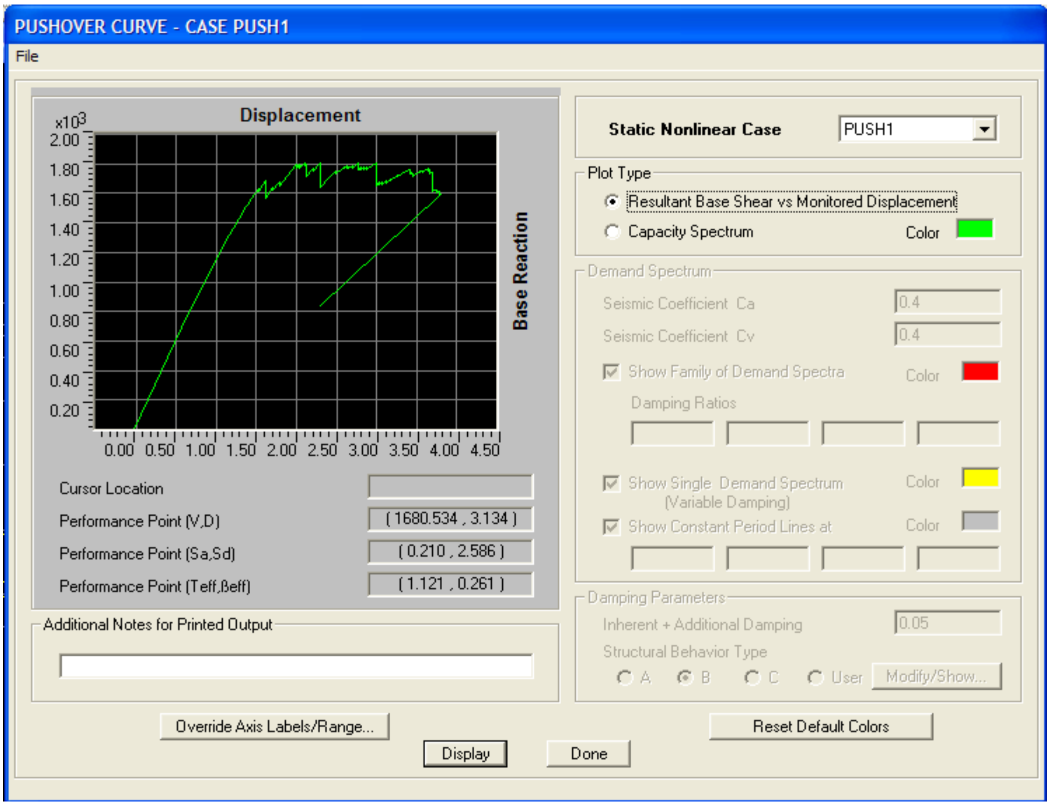


Figure 6. Pushover load-deformation curve

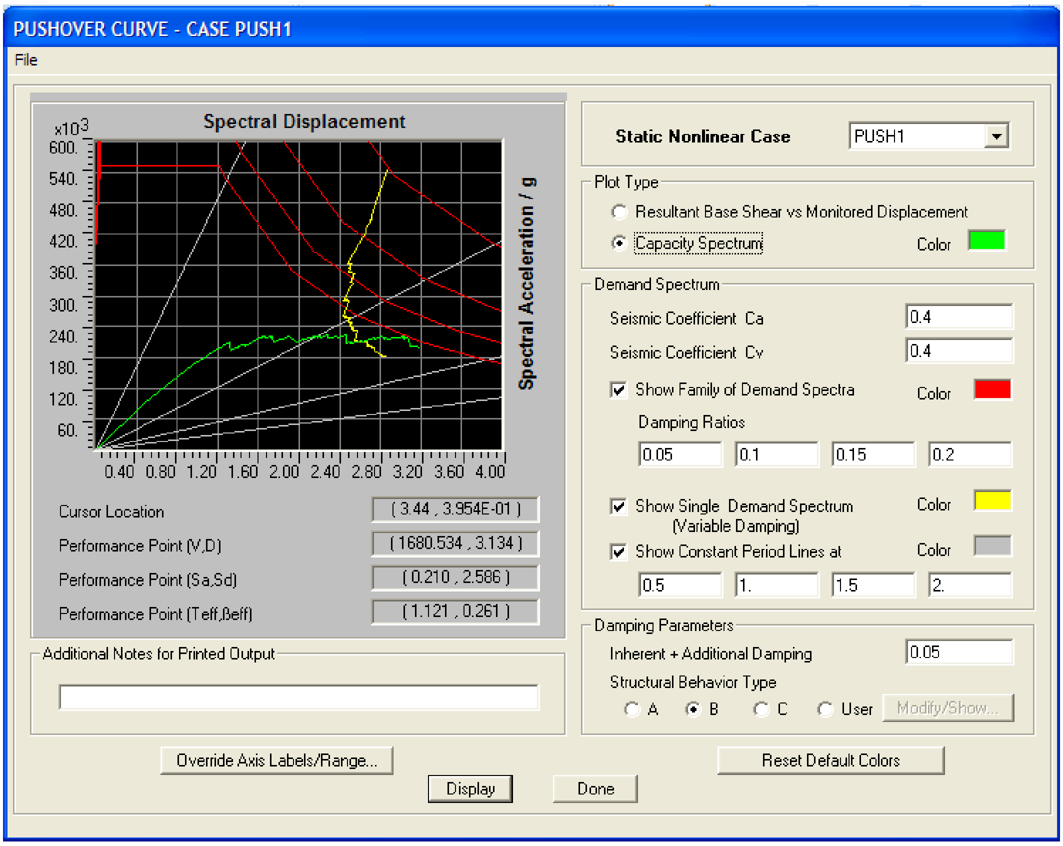


Figure 7. Performance level

Figure 8 shows the force deformation relation for the plastic hinges, and shows how the acceptance criteria and Immediate Occupancy, Life Safety and Collapse Prevention performance states are defined. Other points on the curve represent behavior states: B means yielding has occurred, C is point just before major strength loss, D is the point just after major strength loss, and E represents complete failure. Figure 9 shows the state of some of the nonlinear plastic hinges compared to the acceptance criteria at the performance point. The plot at the left of Figure 9 has a legend with the colors at the bottom. In these plots, B, IO, LS, CP, C, D, and E correspond to points on the force-deformation curve for the hinge shown in Figure 8. Appendix C contains the remaining pushover analysis results for the existing building.

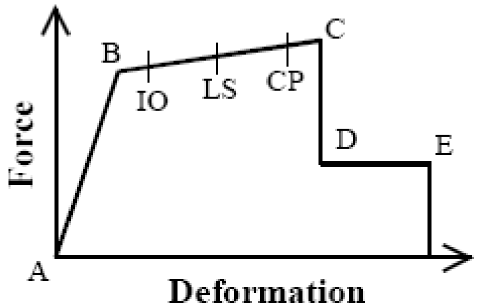


Figure 8. Force-deformation curve for hinges (reprinted from FEMA 356, the precursor to ASCE/SEI 41-06) showing the definition of acceptance criteria and performance states

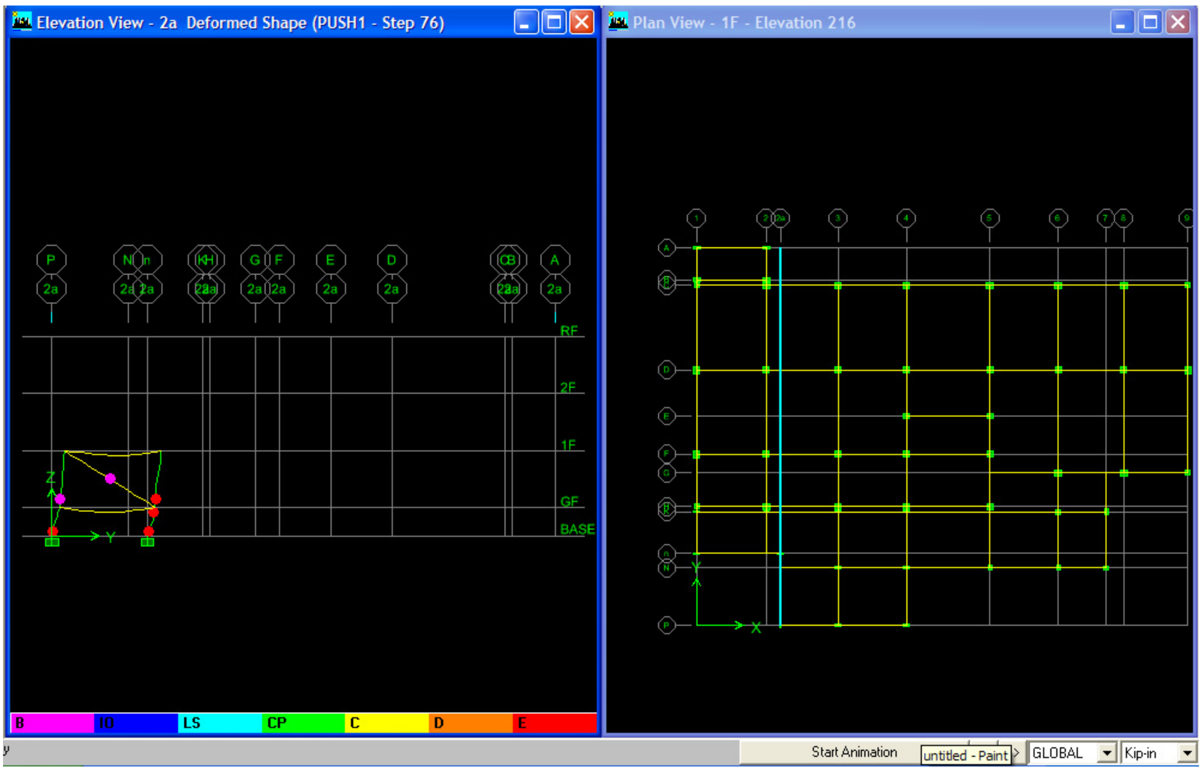


Figure 9. Hinge deformation vs. acceptance criteria

The columns in the small single storey addition at the front of the library are failing, as indicated by the red circles in Figure 9. This shows that retrofitting is needed to achieve stability and to achieve the desired life safety performance level.

Retrofit Solution

Conceptual Solutions Considered

In order to prevent column failure in the small additional block and to prevent pounding with the main building, the case study team decided to stitch the small additional block and the emergency exit stair to the main building. They also considered reinforced two options for adding deformation capacity and strength in the ground storey: infill panels reinforced by a shotcrete to create a shear wall, and a combination of reinforced infill panels and wrapped columns. The team determined that a walls-only solution was preferable, and then investigated two configurations of reinforced infill panels. The first option had reinforced panels in the ground storey only, and not below ground. The second and preferred option, shown in Figure 10, placed panels in more optimal locations for constructability and also provided panels below the ground level, in between the base and ground floor at the outer periphery.

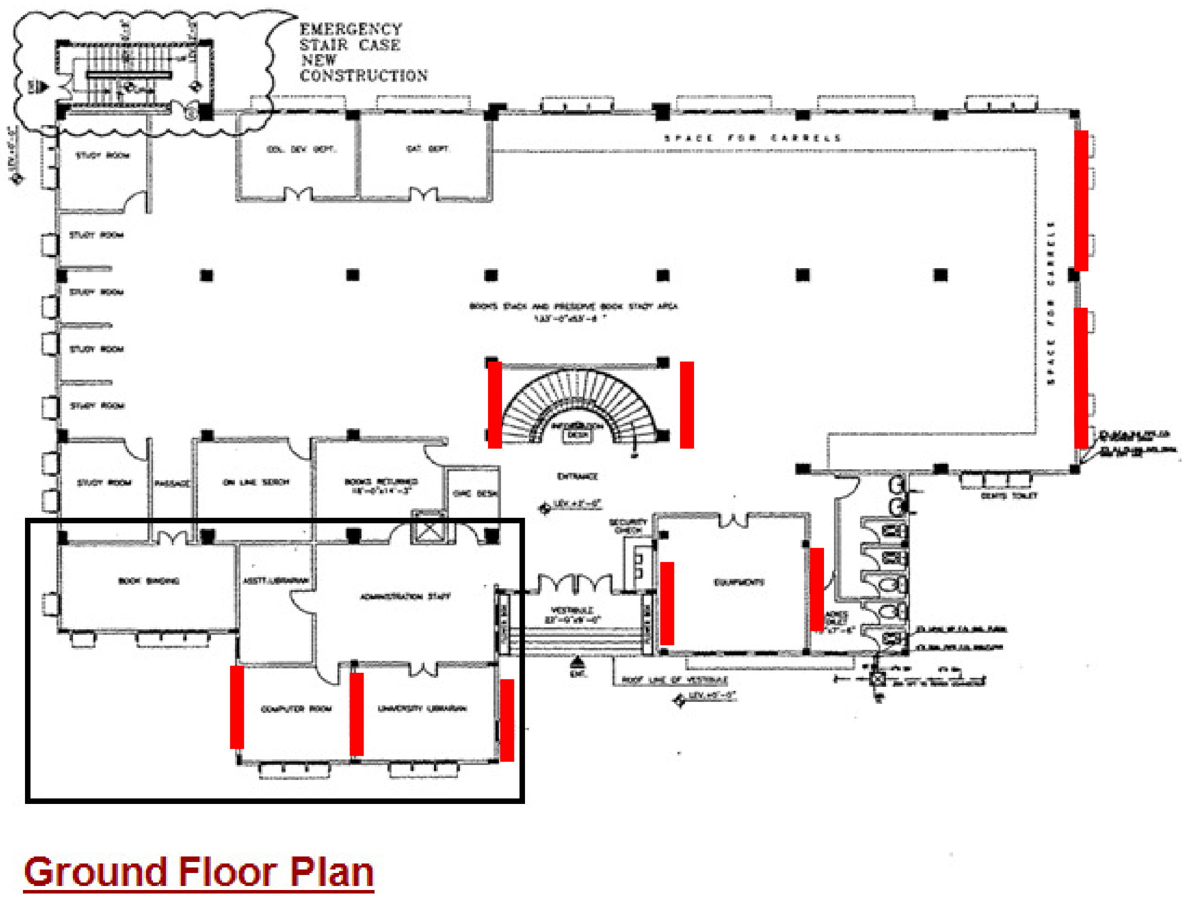


Figure 10. Retrofit option 2 (selected retrofit solution)

Retrofit Analysis Results

Figure 11 shows a comparison of pushover analysis results for the two retrofit options versus the existing building. The capacity and demand spectra for retrofit option 2 are shown in Figure 12. Retrofit option 2 provides better performance than option 1, and rectifies the column failures experienced by the existing building, as Figure 13 shows.

