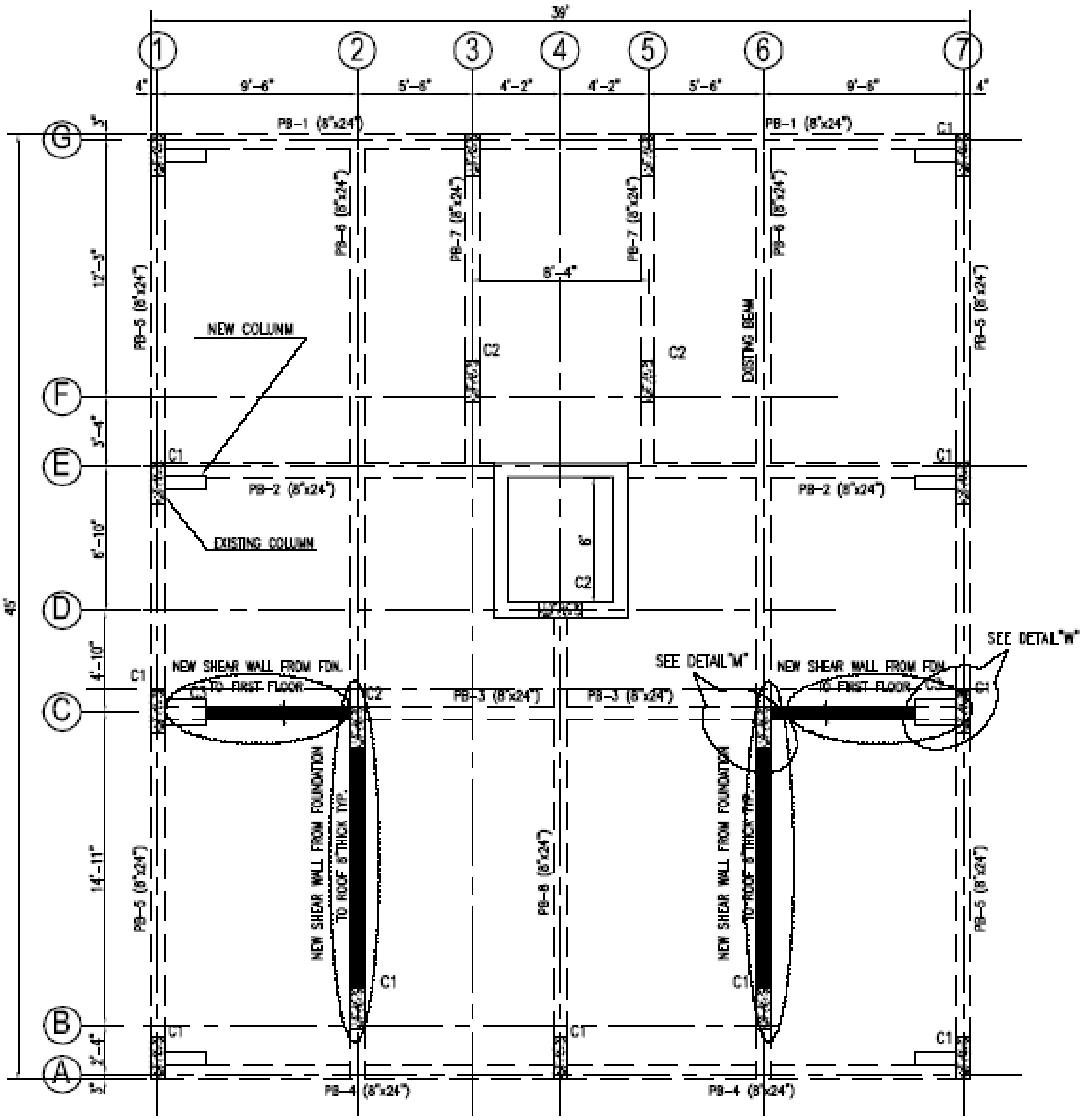

Five Storey Mixed Use Building in Karachi

A Case Study of Seismic Assessment and Retrofit Design

GeoHazards International
A Nonprofit Working Toward Global Earthquake Safety

Supported by the Pakistan-US Science and Technology Cooperation Program

THE NATIONAL ACADEMIES
Advisers to the Nation on Science, Engineering, and Medicine

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Summary

The building is located in Gulistan-e-Johar, a densely populated area in Karachi. The building consists of reinforced concrete framed building with five storeys including the ground floor. The building has shops located at the ground floor, while the above floors have residential apartments. 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 mixed-use residential buildings in Karachi: a weak story created by open shop fronts at the ground floor, an irregular framing pattern and heavy, stiff unreinforced masonry infill walls that were not considered during the building's structural design.

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 very weak in one direction, and was inadequate for collapse prevention under the maximum considered earthquake (taken as Zone 4 for Karachi). The team selected a retrofit scheme consisting of a combination of column strengthening by adding column area to create T-columns, beam strengthening near the lift core, and adding new shear walls in the ground storey and above.

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 building was investigated by Mr. Fawwad Masood, Assistant Professor, Mr. Aslam Faqeer Mohammed, Assistant Professor, and a group of undergraduate students from the Department of Civil Engineering, NED University of Engineering and Technology.

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 Tier 1 vulnerability assessment exercise carried out provided students with an opportunity to evaluate a real building with all the physical constraints. 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. This case study gave students a chance to do hands-on practice on ETABS and to learn to use the ASCE/SEI 31-03 and 41-06 documents.

Building Information

The building is a five storey (ground plus four) mixed use apartment building with shops at the ground floor. The building's overall dimensions are 39 feet wide by 48 feet long and it is approximately 53 feet tall. The building has a reinforced concrete moment frame structural system with unreinforced concrete block infill walls, and a reinforced concrete core. The column layout is irregular with many columns the same 6 inch thickness as the infill walls. The foundations are reinforced concrete spread footings. The building is older and has had some repairs but no condition assessments. Figure 1 through Figure 4 show the architectural and structural drawings.

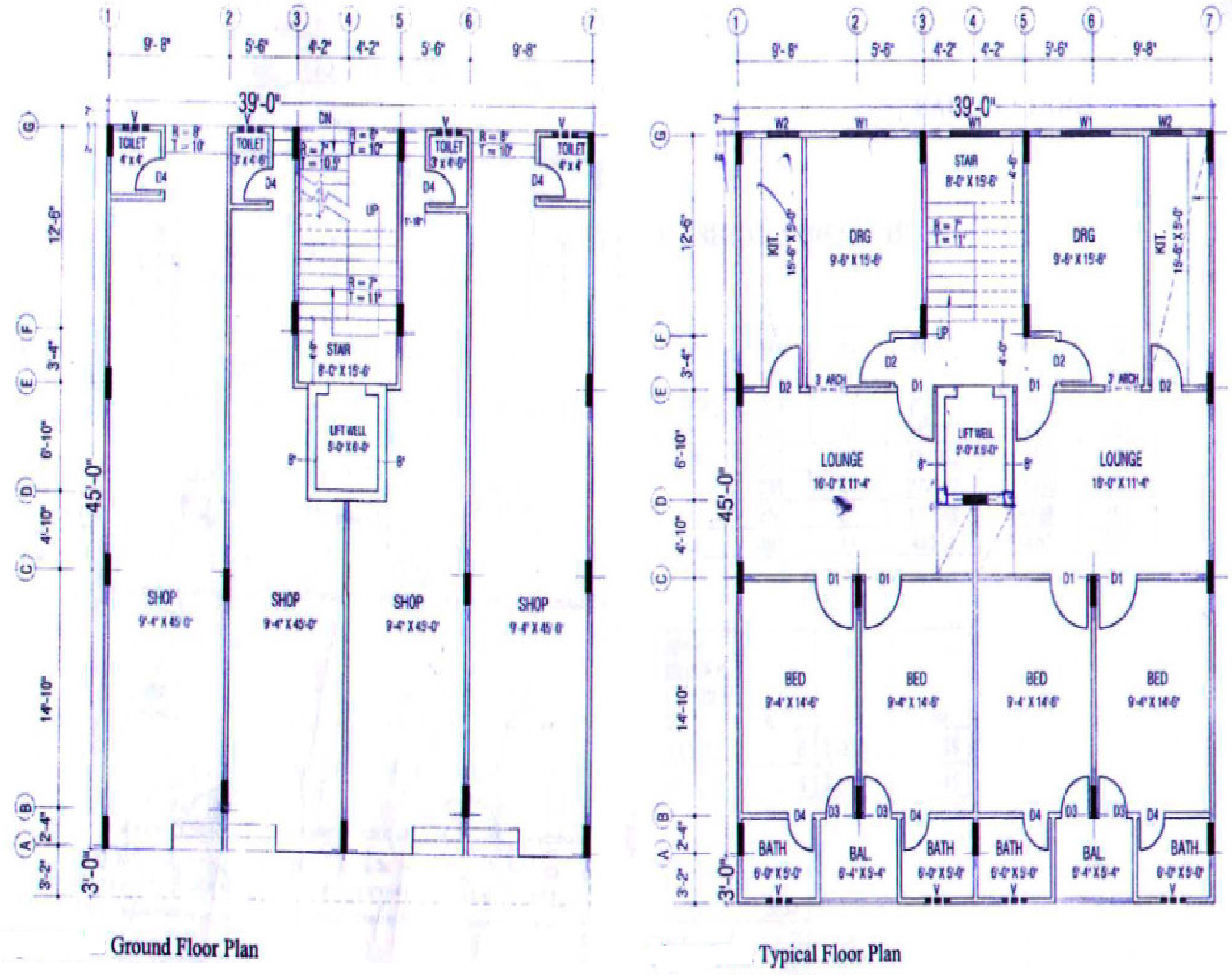


Figure 1. Architectural plan of ground (left) and plan of typical floors (right)