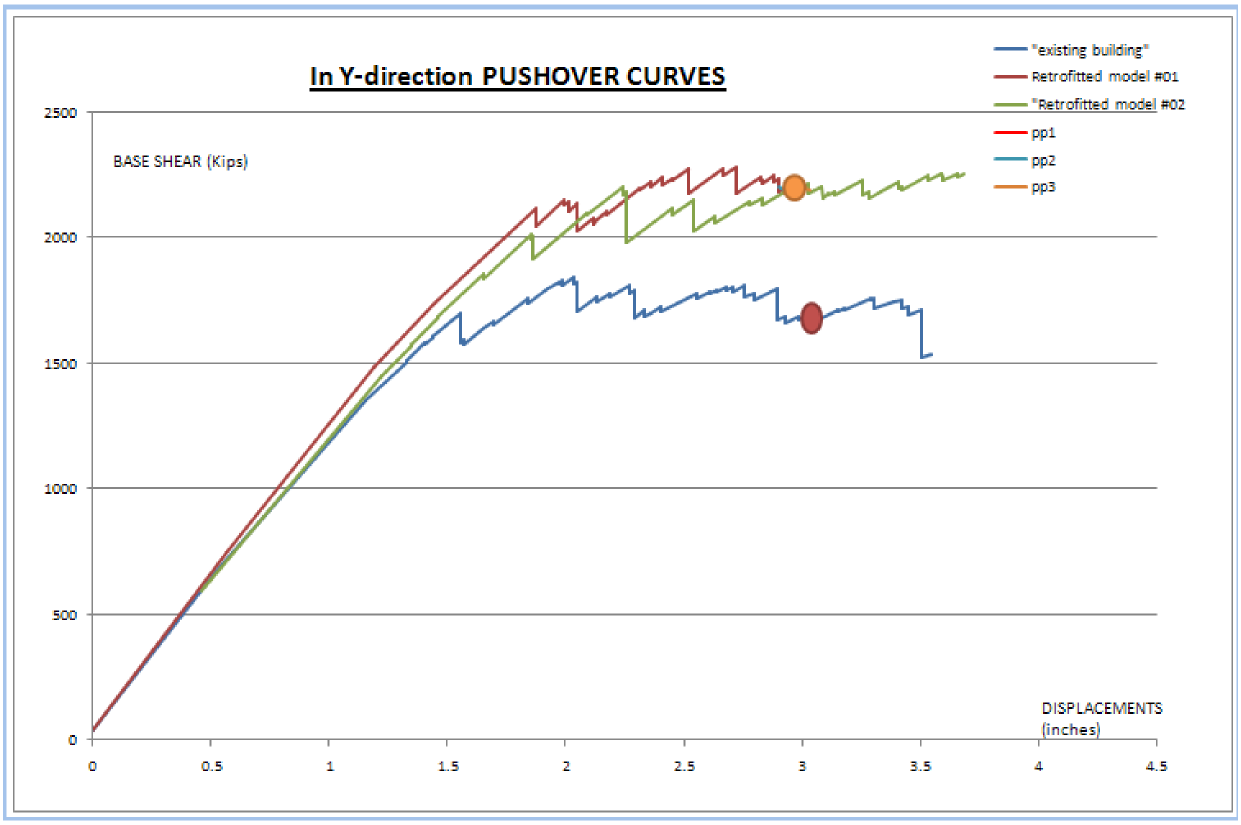


Figure 11. Comparison of pushover curves; Retrofitted model #01 is Option 1, Retrofitted model #02 is Option 2

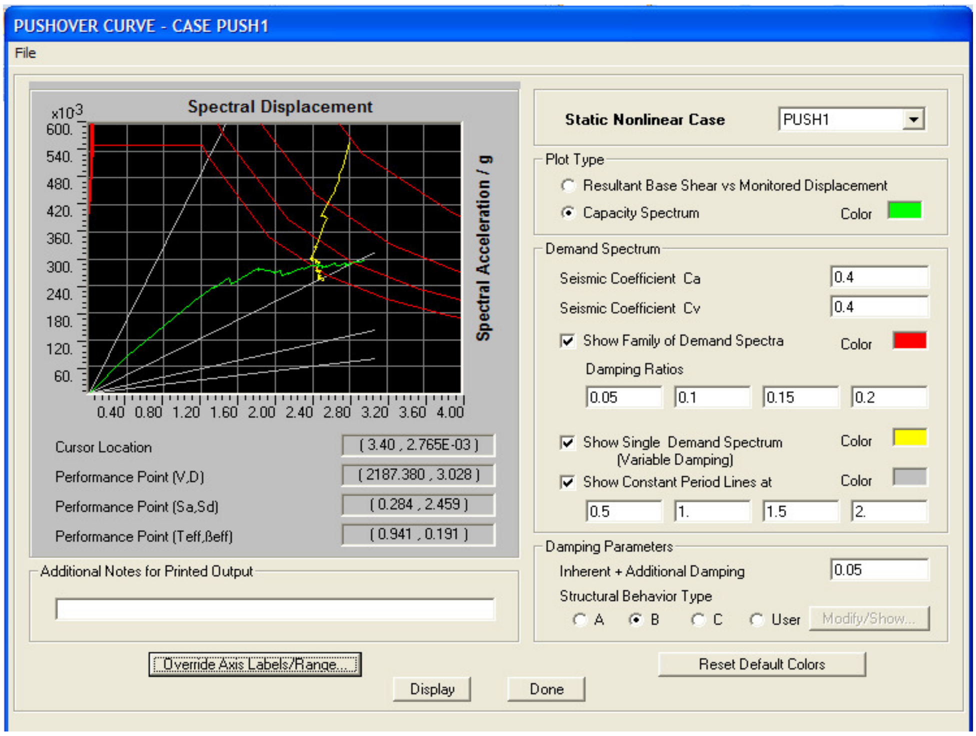


Figure 12. Performance level for retrofitted building

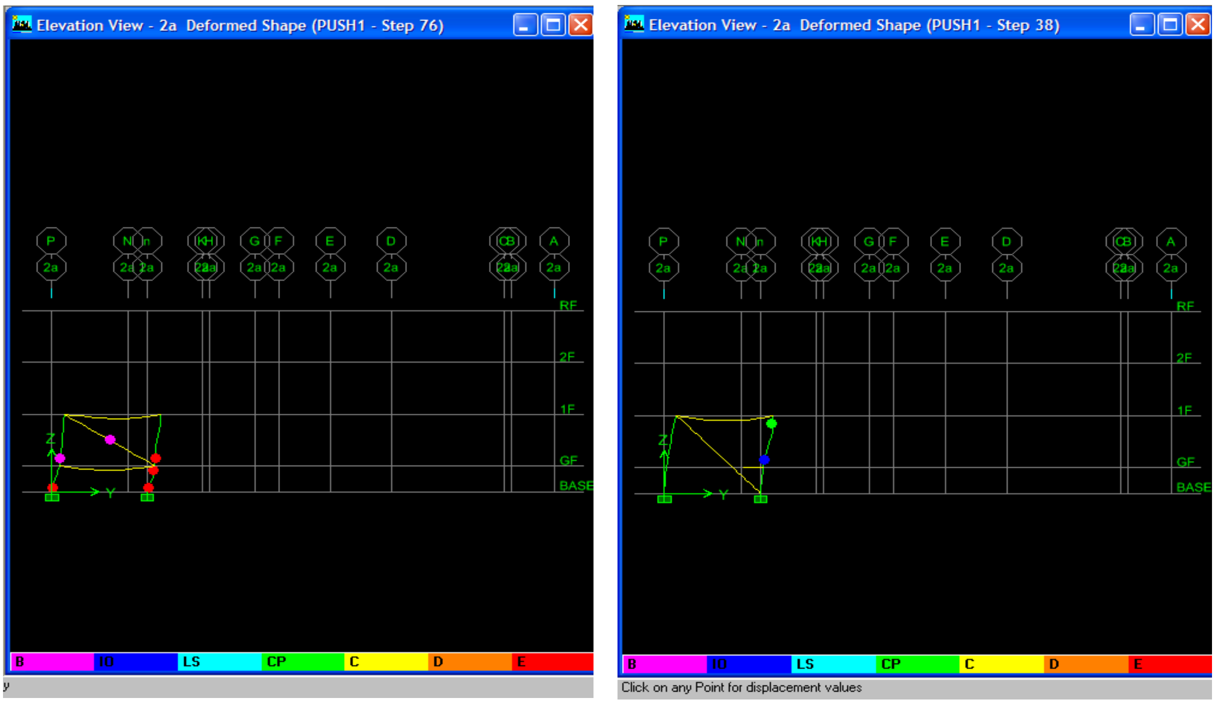


Figure 13. Hinge deformation vs. acceptance criteria before retrofit (left) after retrofit (right)

Recommended Retrofit Solution

The case study team selected option 2, shown in Figure 10, as the recommended retrofit solution. It is simpler to construct than option 1 because it does not require construction near the toilets which would have been difficult due to the plumbing and fixtures.

Design and Detailing of Retrofit Solution

Engineering drawings containing selected details of the retrofit solution are shown below. Appendix E contains the full set of retrofit drawings. **Figure 14** shows the locations of the reinforced infill panels in plan, and Figure 15 shows details of the reinforced infill panels. New tie beams were provided beneath the reinforced panels; Figure 16 shows a typical detail. Stitching between the buildings was accomplished by attaching tie plates with drilled anchor bolts across the joint. A through-bolt detail, where holes are drilled through the beams just below the slab using large plate washers to secure the through-bolts (anchor rods) on either side of the beam, was used to ensure that the anchor rods are strong enough. Figure 17 shows the stitching details.