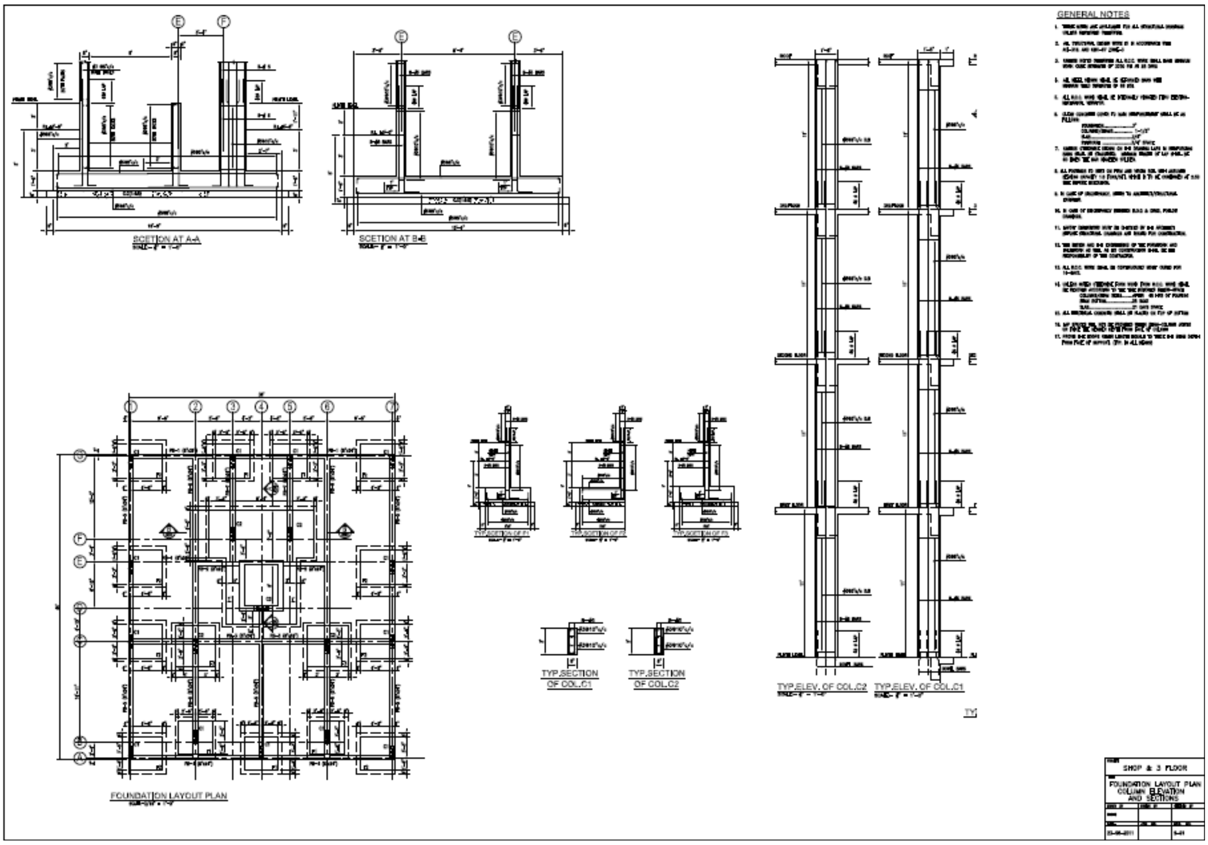


Figure 2. Structural foundation plan

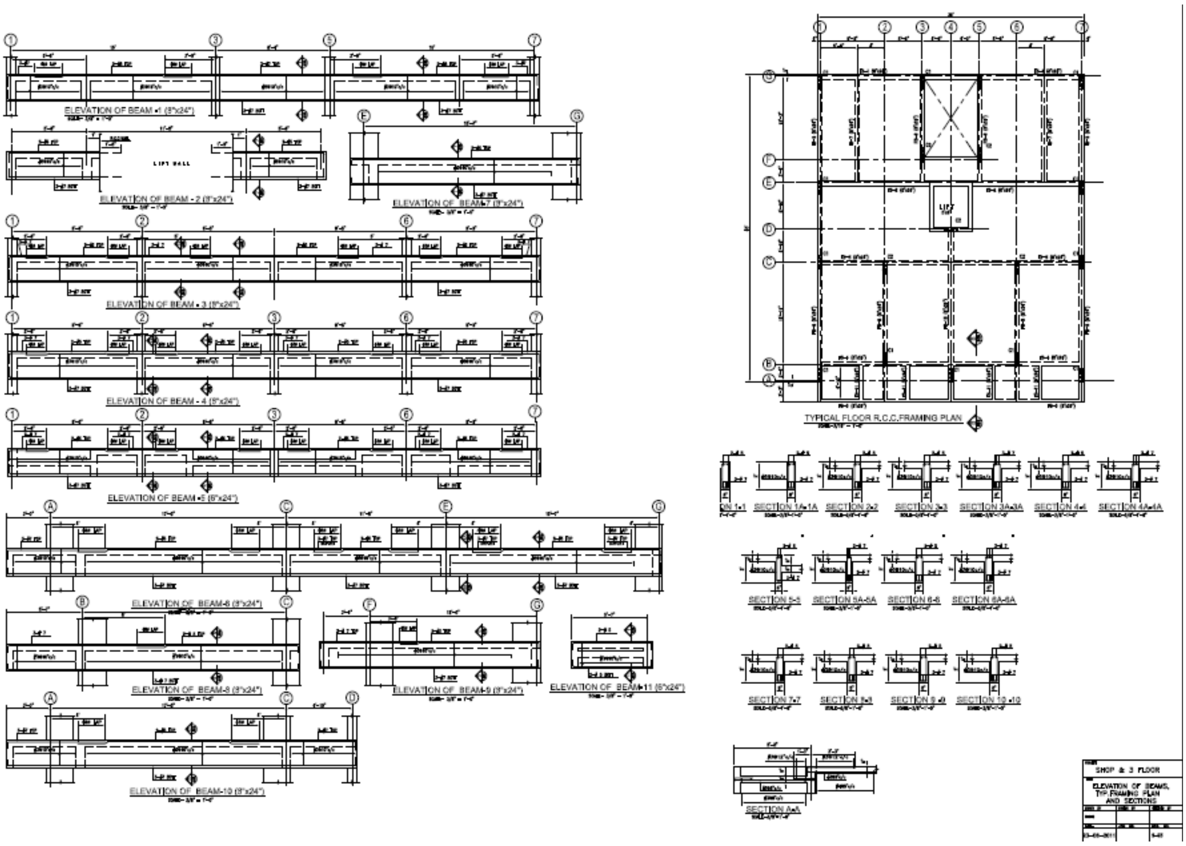


Figure 3. Structural typical floor plan with beam elevations and sections

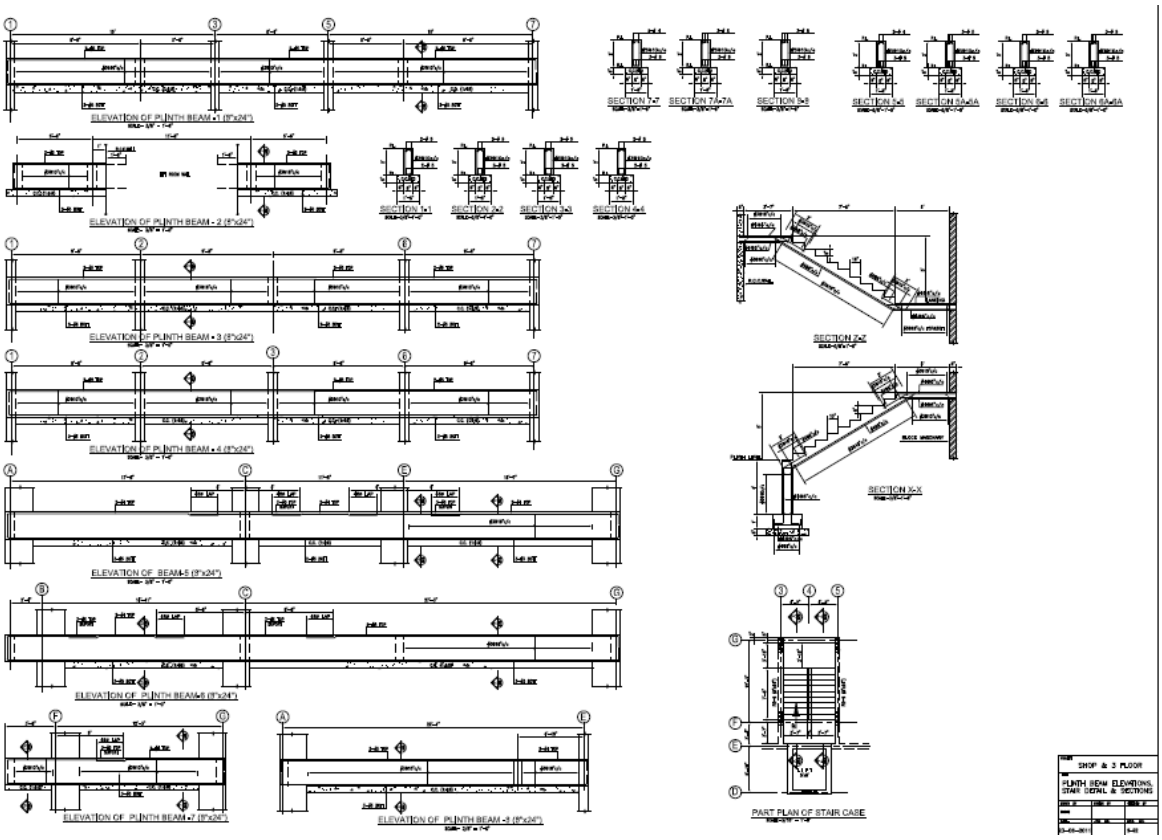


Figure 4. Structural beam elevations and sections for typical floor plan and staircase

Concrete of f'c =3000psi and steel of fy = 60000 psi are assumed. The typical beam sizes are 8"x24" and column sizes from ground to roof are 8"x24" with a few columns with dimensions of 8"x30". The slabs are 6" thick. Original design calculations are not available but ACI-99 was used to design the frame elements and earthquake analysis might possibly have been carried out using UBC-97. As the drawings show, the building has few structural members or infill walls in the transverse direction, especially at the ground floor.

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	Weak storey Soft storey Mass irregularity Torsion
Lateral Force-resisting System	Interfering wall Proportion of infill walls Strong column/ weak beam Column/beam bar splices Joint reinforcement Shear stress check Axial stress check
Geologic Hazards and Foundation	Ties between foundation elements

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 greater than two, so the building is expected to respond in the nonlinear range. Please see Appendix B for linear analysis results.

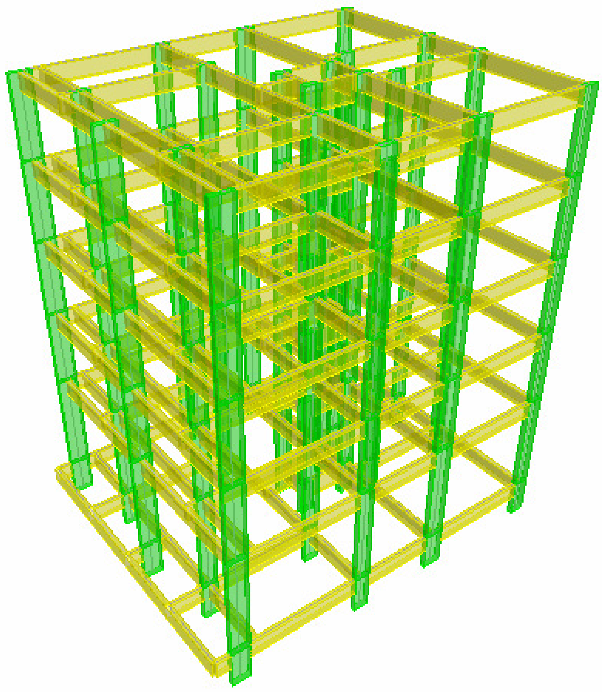


Figure 5. Rendering of linear ETABS model of the building

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%)
ROOF	D1	228.475	280.973	229.326	299.913	0.2	3.3
FOURTH	D1	232.04	285.677	229.82	301.667	0.5	2.8
THIRD	D1	232.04	285.677	230.335	303.551	0.4	3.1
SECOND	D1	232.04	285.677	230.871	305.992	0.3	3.5
FIRST	D1	231.875	286.051	231.449	309.891	0.1	4.2

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 as 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
ROOF	1.1414	---	11
FOURTH	1.2825	12	0
THIRD	1.2825	0	0
SECOND	1.2825	0	2
FIRST	1.3123	2	70
GROUND	0.7738	41	---

Table 2 shows that the building has mass irregularity at first 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
ROOF	0.001398	0.00538	0.001035	0.00398
FOURTH	0.002195	0.00845	0.001561	0.00601
THIRD	0.002821	0.01086	0.002001	0.00770
SECOND	0.003219	0.01239	0.00235	0.00905
FIRST	0.003855	0.01484	0.002635	0.01014
GROUND	0.002528	0.00973	0.001067	0.00411

Table 3 shows that drifts are less than the allowable drift value of 0.02 in both X and Y direction.

Table 4. Soft Storey check

Story	Load	storey force	Total Displacement	Stiffness	% diff in K (30% allow) % difference compare to	
		kips	inches	kip/in	Above storey	Below storey
ROOF	EX	150	1.8523	80.98	----	2.3
FOURTH	EX	140	1.6886	82.91	2.4	7.2
THIRD	EX	111	1.4357	77.31	6.7	4.2
SECOND	EX	82.5	1.1114	74.23	4.0	0.7
FIRST	EX	54.6	0.7409	73.69	0.7	19.3
GROUND	EX	8.7	0.1408	61.79	16.2	----

Story	Load	storey force	Total Displacement	Stiffness	% diff in K (30% allow) % difference compare to	
		kips	inches	kip/in	Above storey	Below storey
ROOF	EY	150	1.3175	113.85	----	2.9
FOURTH	EY	140	1.1943	117.22	3.0	6.4
THIRD	EY	111	1.0075	110.17	6.0	2.5
SECOND	EY	82.5	0.7675	107.49	2.4	4.2
FIRST	EY	54.6	0.4867	112.18	4.4	18.0
GROUND	EY	8.7	0.0636	136.79	21.9	----

Table 4 shows that the building has no soft storey present in both in x and y directions.

Detailed Evaluations of Existing Building

Through linear static analysis of this building, Tier-2, the checks for building system (mass irregularities, torsion etc.) in Tier 1 analysis which were shown to be non-compliant through visual inspection, came out to be compliant, except for mass irregularity. In addition it was also observed that many columns had DCR > 1 and even 2 (Appendix B: Linear Analysis (Tier 2) Results). This shows that the building is expected to respond in the nonlinear range. In Tier 3 (non linear analysis), pushover static analysis according to ATC-40 and ASCE 41-06 criteria was adopted. After the nonlinear analysis, it was seen that a performance point could not be found for Y-direction. The team determined that the likely reason for this was the loss of stability and stiffness of the overall structure due to failing of so many columns. Therefore it was advised to increase the sizes of the columns and see if a solution could be found for y-direction analysis.

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 standard (Seismic Rehabilitation of Existing Buildings) was adopted to determine the plastic hinge properties for compression 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.

Loading and Performance Criteria

For the pushover analysis, the team used restart using secant stiffness for member unloading method with P-Delta effects for geometric nonlinearity. Table 5 shows loading input parameters. A life safety performance criterion was selected for the study building.

Table 5. ETABS loading input parameters

Gravity loads:	Slab loads transferred to beam were manually calculated and applied to each of the beams in the 3-D model.
Earthquake load:	
Z	0.4g.
R	5.5
C_a	$0.4N_a$ (Ref: Table 16-Q (UBC 97)) where $N_a = 1.0$
C_v	$0.4N_v$ (Ref: Table 16-R (UBC 97) where $N_v = 1.0$
Soil type	S_B (Ref: Table 16-J UBC-97)

Analysis Results

Figure 6 shows the load-deformation curve in the Y-direction analysis and Figure 7 shows the performance level where the demand and capacity spectra intersect each other, at the point called the *performance point* where it is necessary to see the condition of the structure, and whether it is fulfilling the demand or not.

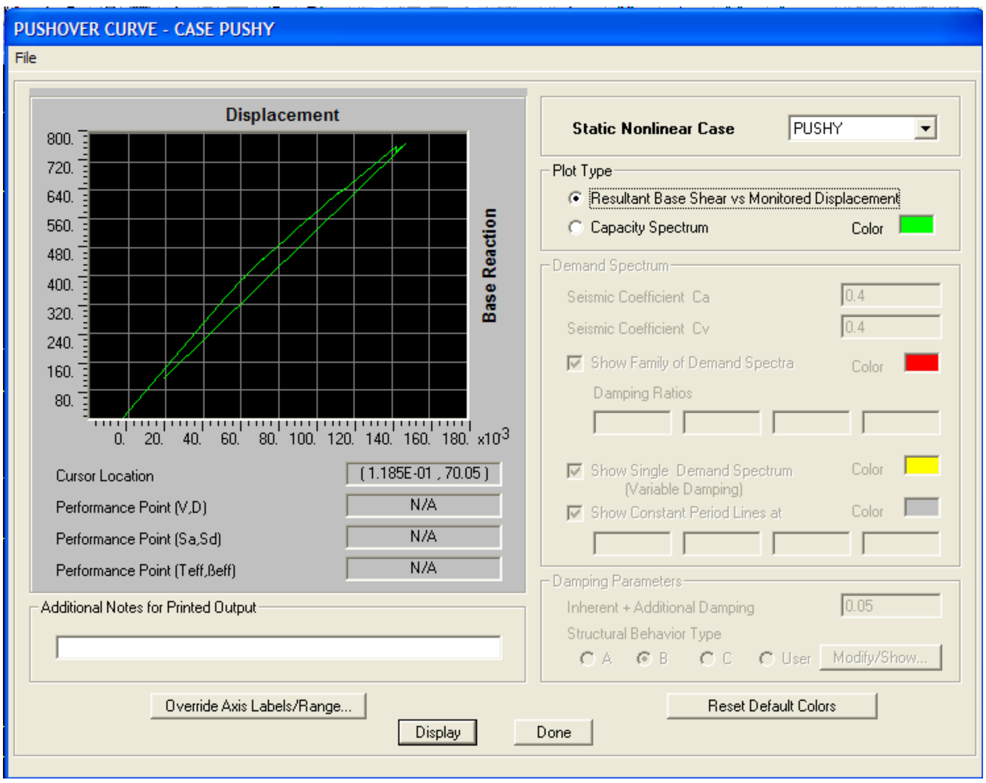


Figure 6. Pushover curve for Y-direction

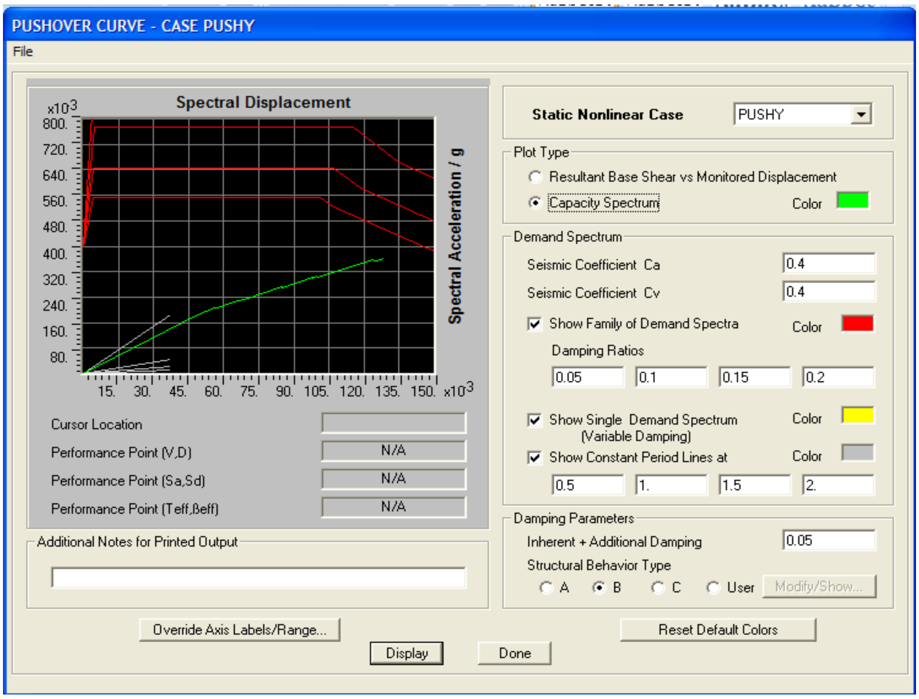


Figure 7. Performance level for Y-direction

Figure 8 shows the state of the nonlinear hinges at the performance point for Y-direction analysis. Note the failure of the ground storey columns.