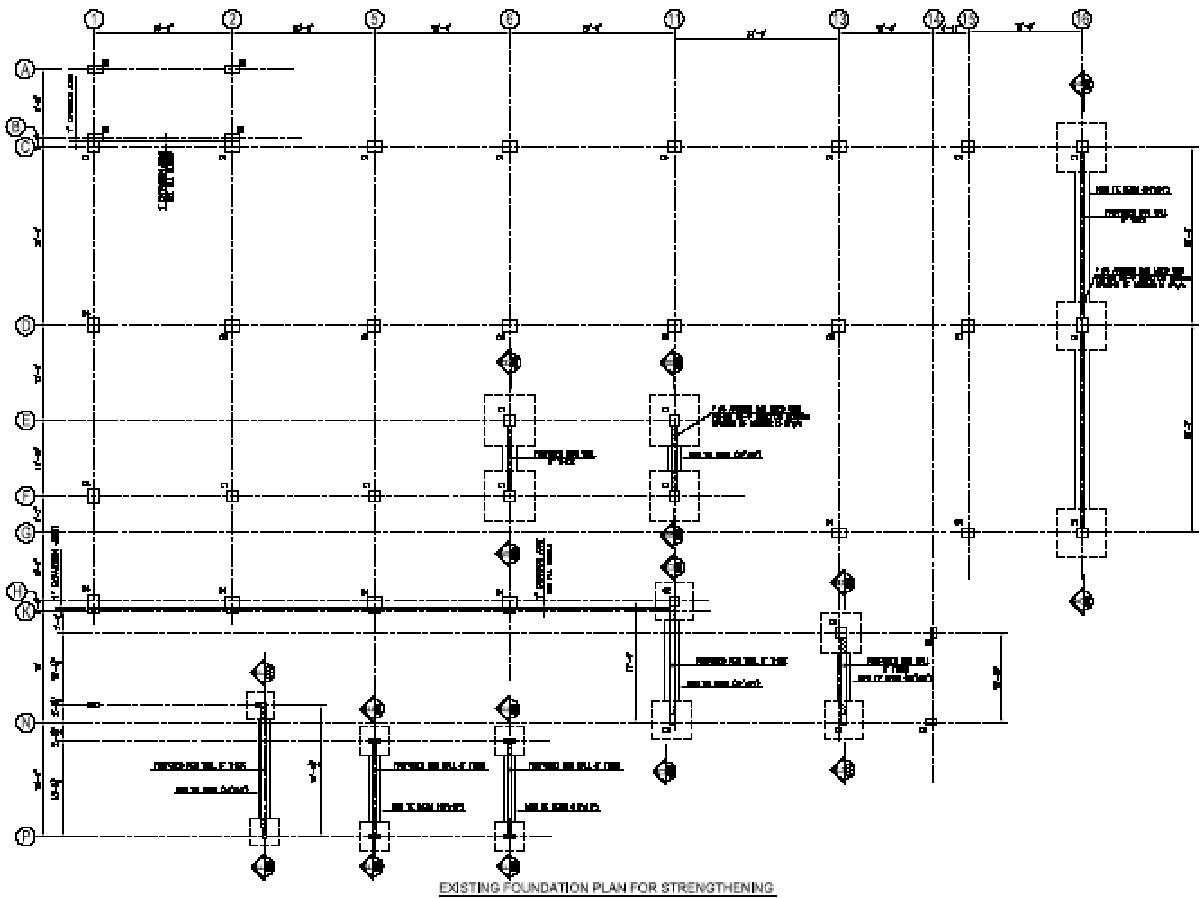


Figure 14. Locations of retrofitted infill walls in plan

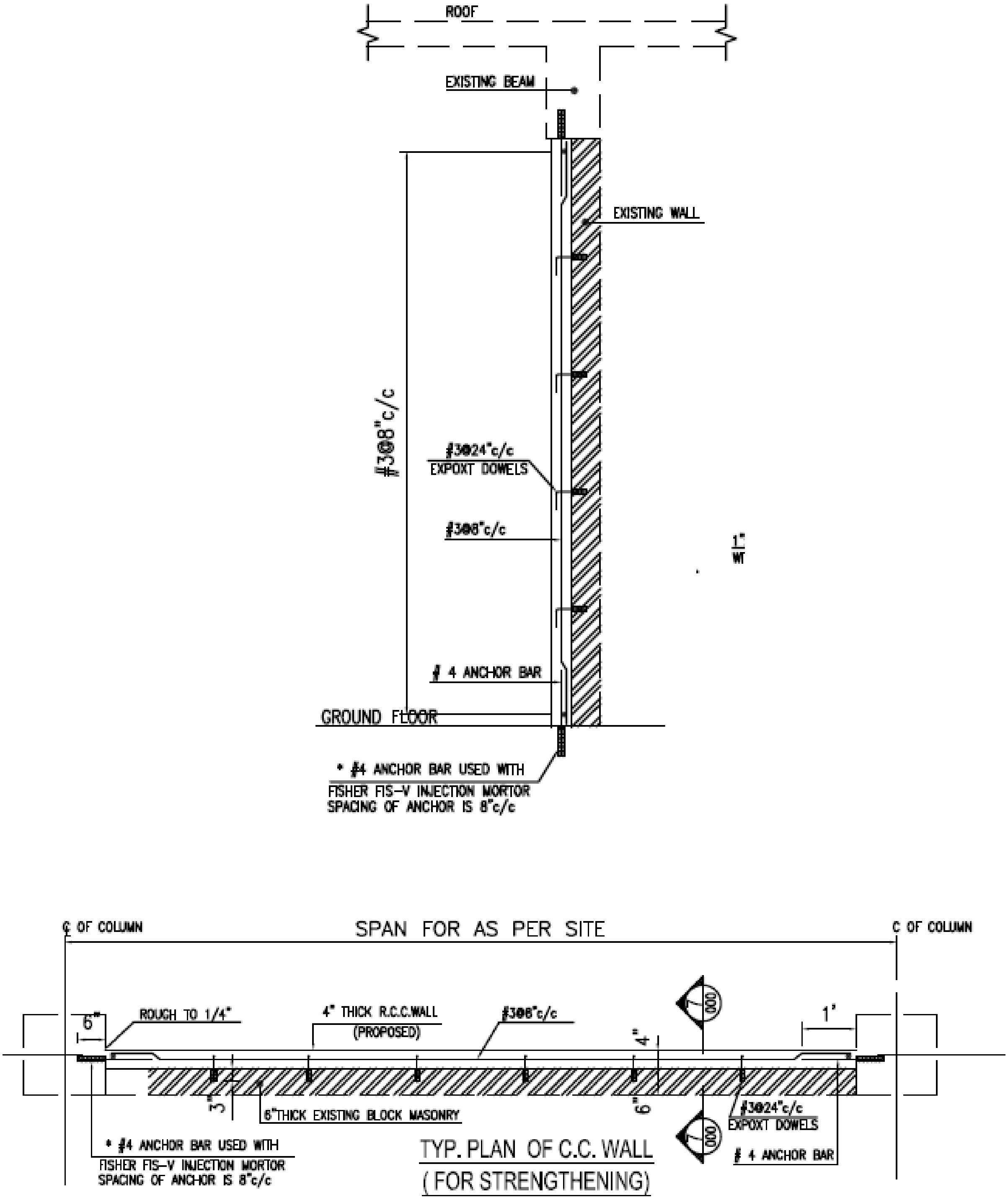


Figure 15. Detail for strengthened infill panels

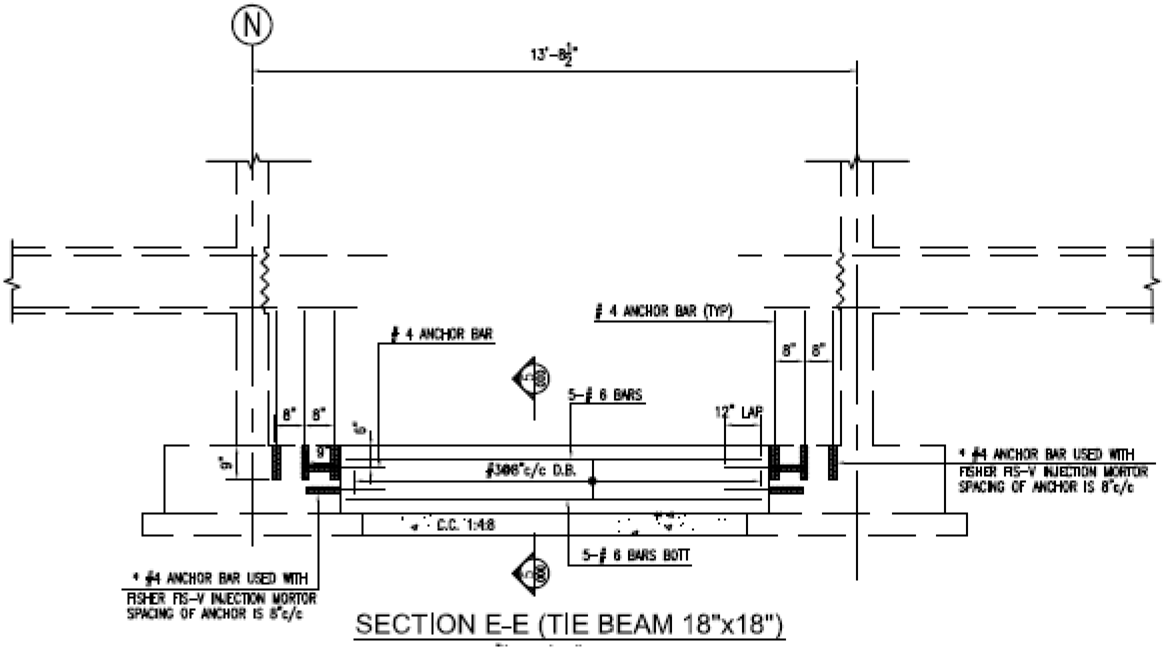


Figure 16. Detail for new tie beam

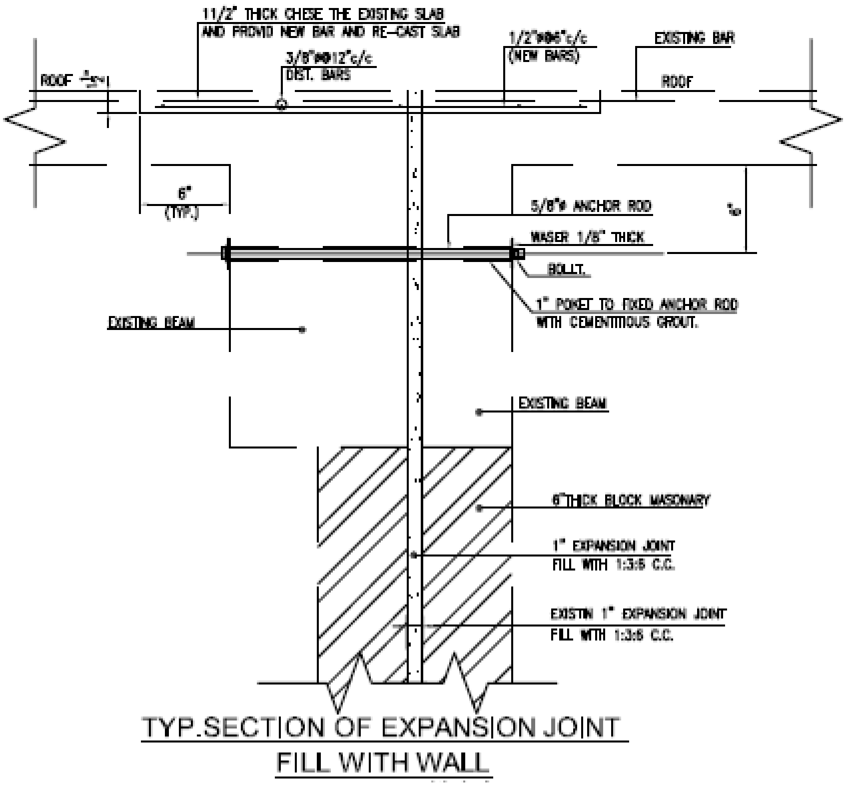


Figure 17. Details for stitching

Observations and Future Work

In this building, the infill walls were located at the periphery of the building and did not contribute as significantly to the building behavior as in other buildings investigated during the project. Also, the retrofit solution was strongly influenced by architectural and functional considerations. The lack of infill walls and the large number of windows necessitated careful selection of locations for reinforced infill panels.

Appendix A: Tier 1 Checklists

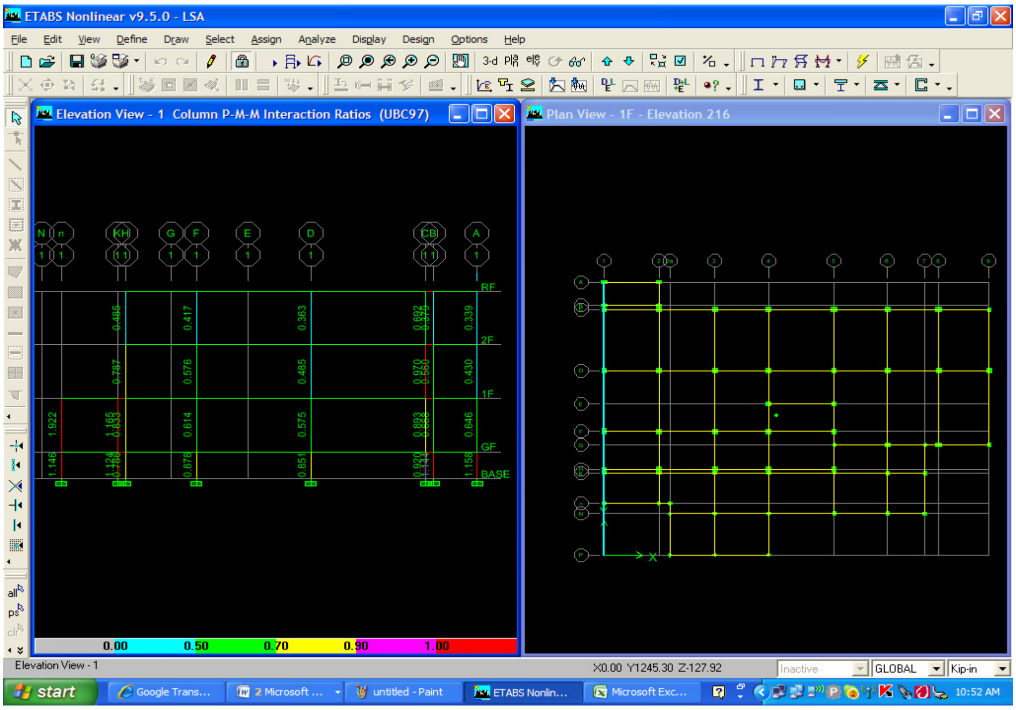
BUILDING SYSTEM	
Load Path	C
Adjacent Building	C
Mezzanine	NA
Weak Story	C
Soft Story	C
Geometry	C
Vertical Discontinuities	C
Mass Irregular	C
Torsion	NC
Deterioration	C
Post Tensioning Anchors	NA

GEOLOGIC SITE HAZARDS AND FOUNDATION CHECKLIST	
Liquefaction	C
Slope Failure	N/A
Surface Fault rupture	C
Foundation Performance	C
Deterioration	C
Pole Foundation	N/A
Over turning	C
Ties between Foundation element	C
Deep foundation	N/A
Sloping Sites	N/A

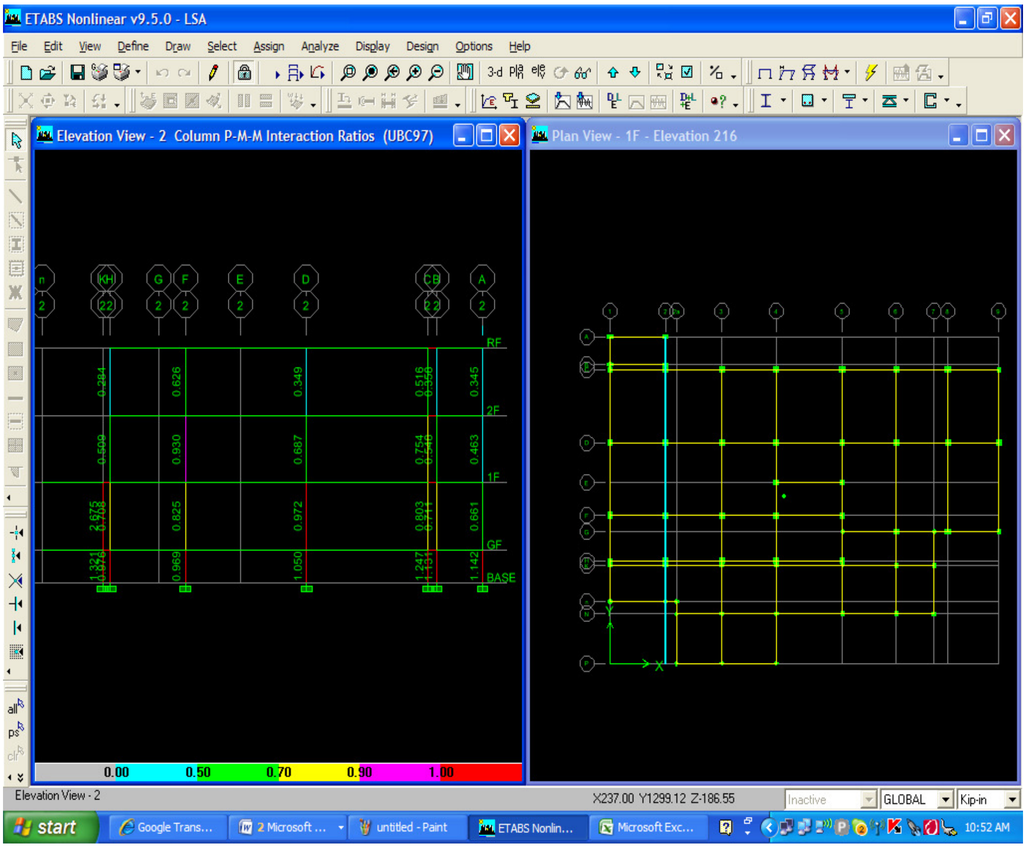
BASIC NONSTRUCTURAL COMPONENT CHECKLIST	
Partitions	C
Ceiling Systems	N/A
Light Fixtures	C
Cladding and Glazing	N/A
Masonry Veneer	N/A
Parapets	NC
Canopies	C
Masonry Chimneys	N/A
Stair's Urm Walls	N/A
Stair Details	NC
Tall Narrow Contents	NC
Emergency Power	NC
Hazardous Material Equipment	C

BASIC NONSTRUCTURAL COMPONENT CHECKLIST *(Cont.)*	
Deterioration	C
Attached Equipment	NC
Fire Suppression Piping	N/A
Flexible Couplings	NC
Toxic Substances	C

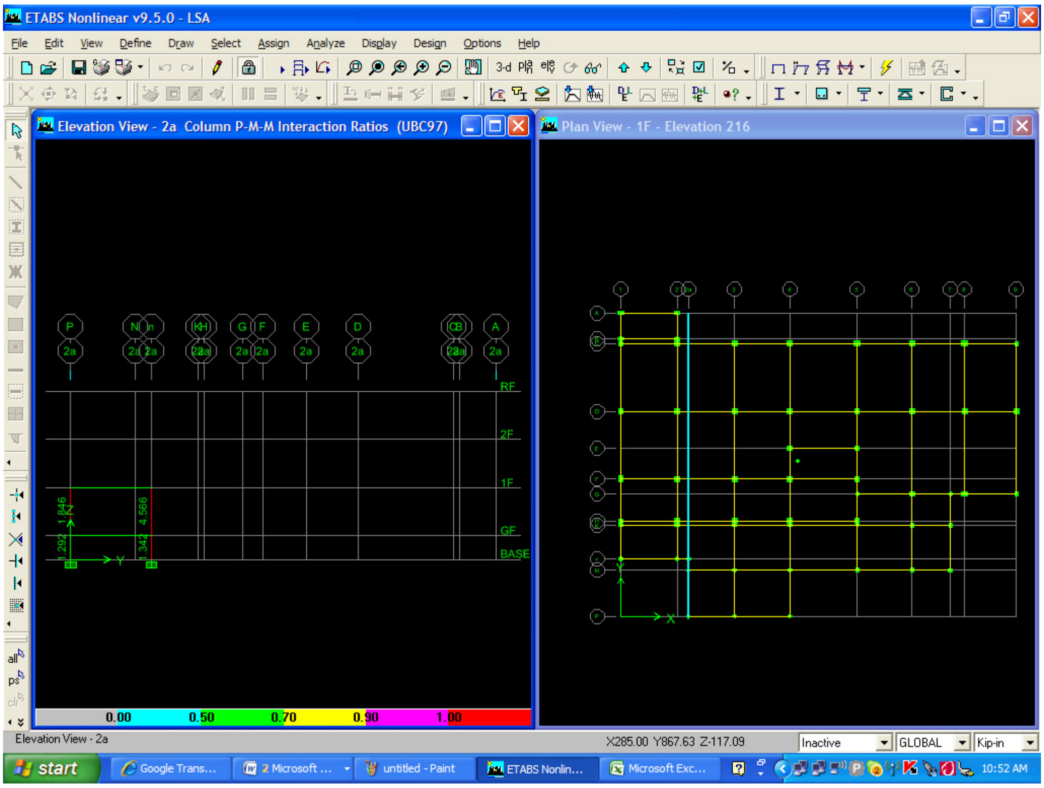
Appendix B: Linear Analysis (Tier 2) Results