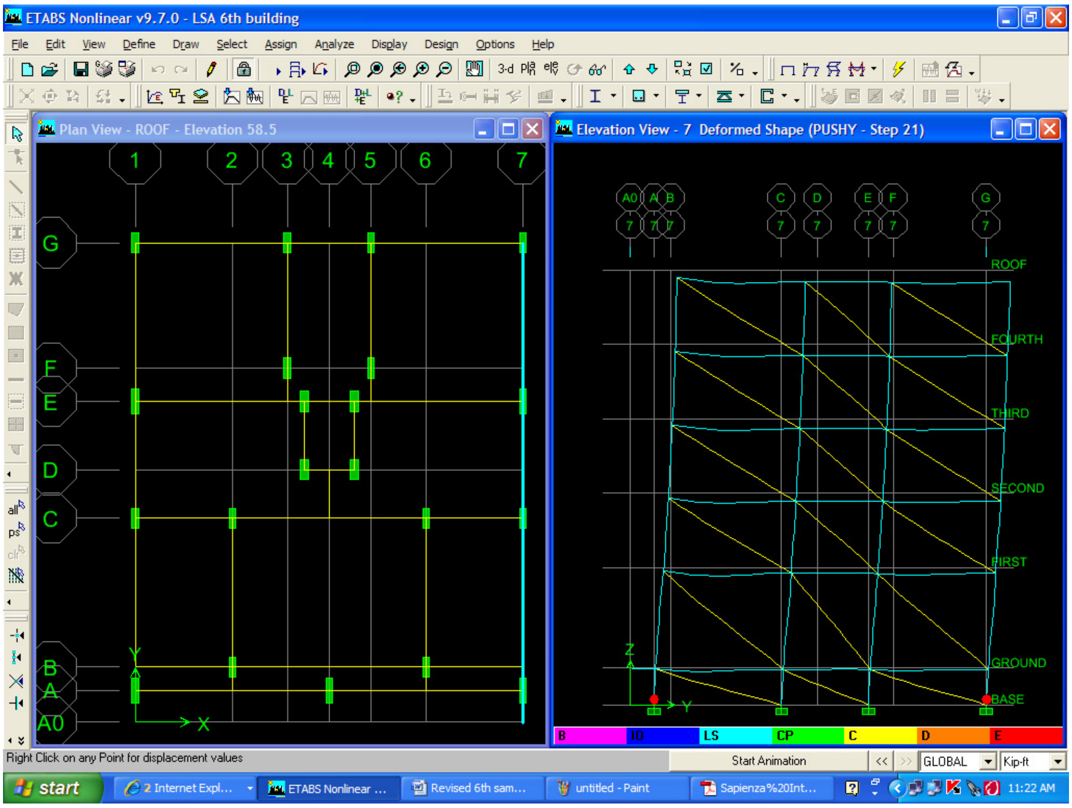


Figure 8. Hinge deformation vs. acceptance criteria

Figure 9 shows the load-deformation curve in the X-direction analysis and Figure 10 shows the capacity spectrum and the performance point.

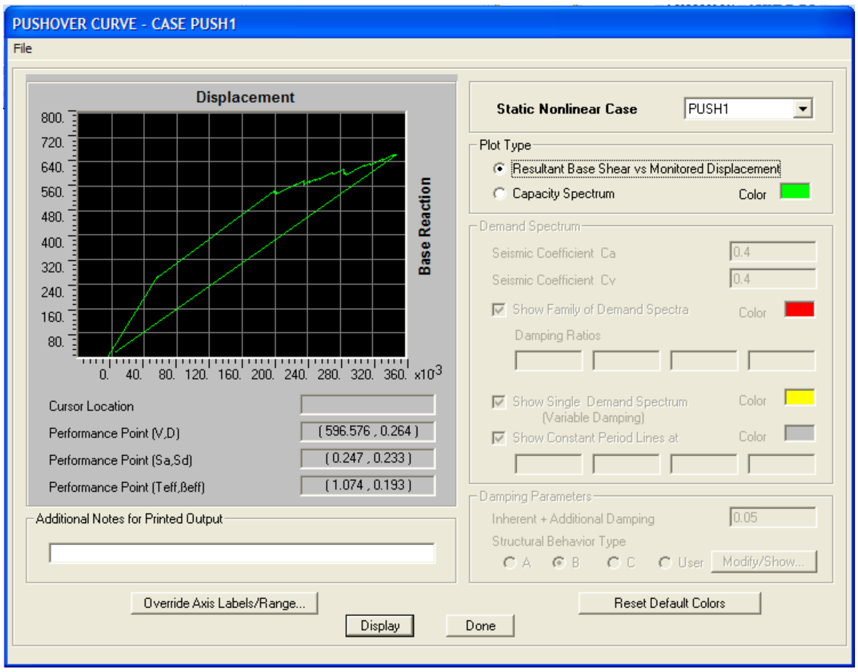


Figure 9. Pushover curve for X-direction

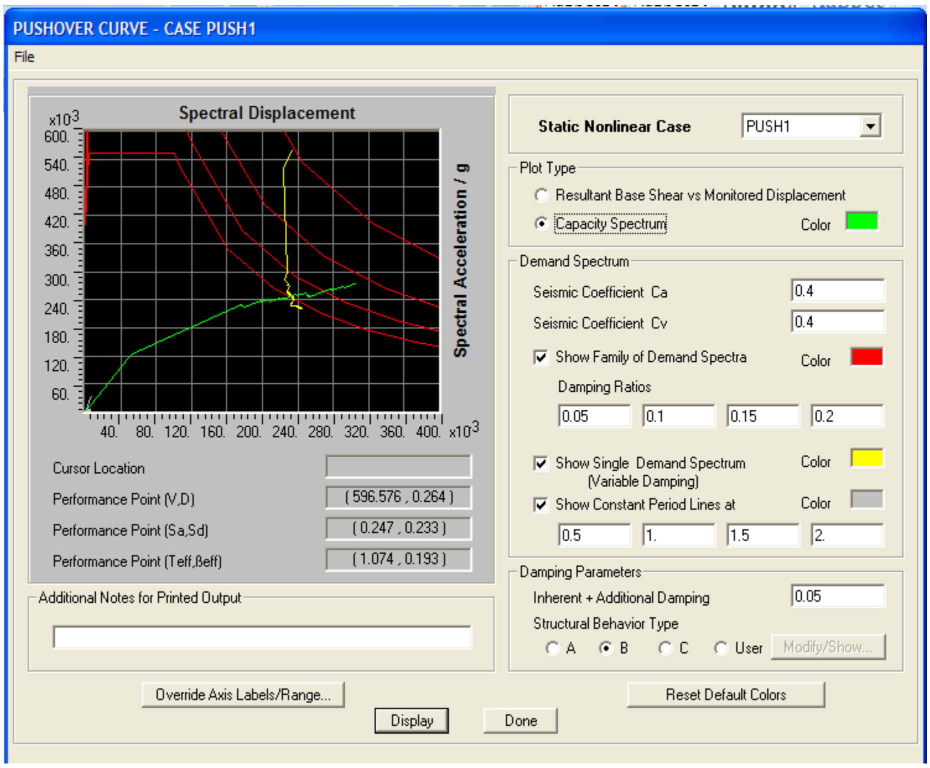


Figure 10. Performance level for X-direction

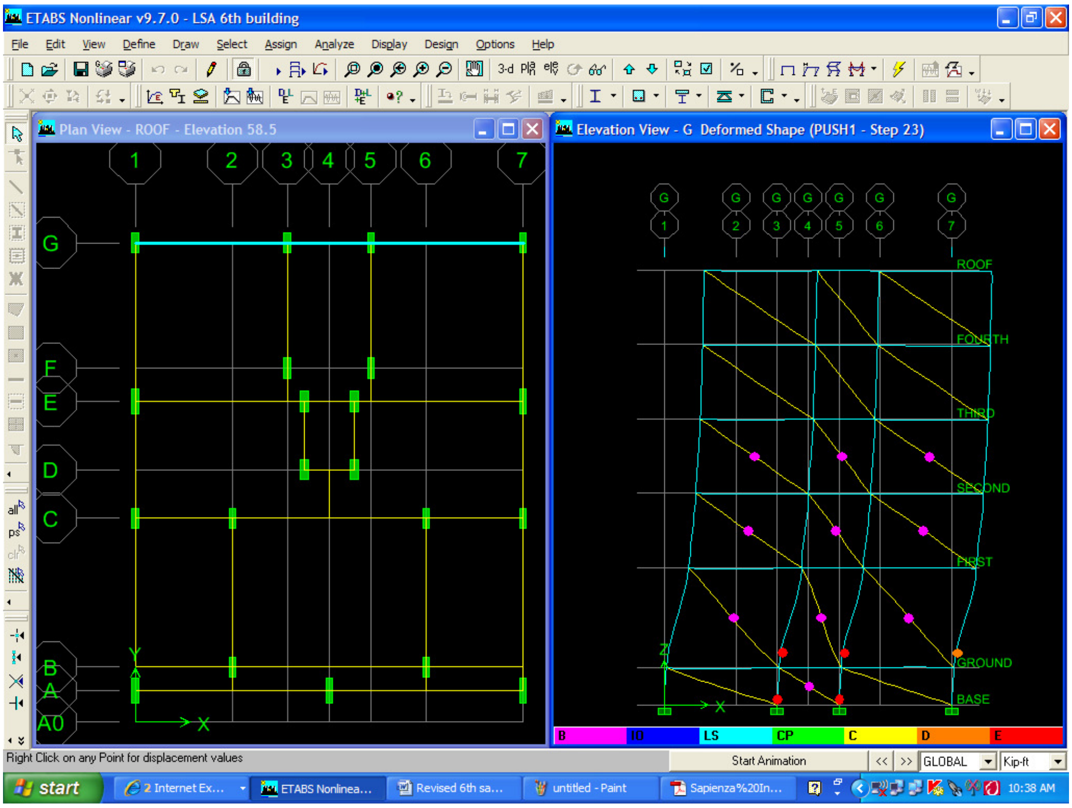


Figure 11. Hinge deformation vs. acceptance criteria for x-direction analysis

Figure 11 shows the state of the nonlinear hinges at the performance point. Columns at the ground storey and base have failed, and the building is forming a single-storey collapse mechanism at the ground storey. This shows that retrofitting is needed to achieve stability and to prevent collapse.

Retrofit Solution

Conceptual Solutions Considered

For retrofitting, a technical advisor suggested that the team consider increasing the sizes of the periphery columns from 8" to 16", increase the size of the beam on grid-E that frames into the lift core, and to add RCC shear walls to replace ordinary masonry infill walls to form spines. The beam on grid-E is a primary connection in the X-direction between the lift core and the building's RCC frame structure, so increasing the strength of this beam improves performance. Therefore, in addition to RCC shear walls the width of the beam was increased and new rectangular columns are introduced at the outer periphery of the building, as shown in Figure 12. Due to weakness in the ground storey two new walls were added in only that storey.