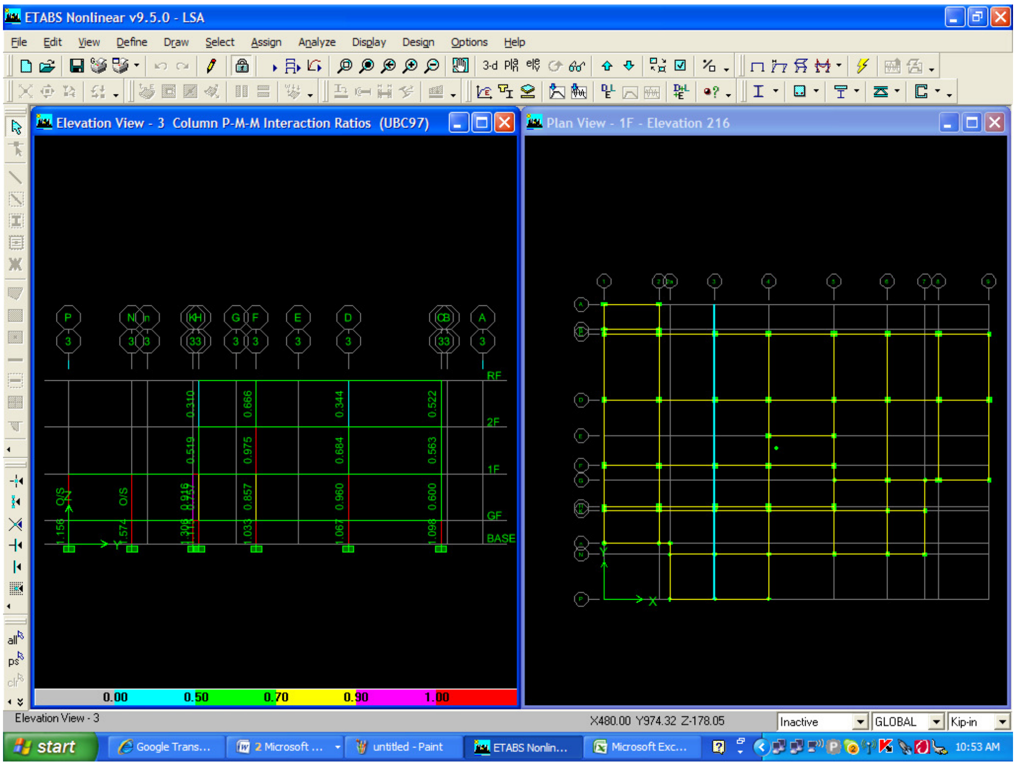
Demand/Capacity Ratios for Frame at Grid-1



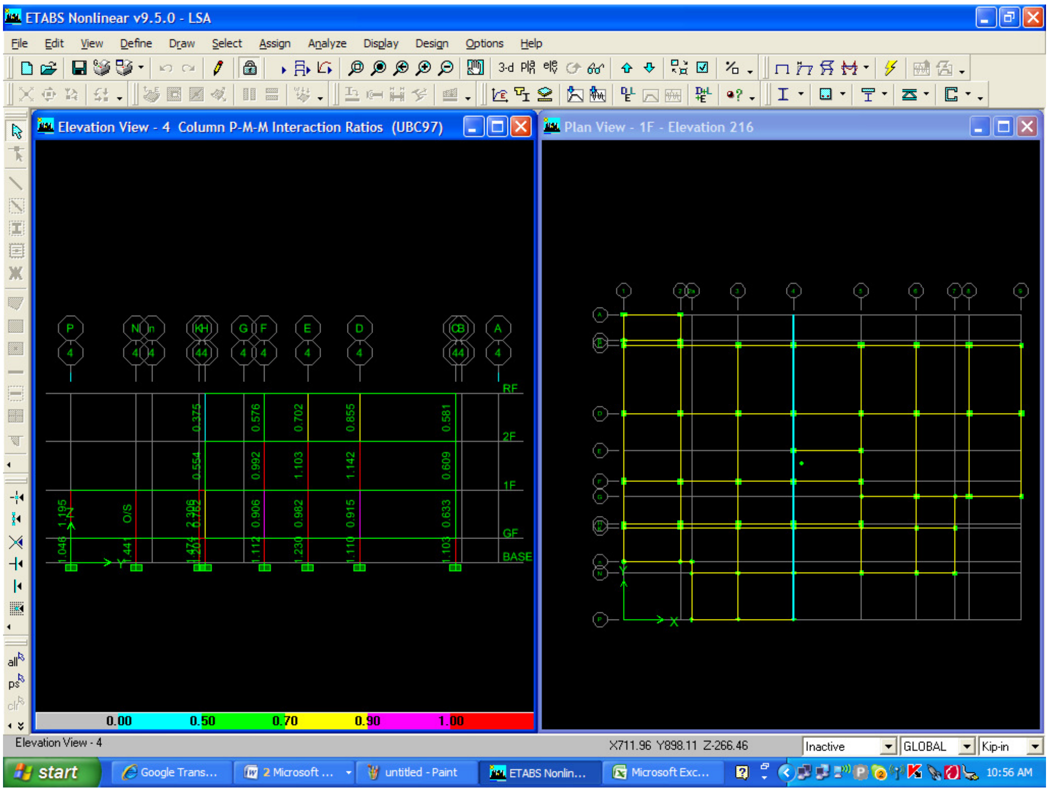
Demand/Capacity Ratios for Frame at Grid-2



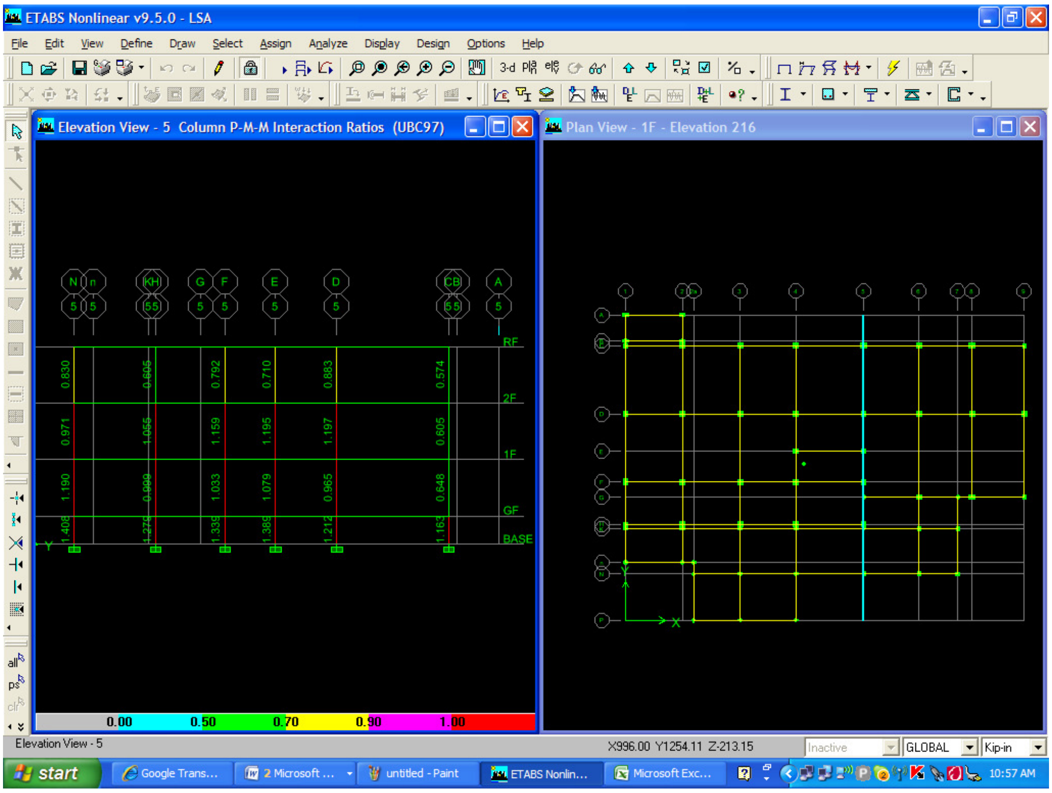
Demand/Capacity Ratios for Frame at Grid-2a



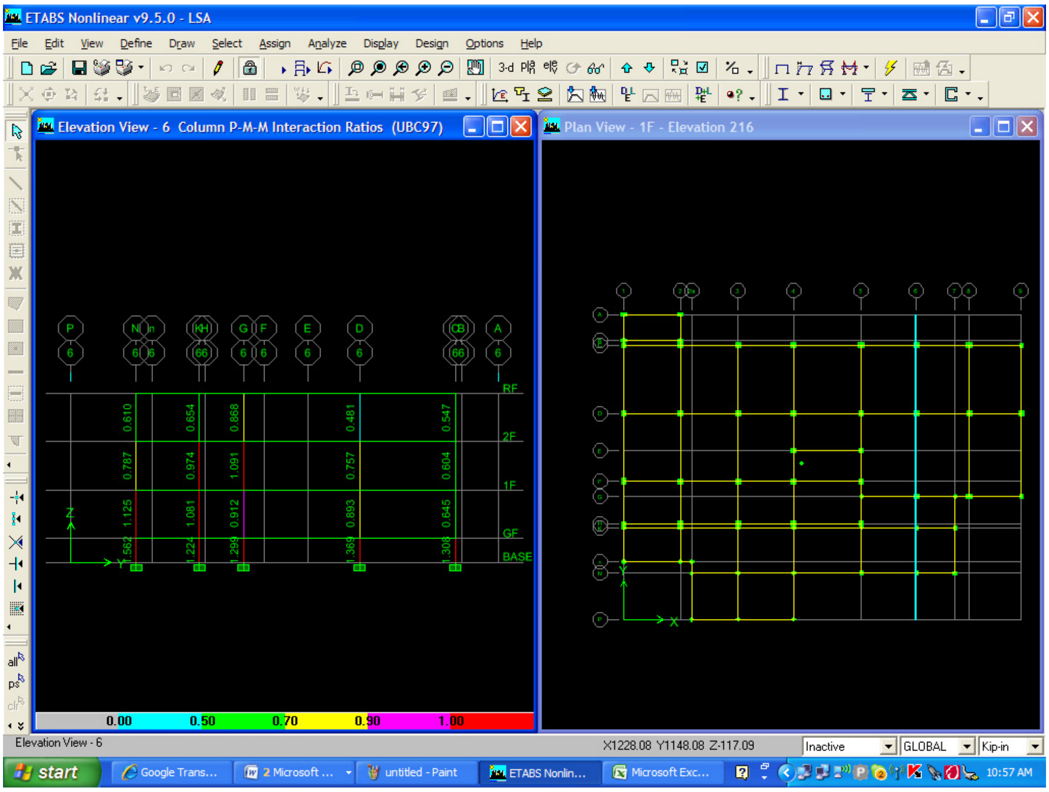
Demand/Capacity Ratios for Frame at Grid-3



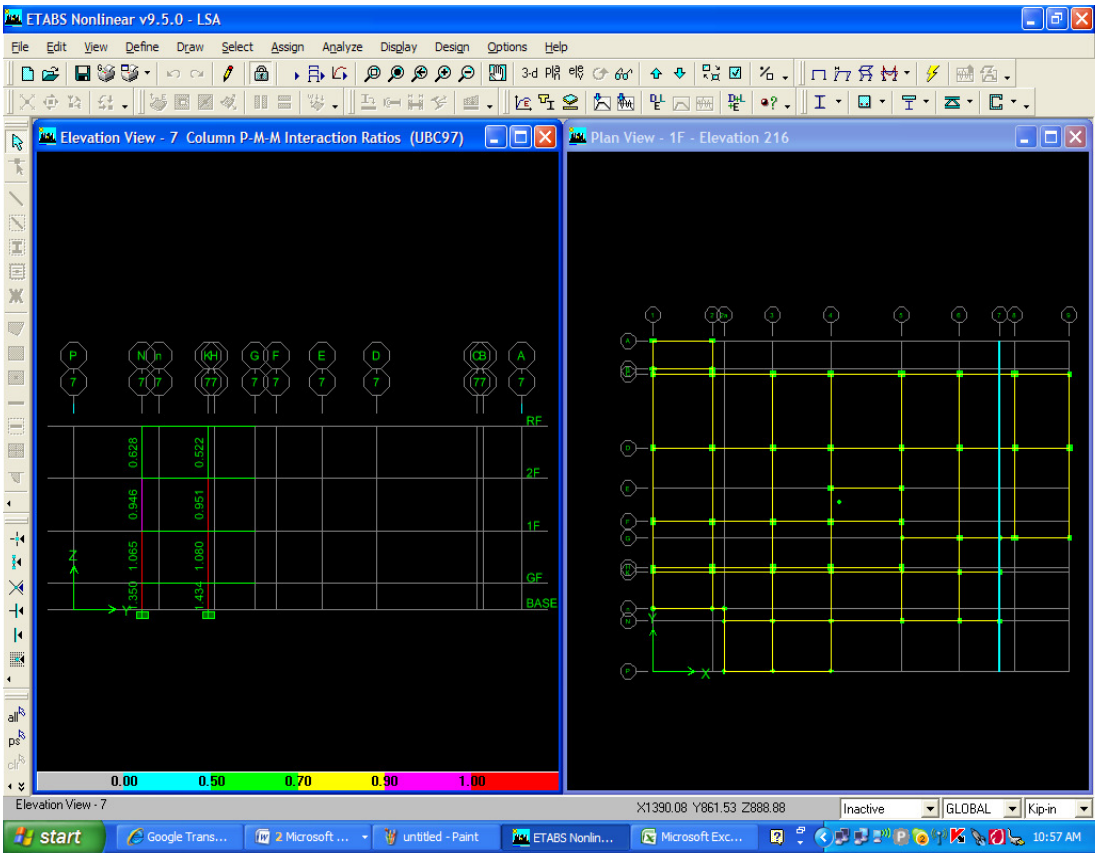
Demand/Capacity Ratios for Frame at Grid-4



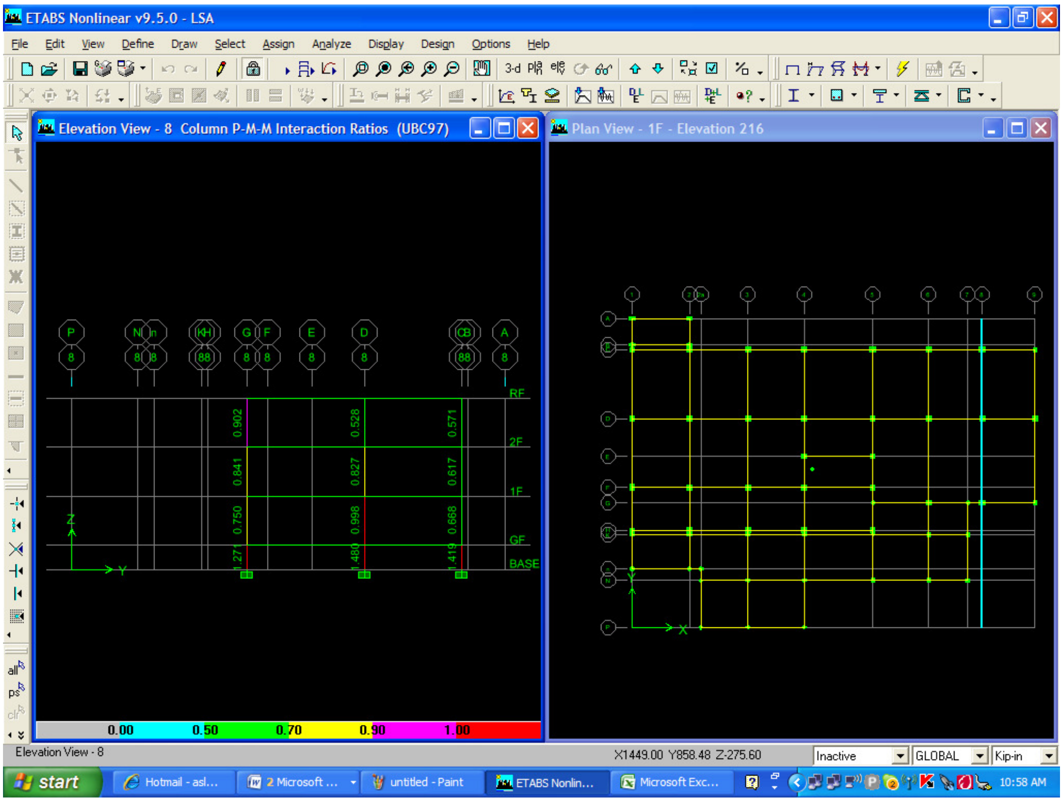
Demand/Capacity Ratios for Frame at Grid-5



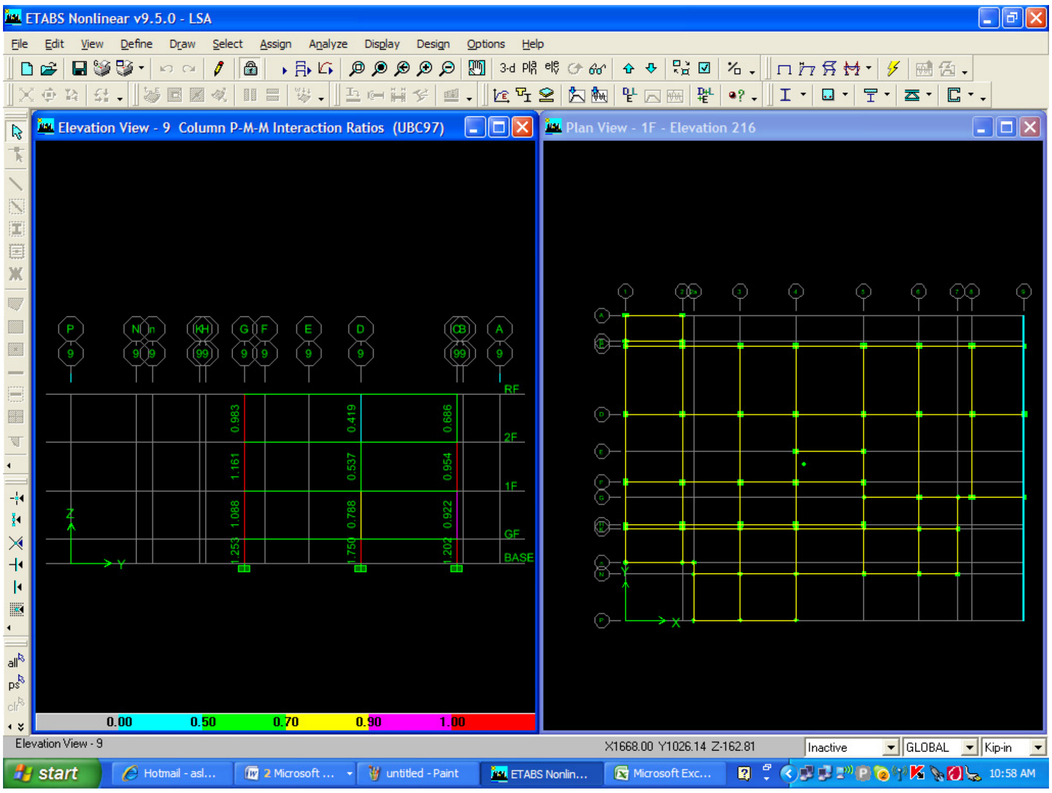
Demand/Capacity Ratios for Frame at Grid-6



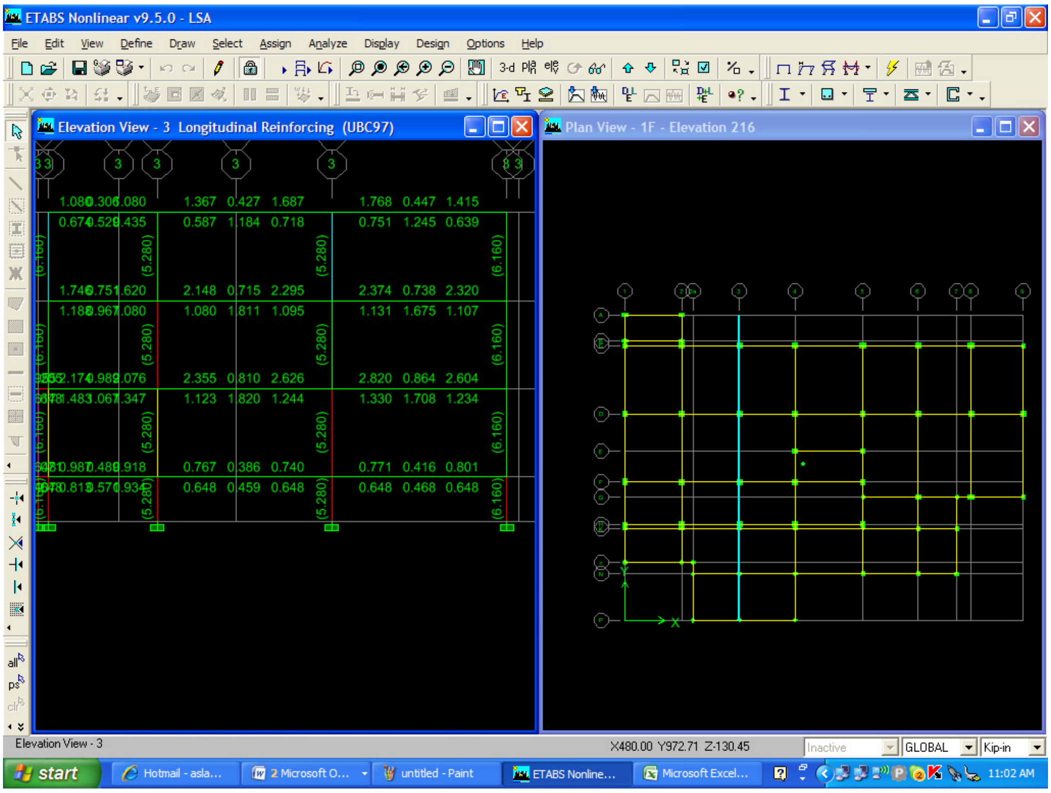
Demand/Capacity Ratios for Frame at Grid-7



Demand/Capacity Ratios for Frame at Grid-8



Demand/Capacity Ratios for Frame at Grid-9



Demand/Capacity Ratios for Beams

At First Floor Level:

Required reinforcement at support = 2.82 in2

Required reinforcement at mid span = 1.71 in2

Provided reinforcement at support = 1.5 in2

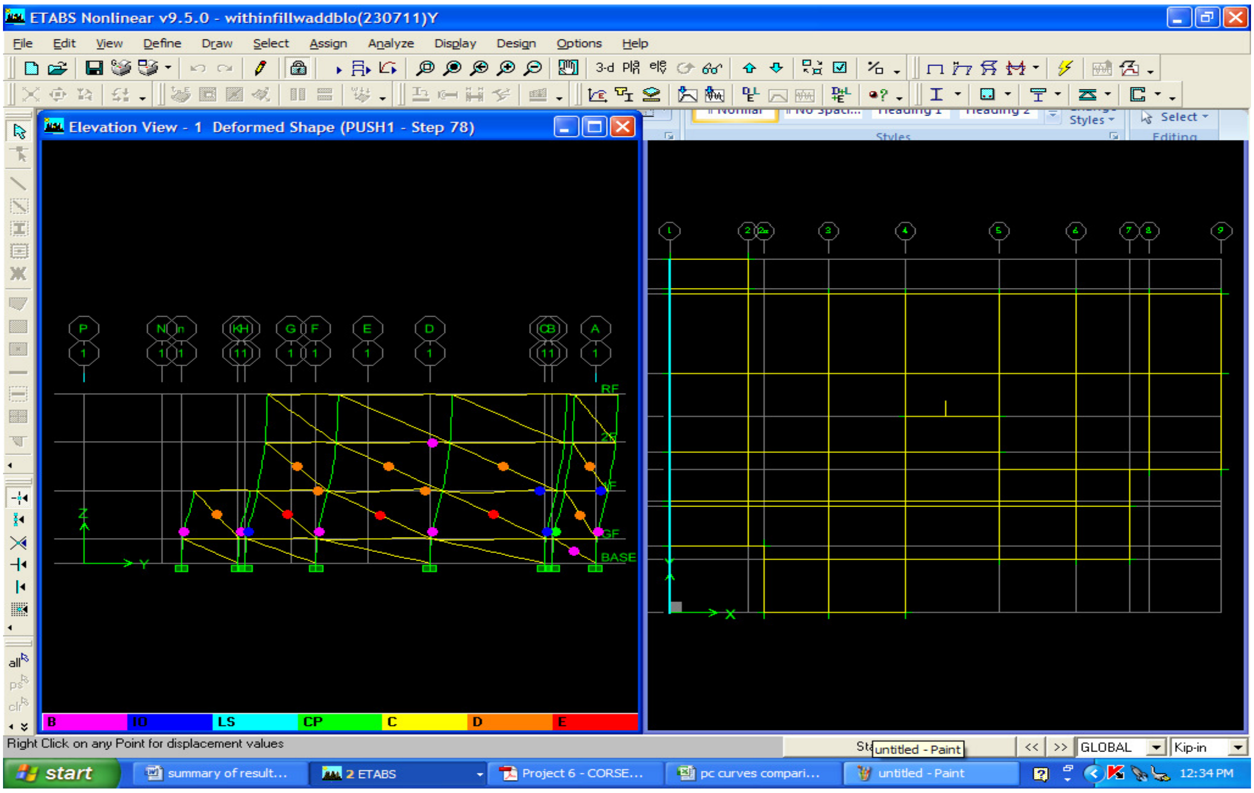
Provided reinforcement at mid span = 1.5 in2

Demand capacity ratio at support = 2.82/1.5 = 1.88

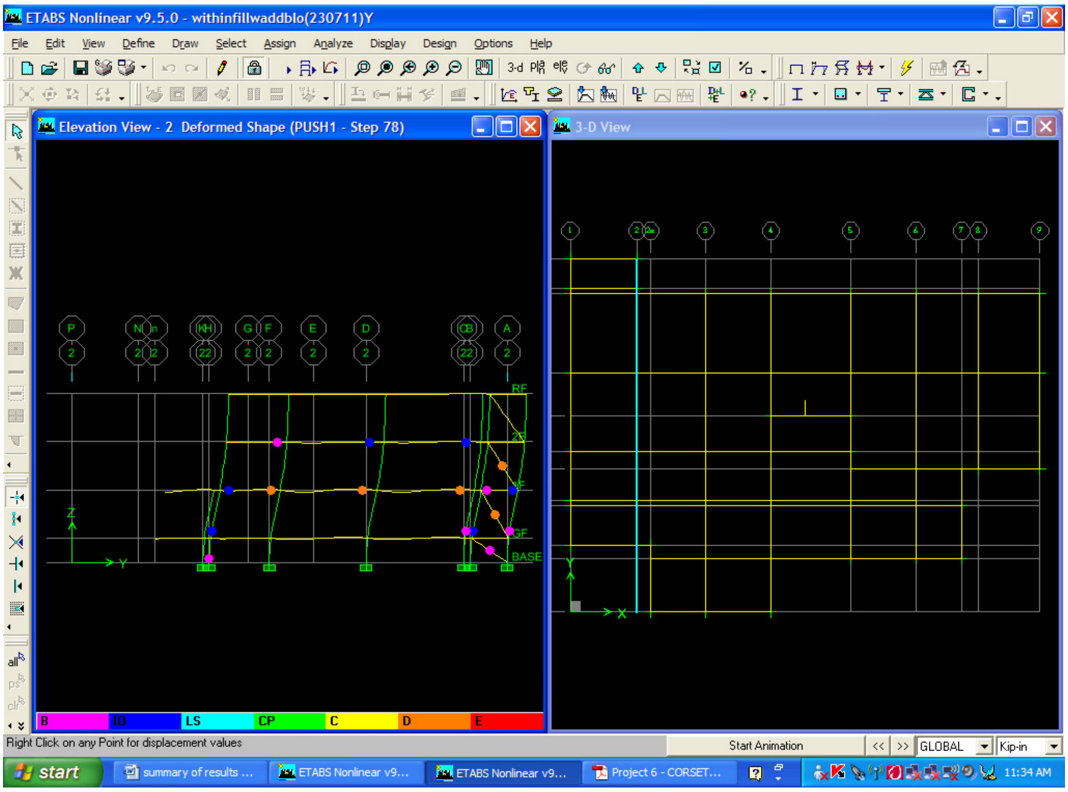
Demand capacity ratio at support = 1.71/1.5 = 1.14

Appendix C: Nonlinear Analysis Results for Existing Building

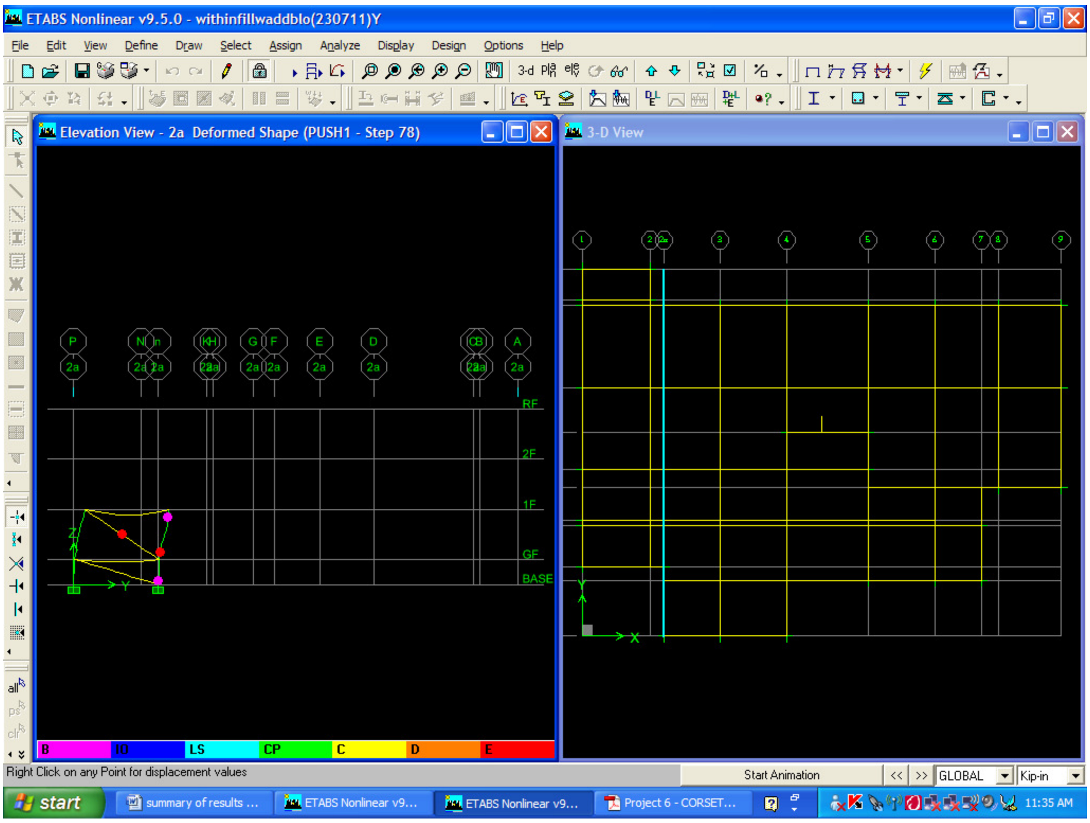
This appendix contains plots of the deformed shapes at the performance point. The state of the hinges versus the acceptance criteria are indicated by the colour of the circles. In these plots, B, IO, LS, CP, C, D, and E correspond to points on the backbone curve for the hinge that represent certain performance levels or behavior states in ASCE 41-06, as defined in Figure 8.