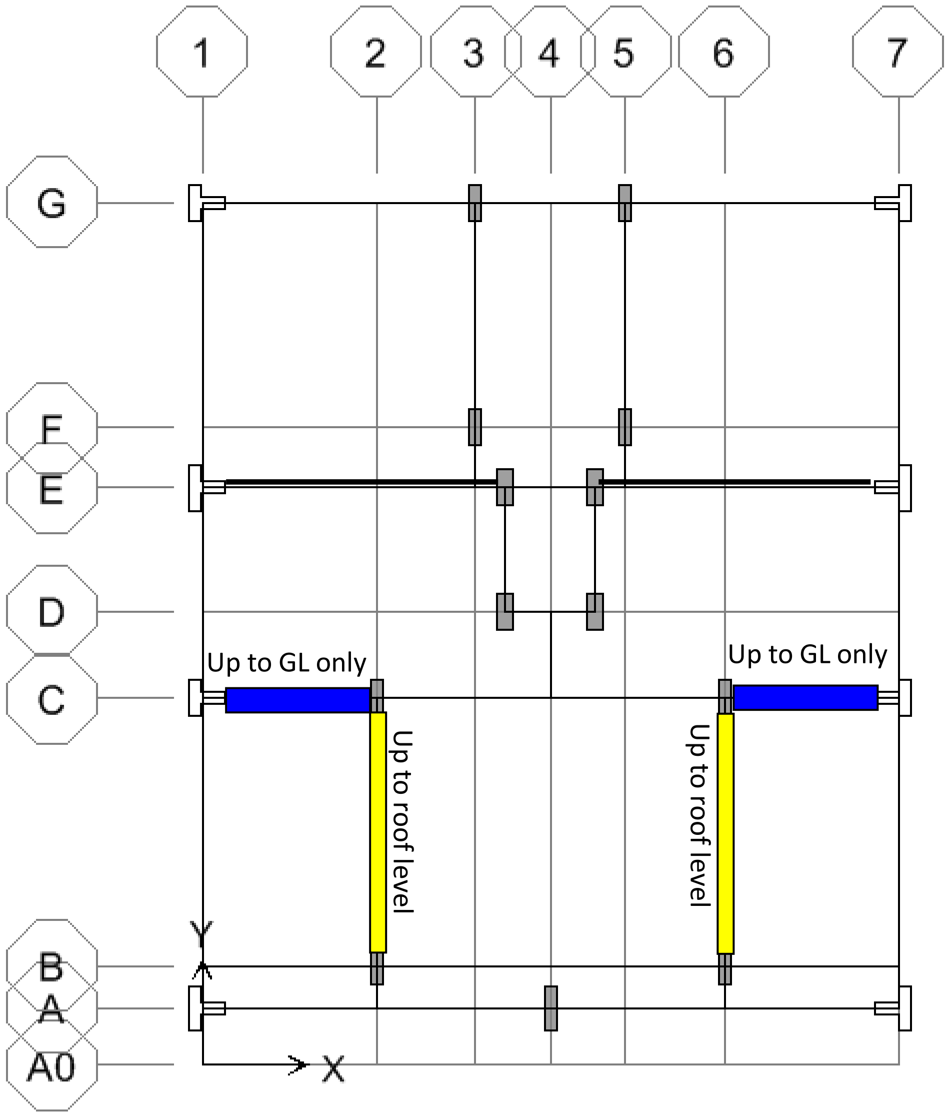
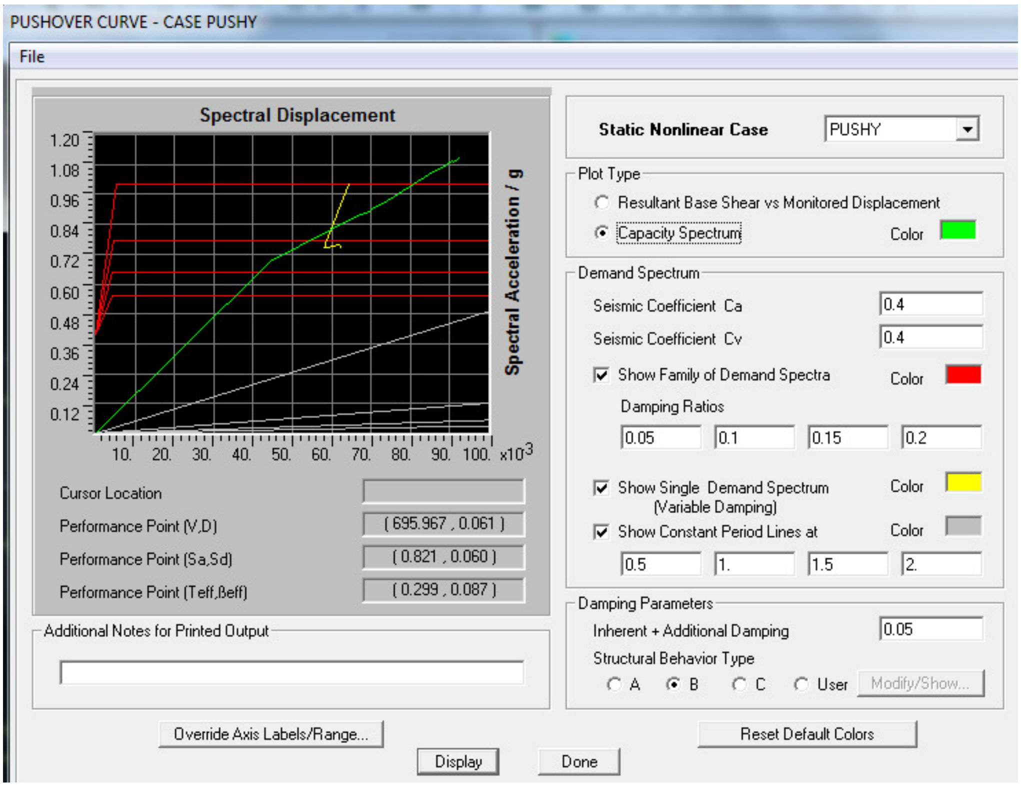


Figure 12. Modified retrofit solution

Comparison of Analysis Results

Figure 13 through Figure 15 compare pushover analysis results for the retrofitted building model with those of the existing building. As shown, the retrofit eliminates the column failures and prevents a weak storey mechanism from forming at the ground storey. Please see Appendix C for additional nonlinear analysis results.



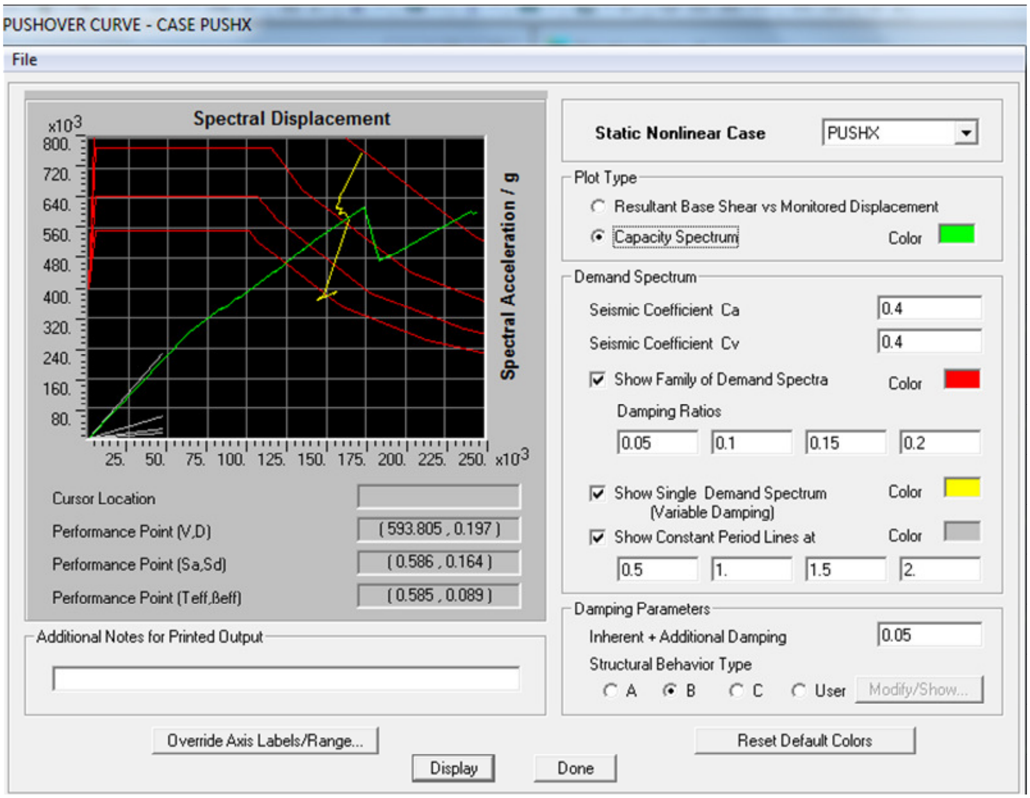


Figure 13. Performance level for the retrofitted building – Y-direction (top) X-direction (bottom)