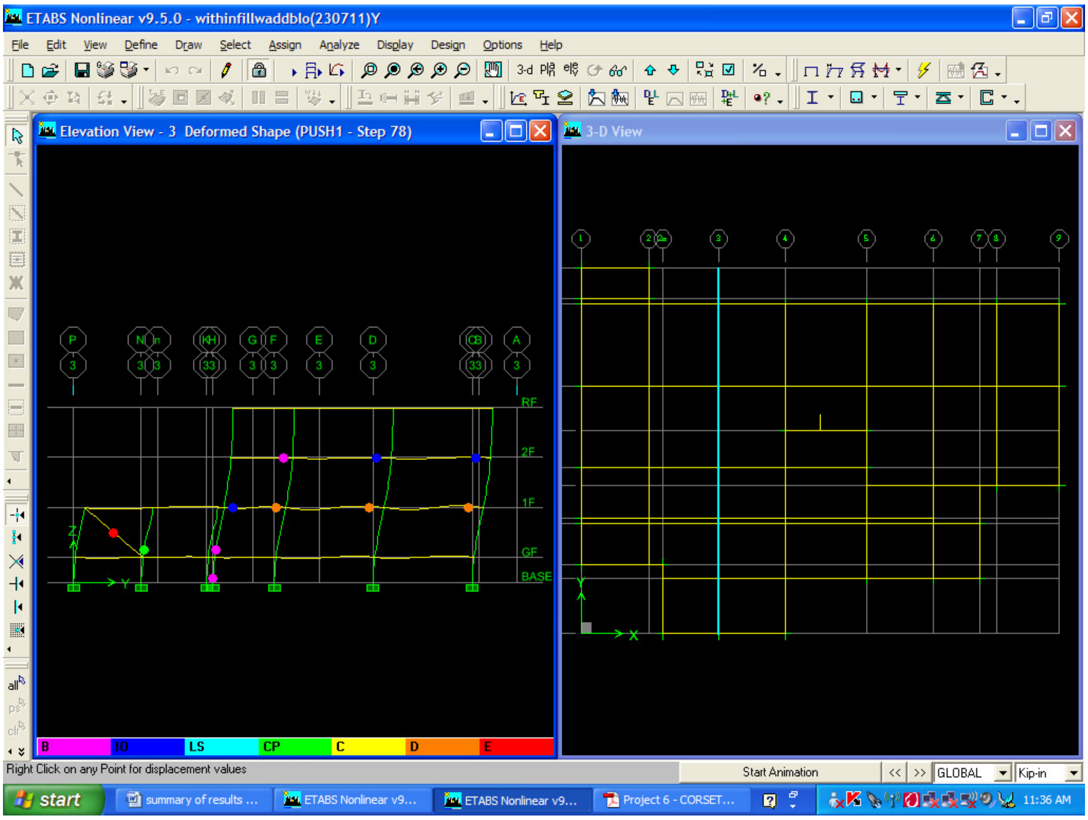
Mechanism or Deformed shapes at Performance Point for grid-1



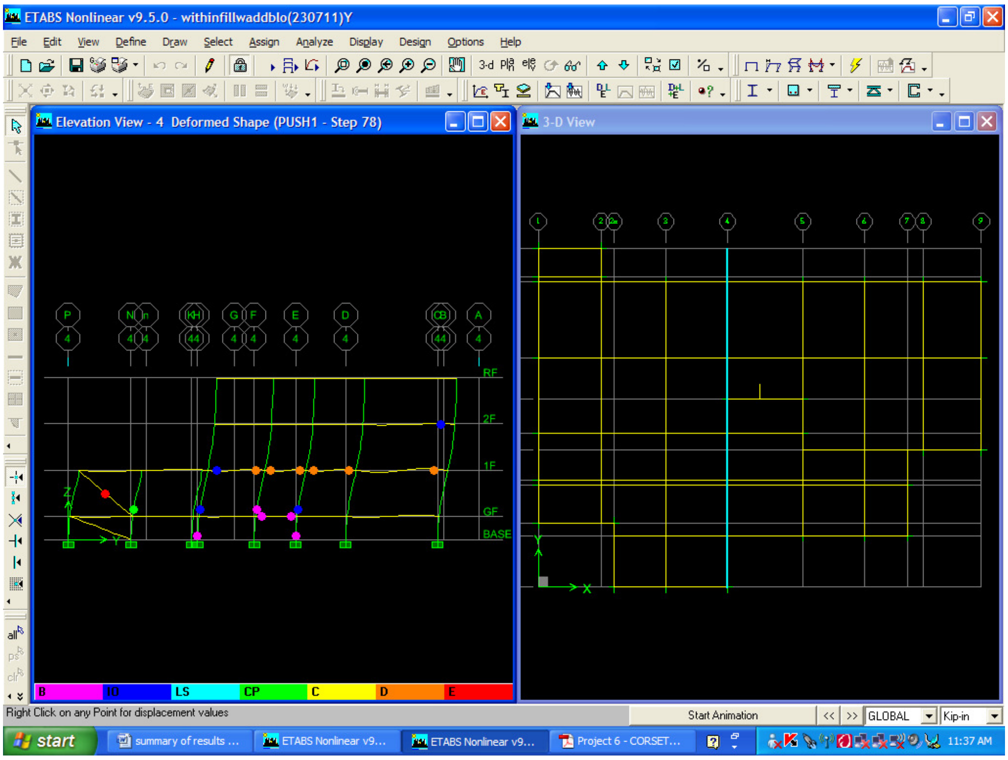
Mechanism or Deformed shapes at Performance Point for grid-2



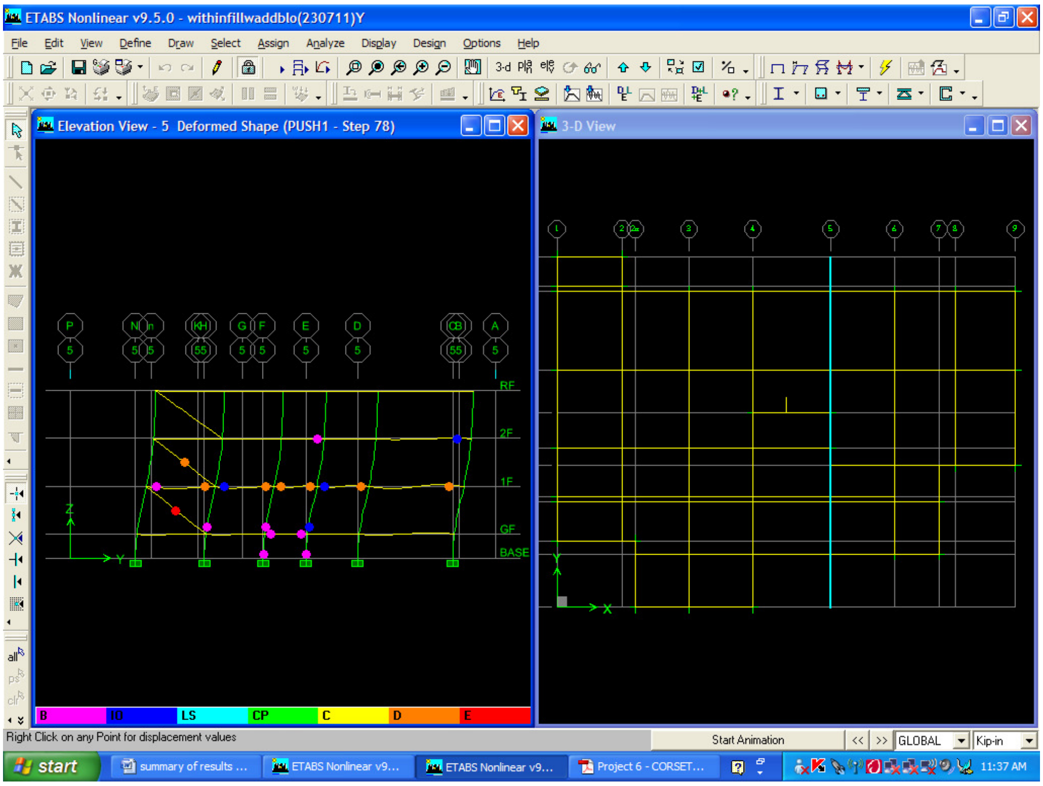
Mechanism or Deformed shapes at Performance Point for grid-2a



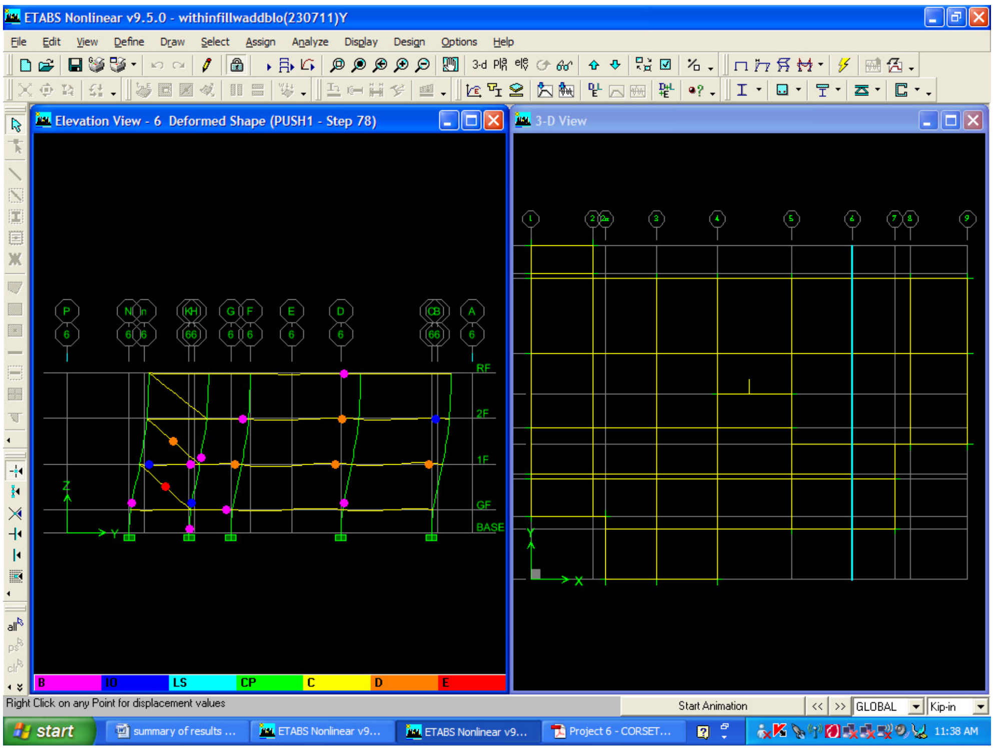
Mechanism or Deformed shapes at Performance Point for grid-3



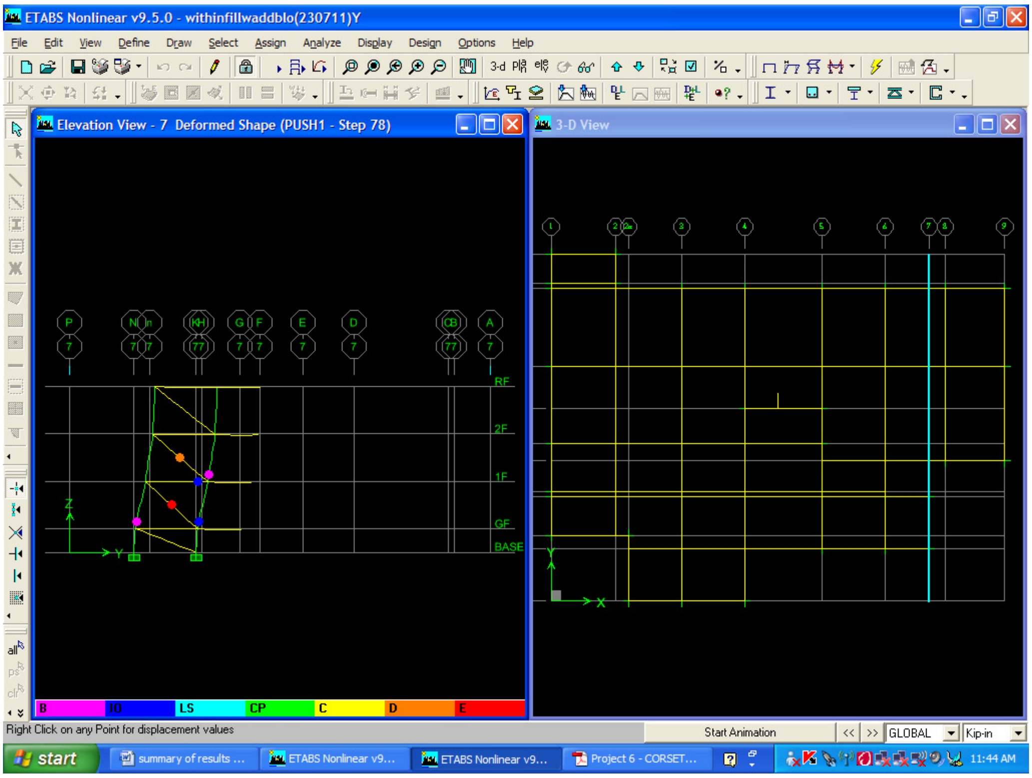
Mechanism or Deformed shapes at Performance Point for grid-4



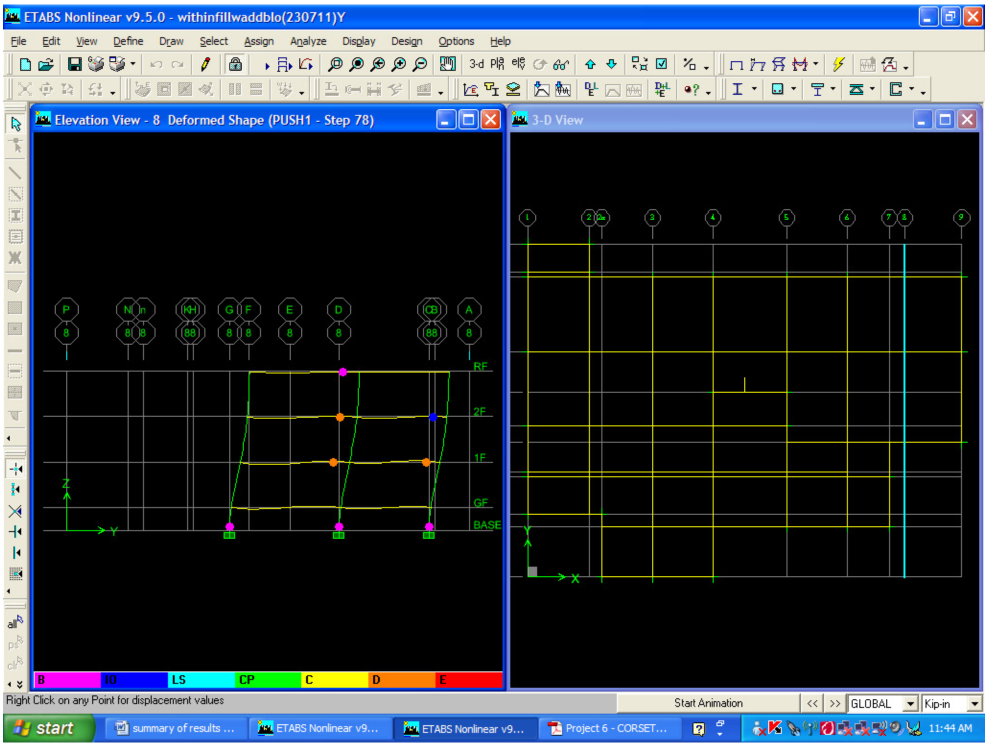
Mechanism or Deformed shapes at Performance Point for grid-5



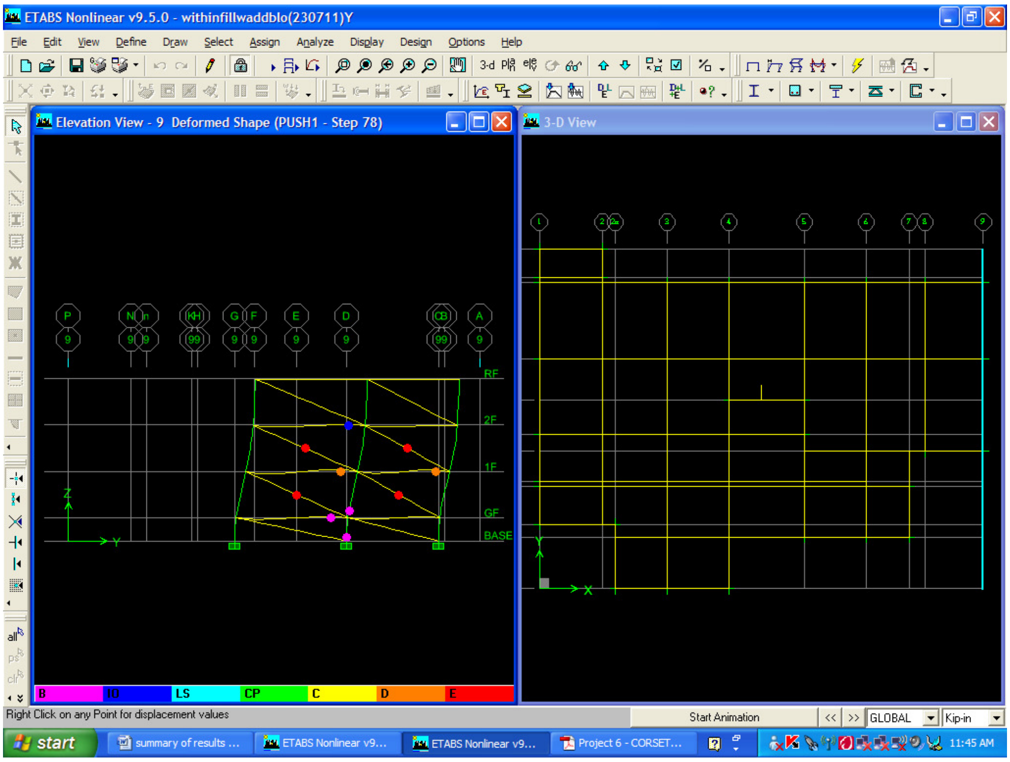
Mechanism or Deformed shapes at Performance Point for grid-6



Mechanism or Deformed shapes at Performance Point for grid-7



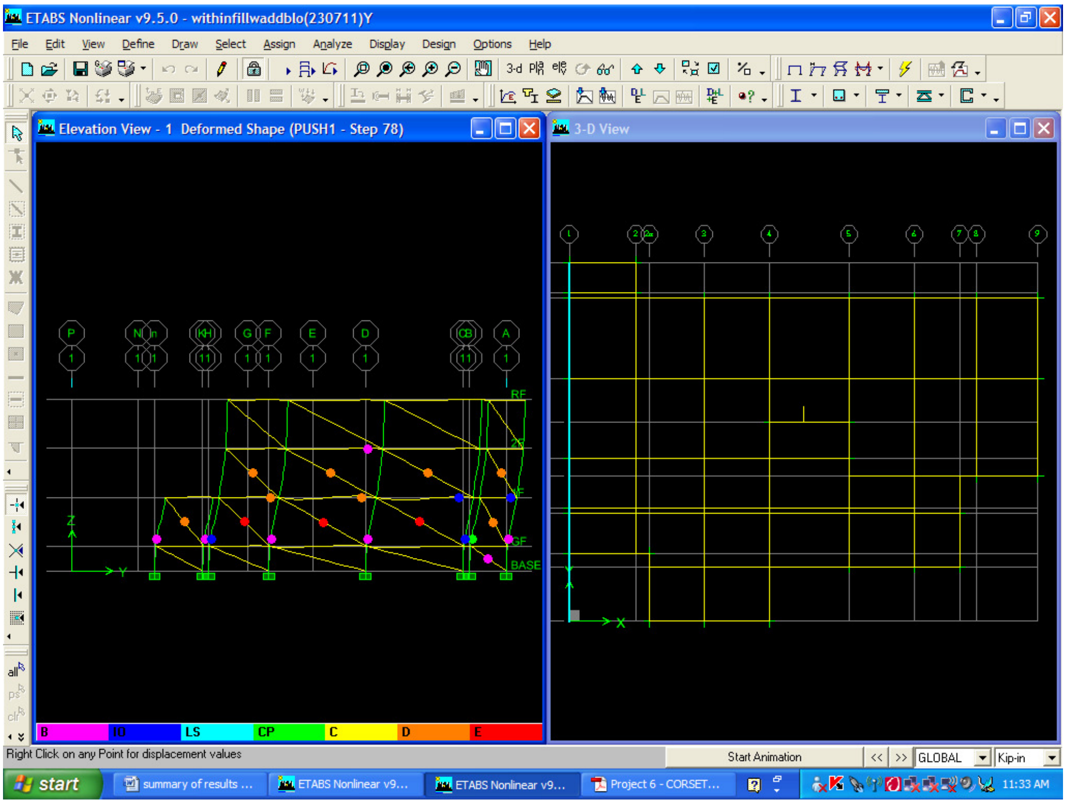
Mechanism or Deformed shapes at Performance Point for grid-8



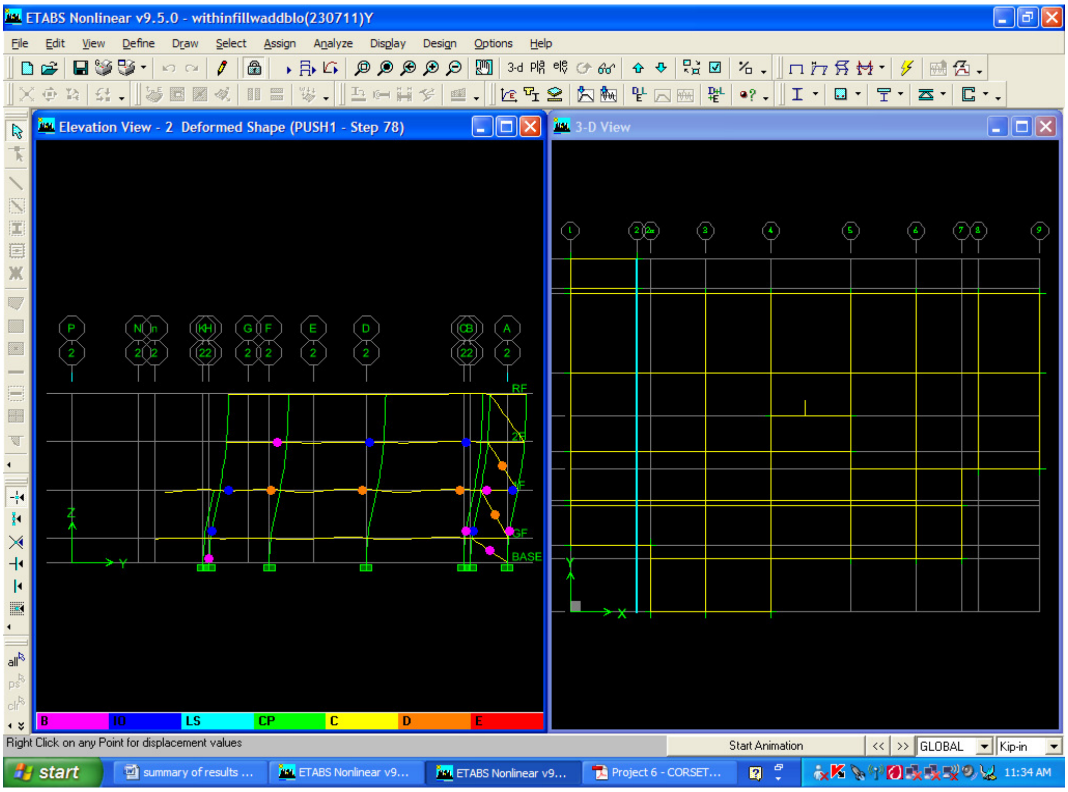
Mechanism or Deformed shapes at Performance Point for grid-9

Appendix D: Nonlinear Analysis (Tier 3) Results for Selected Retrofit Solution

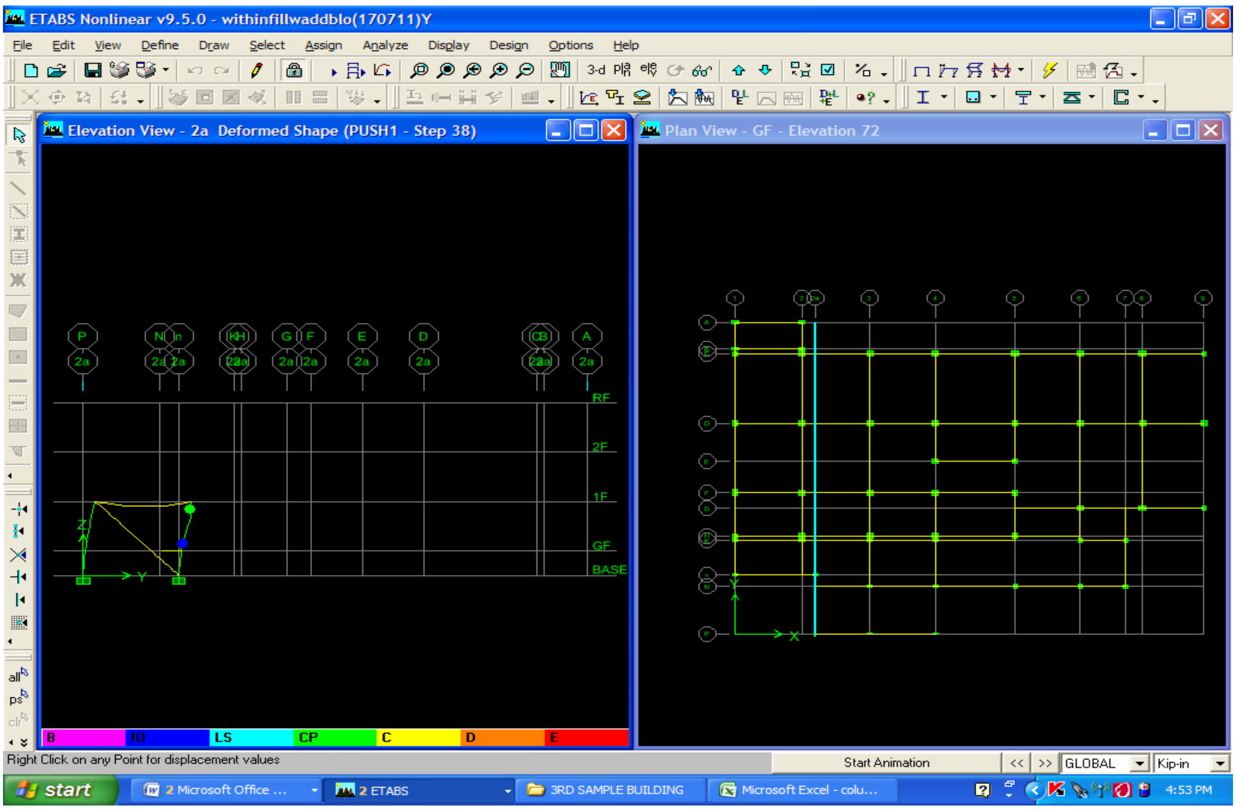
This appendix contains plots of the deformed shapes at the performance point. The state of the hinges versus the acceptance criteria are indicated by the colour of the circles using the same format as in Appendix C.



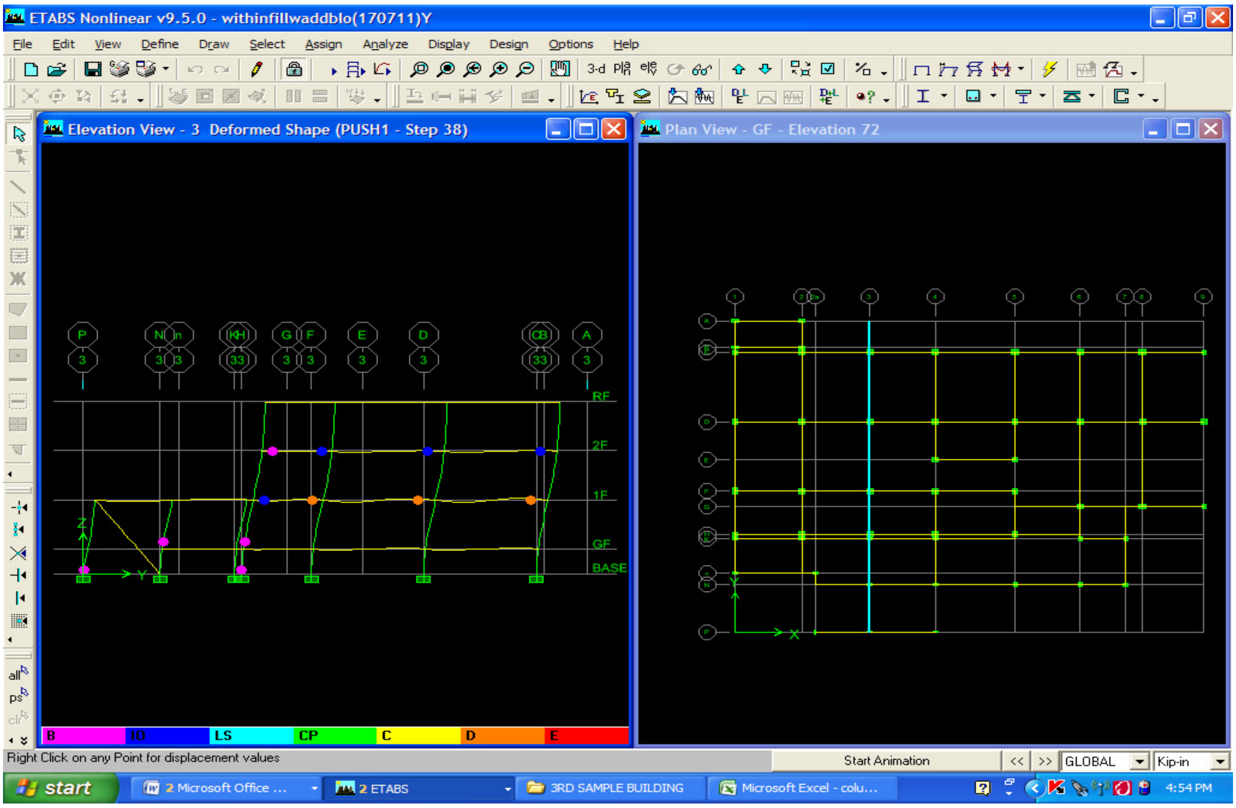
Mechanism or Deformed shapes at Performance Point for grid-1