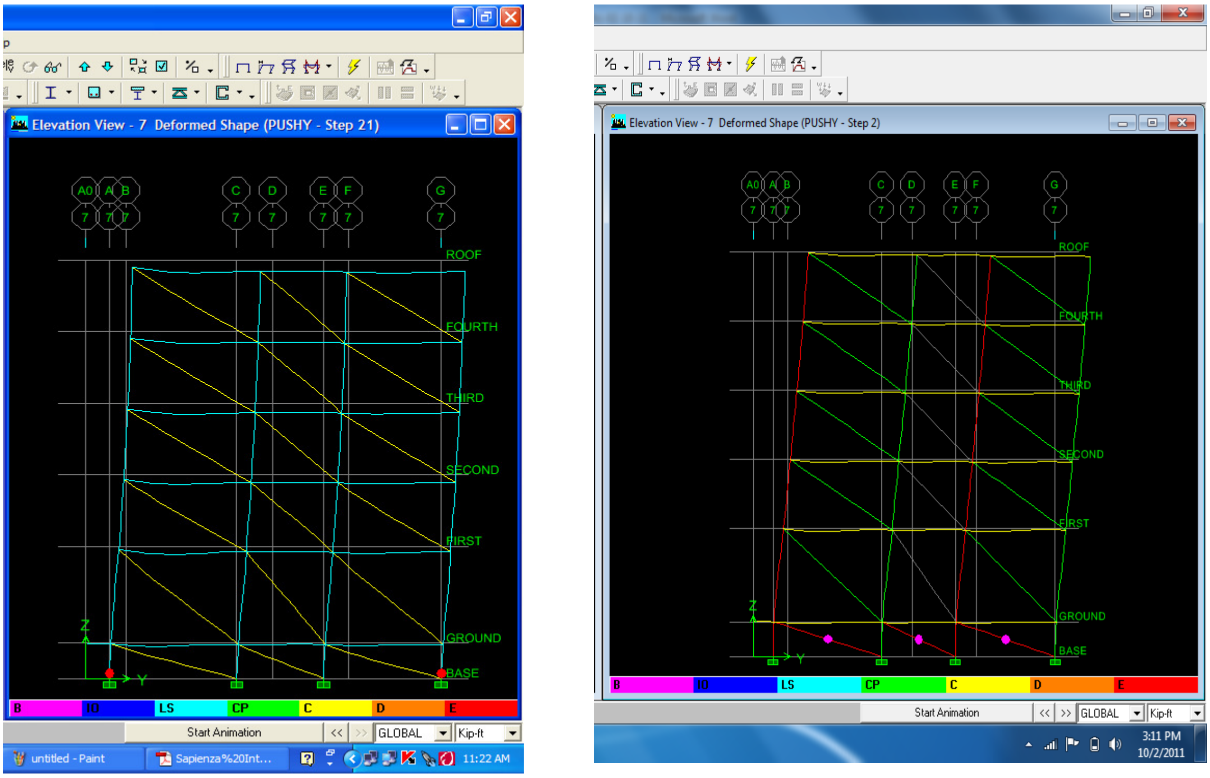


Figure 14. Hinge deformation vs. acceptance criteria in Y-direction before retrofit (left) after retrofit (right)

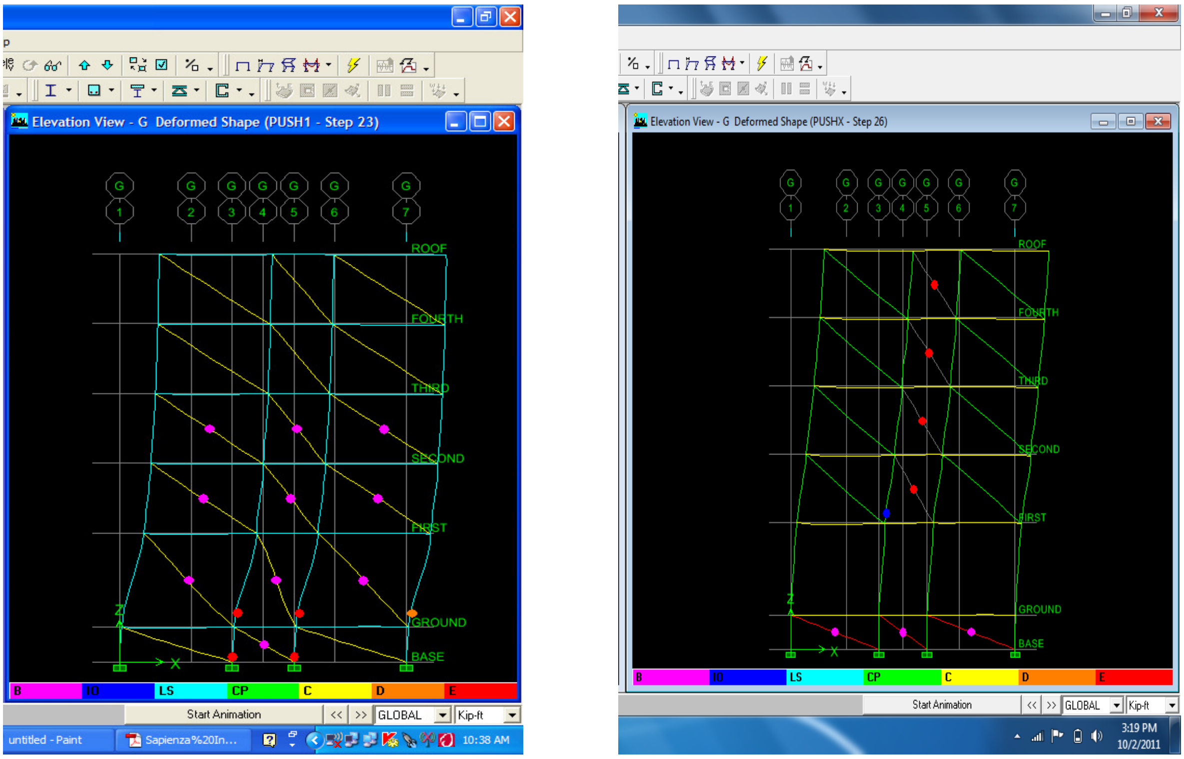


Figure 15. Hinge deformation vs. acceptance criteria in Y-direction before retrofit (left) after retrofit (right)

Design and Detailing of Retrofit

Significant detailing was required to accomplish the increase in beam sizes, jacketing of columns and introduction of RCC walls. Figure 16 shows typical details for the new shear walls, while Figure 17 shows the location of the strengthened columns and beams. Please see Appendix D for the full set of retrofit drawings.