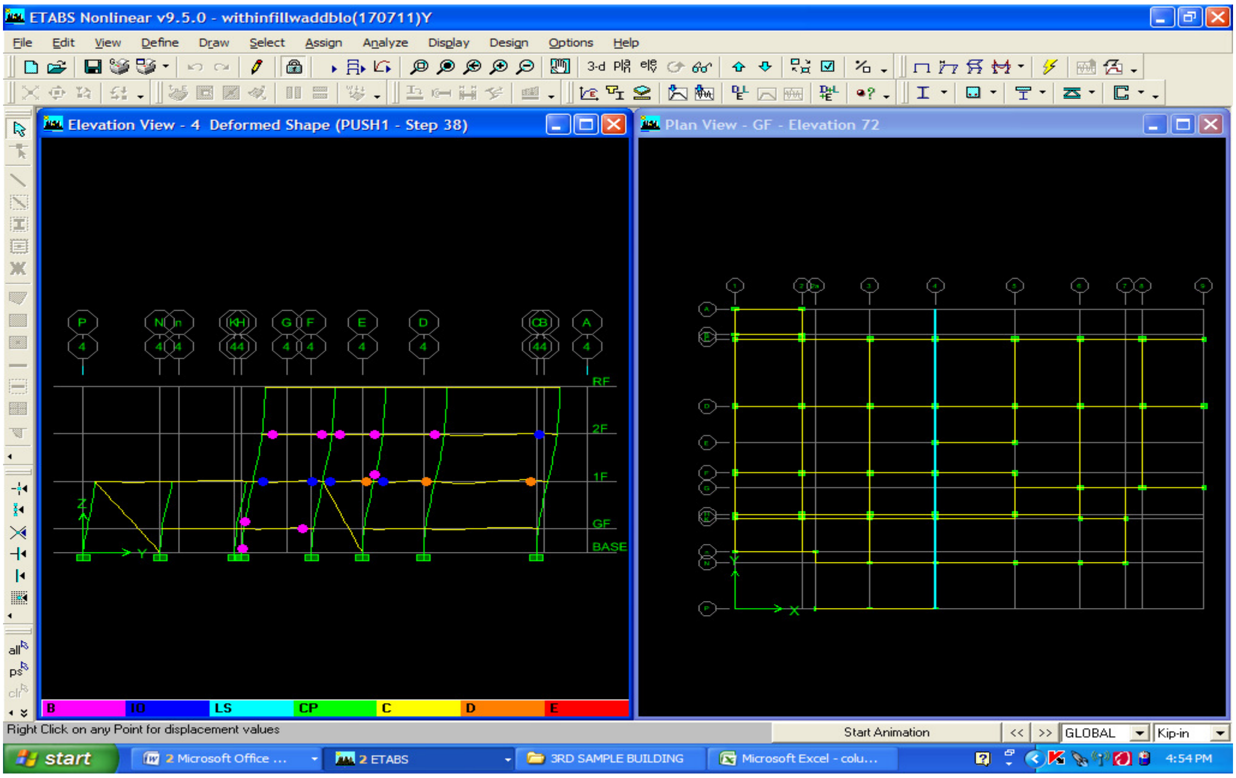
Mechanism or Deformed shapes at Performance Point for grid-2



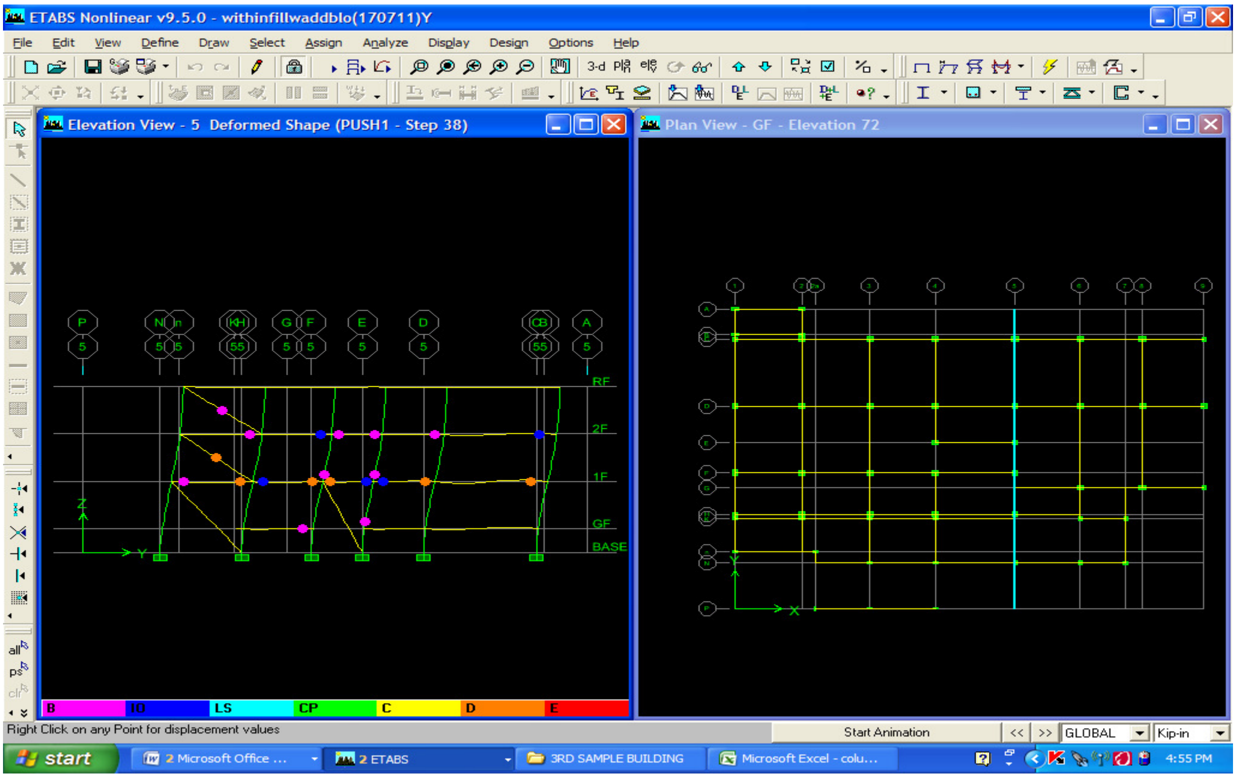
Mechanism or Deformed shapes at Performance Point for grid-2a



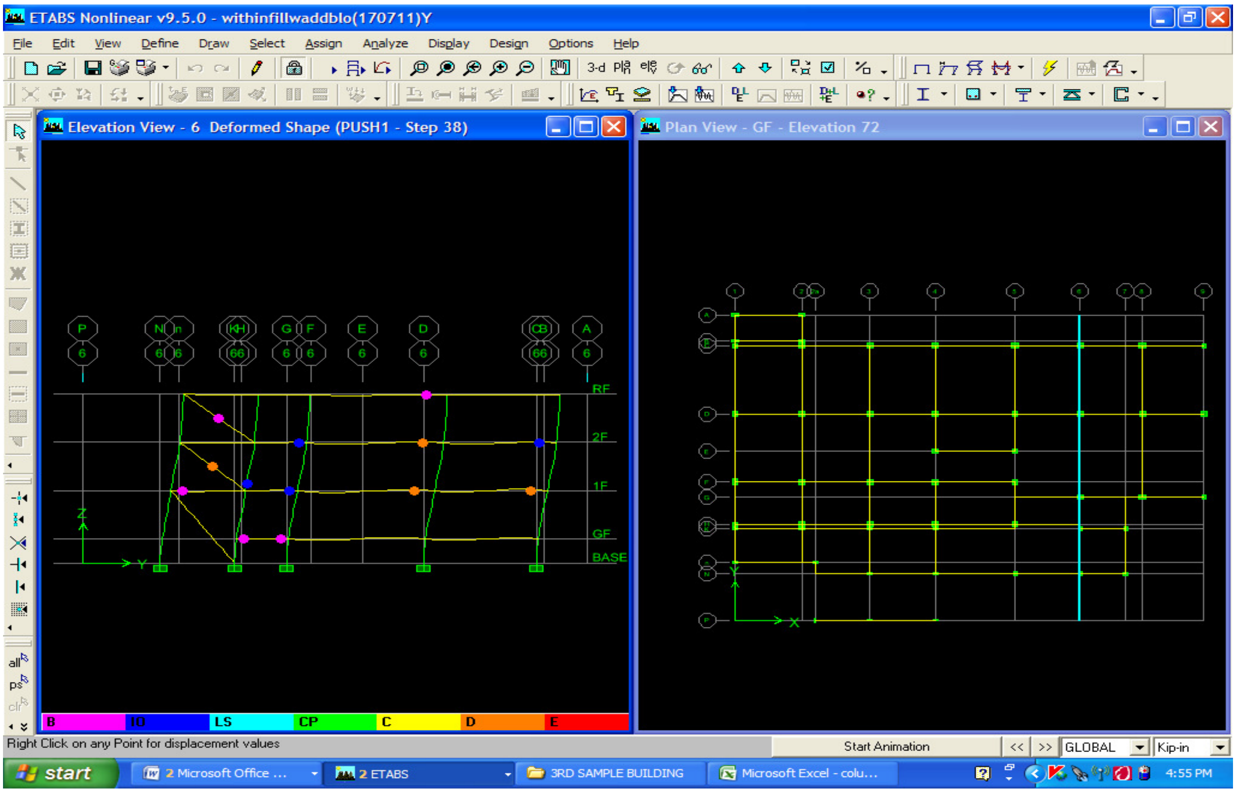
Mechanism or Deformed shapes at Performance Point for grid-3



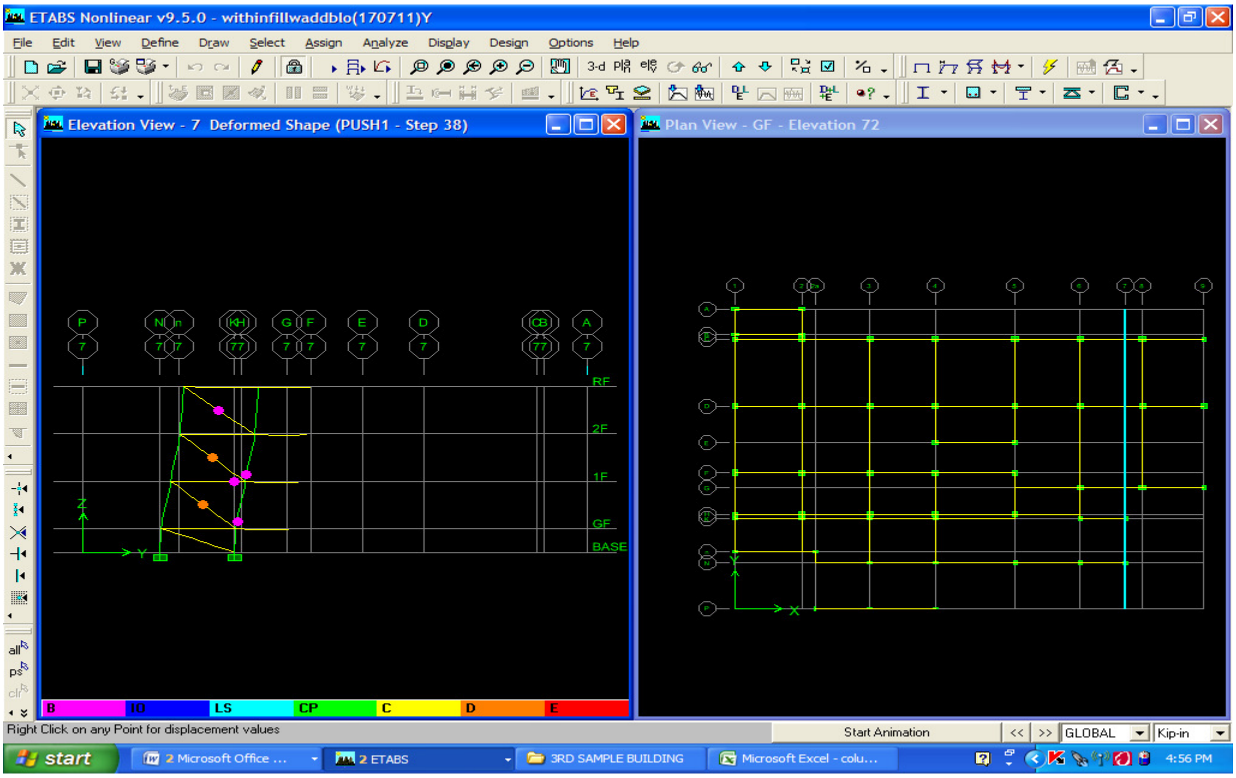
Mechanism or Deformed shapes at Performance Point for grid-4



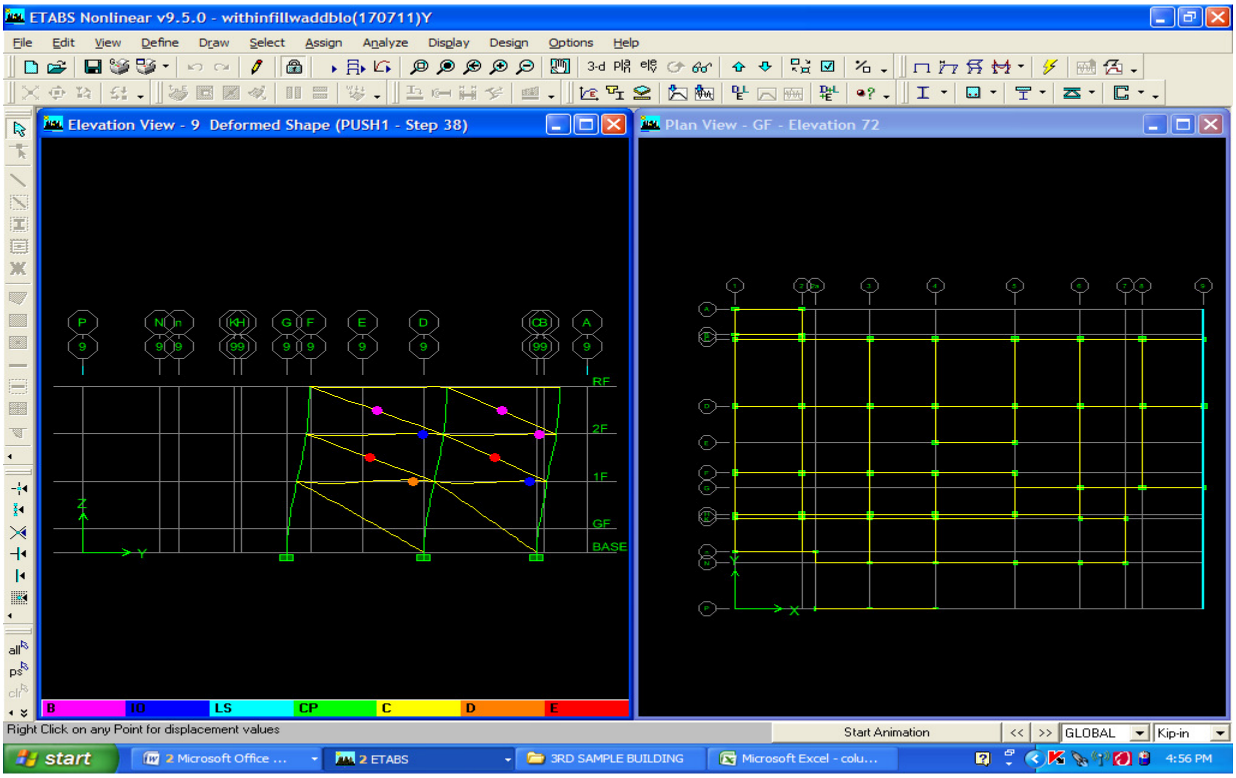
Mechanism or Deformed shapes at Performance Point for grid-5



Mechanism or Deformed shapes at Performance Point for grid-6



Mechanism or Deformed shapes at Performance Point for grid-7



Mechanism or Deformed shapes at Performance Point for grid-9

Appendix E: Retrofit Drawings