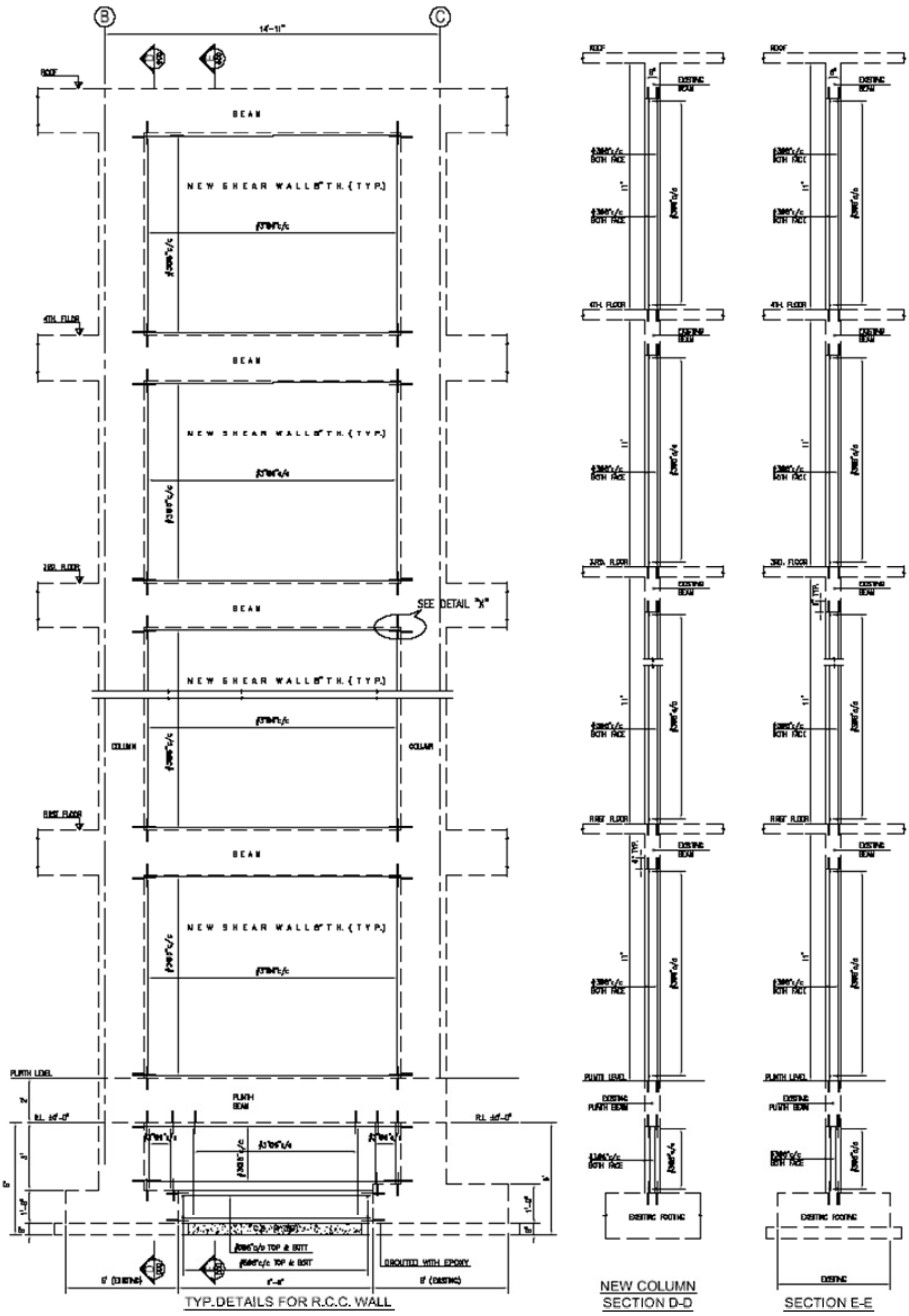


Figure 16. Structural elevation and section of new shear walls

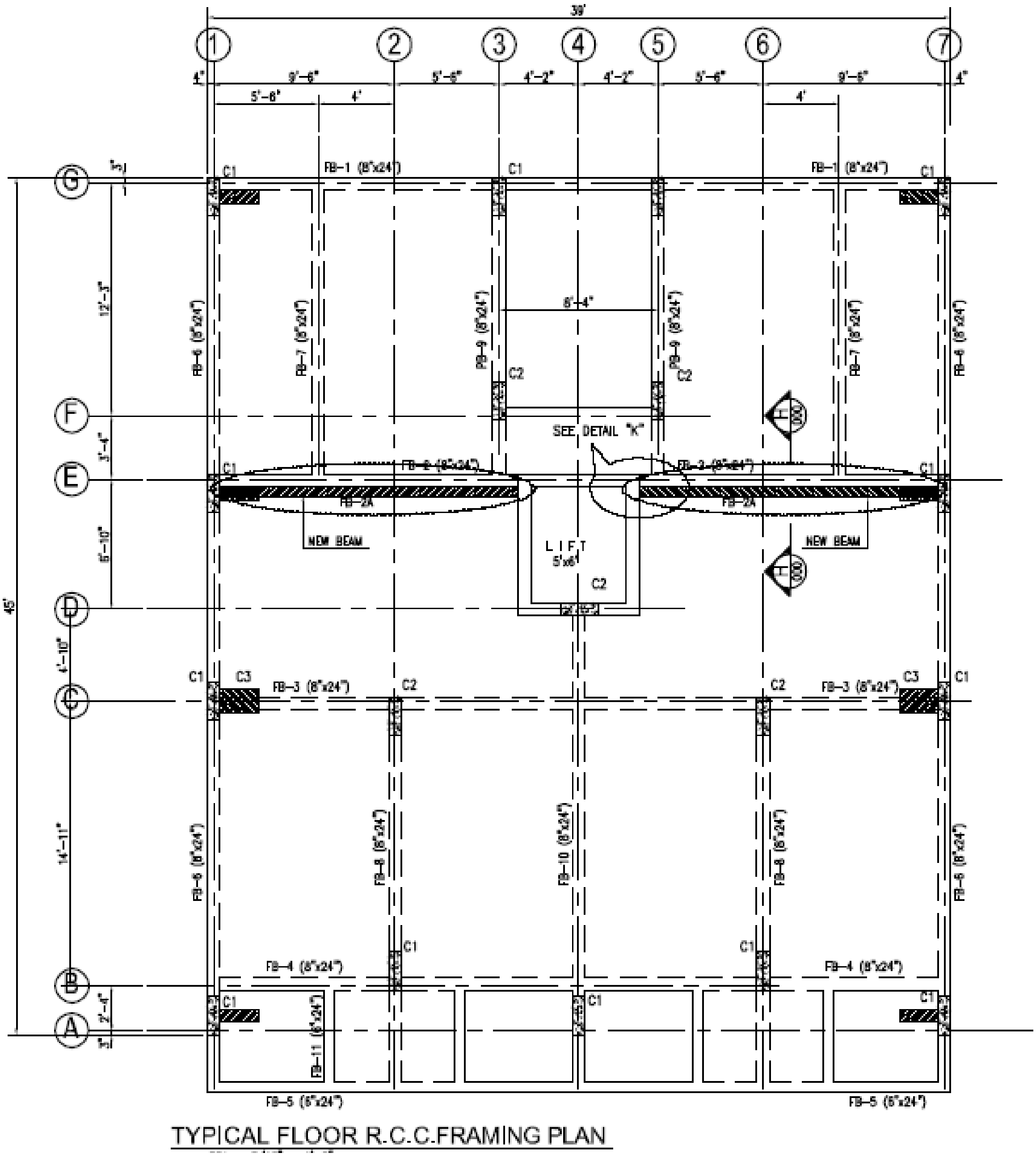


Figure 17. Locations of strengthened beams and columns

Observations and Future Work

This case study building showed the severe seismic vulnerability that can be present in buildings commonly found in Karachi. The building was very weak in one direction and would likely have collapsed in a strong earthquake. Significant efforts are needed to identify these “killer buildings” and mitigate the risks they pose.

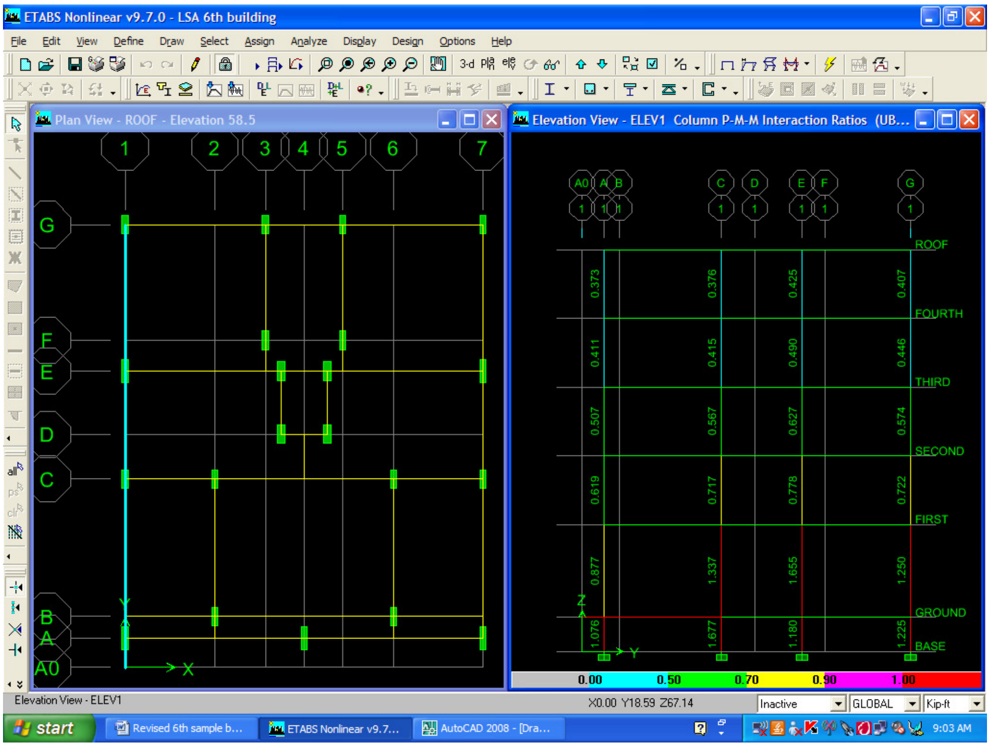
Appendix A: Tier 1 Checklists

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

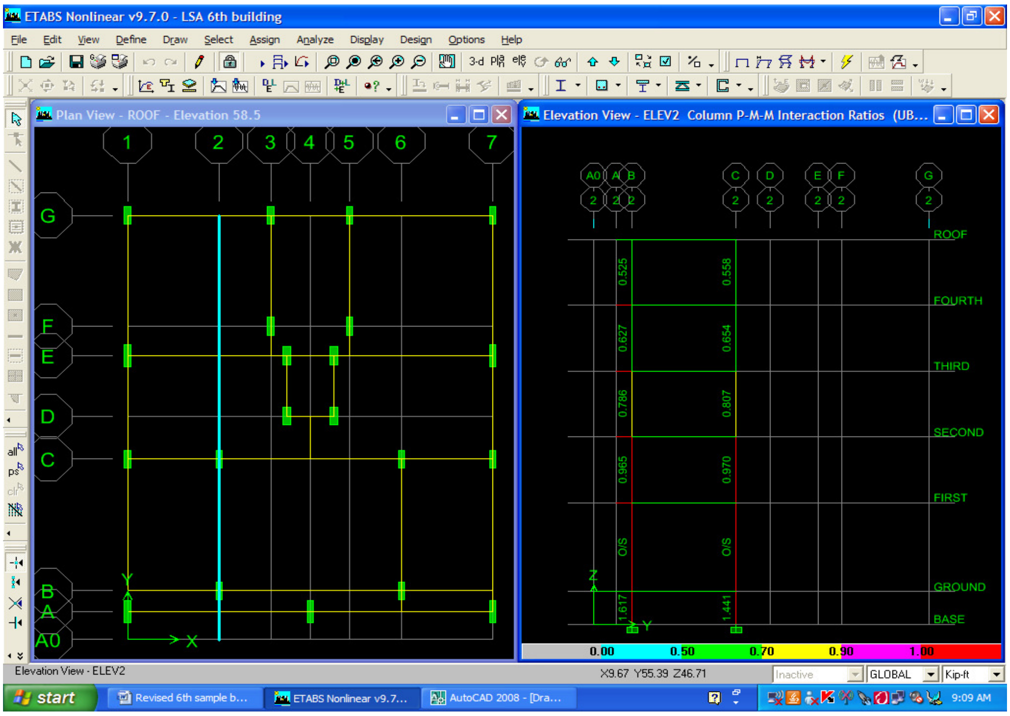
LATERAL-FORCE RESISTING SYSTEM	
Redundancy	C
Interfering Wall	NC
Shear Stress Check	C
Axial Stress Check	C
Proportion of Infill Walls	NC
Concrete Columns	C
Solid Wall	C
Over All Construction Quality	C
Flat Slab Frames	C
Pre-stressed Frames	N/A
Captive Column	C
Column Aspect Ratio	C
No Shear Failure	C
Stirrup and Tie Hooks	C
Deflection Compatibility	N/A
Diaphragm Continuity	C
Plan Irregularity	N/A
Diaphragm Reinforcement at openings	N/A
Transfer to Shear Walls	C
Uplift at Pile Caps	N/A
Strong Column / Weak Beam	NC
Stirrup Spacing	C
Beam Bars	C
Column Bar Splices	C
Beam bar Splices	NC
Column Tie Spacing	NC
Joint Reinforcement	NC
Joint Eccentricity	NC

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

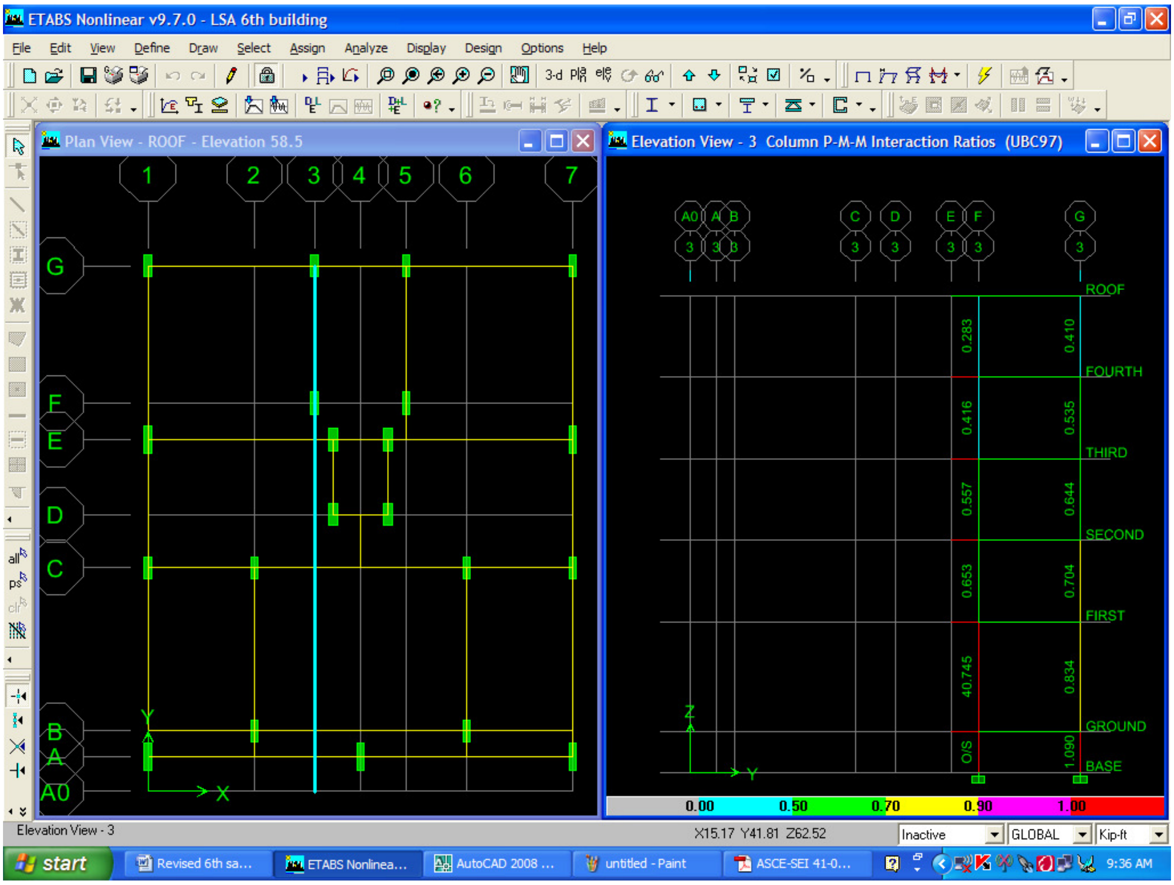
Appendix B: Linear Analysis (Tier 2) Results



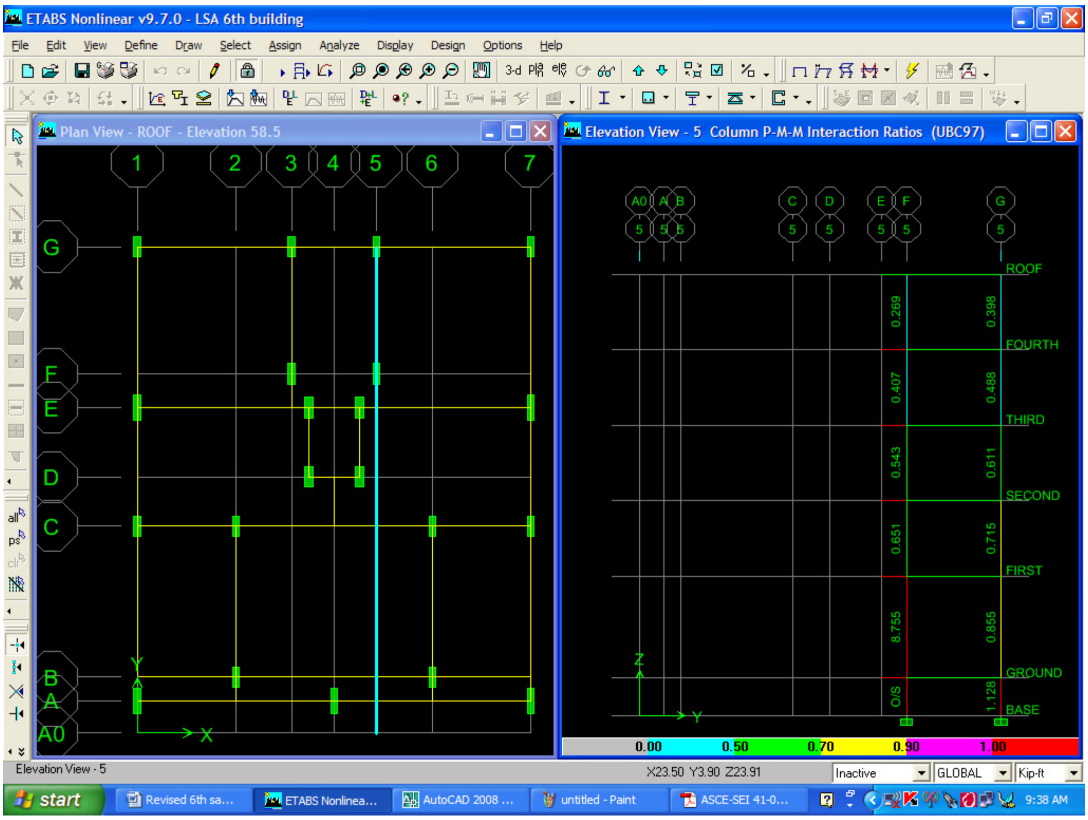
Demand/Capacity Ratios for Frame at Grid-1



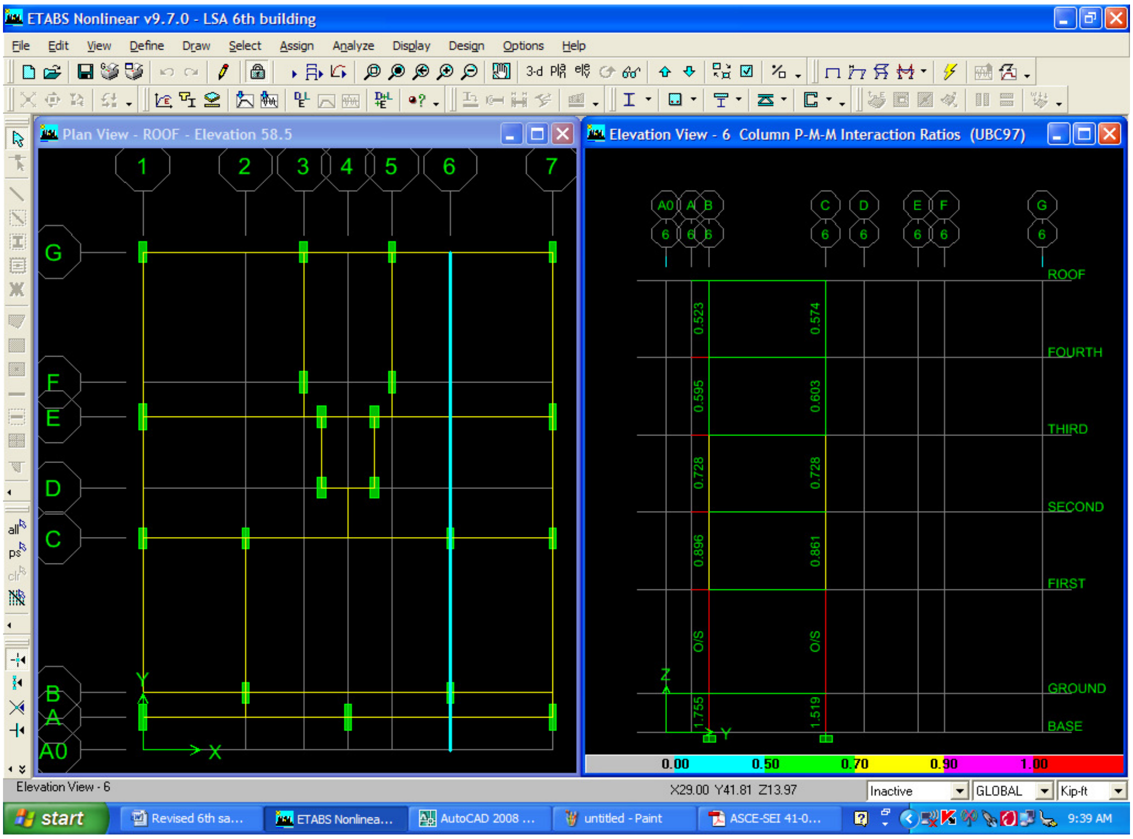
Demand/Capacity Ratios for Frame at Grid-2



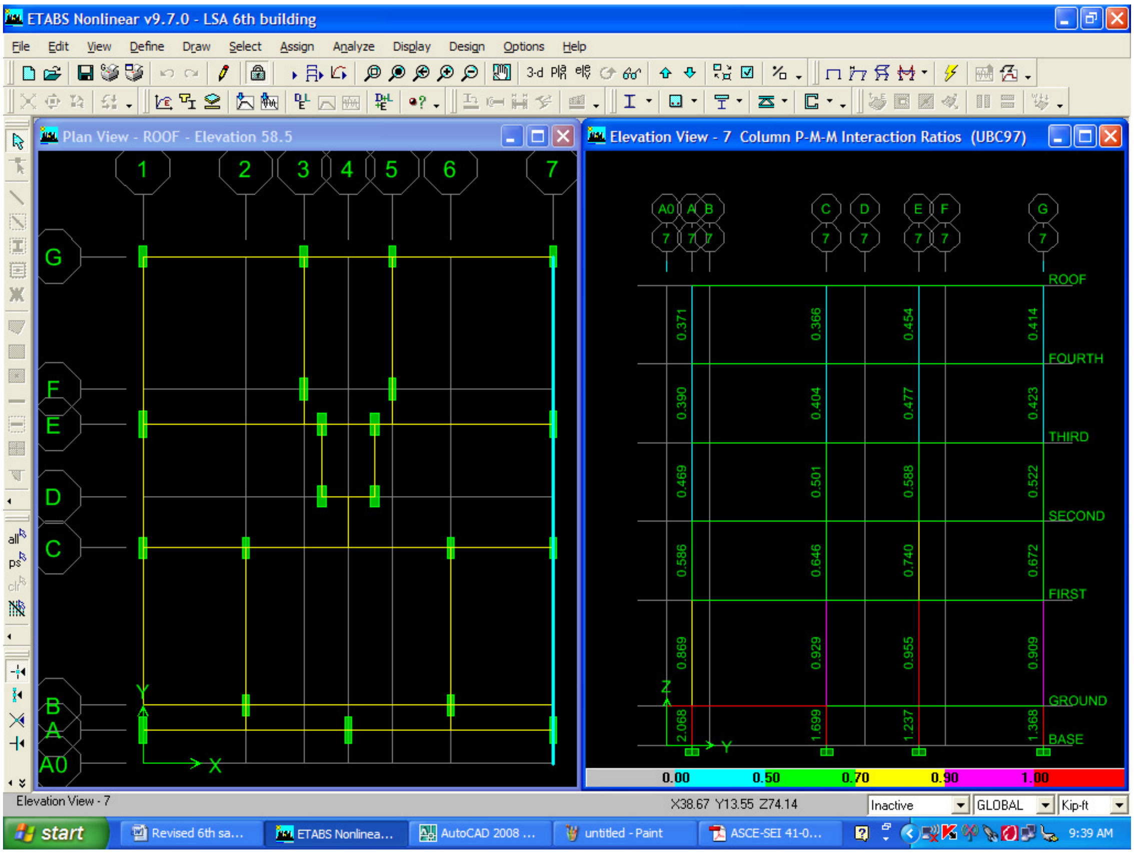
Demand/Capacity Ratios for Frame at Grid-3



Demand/Capacity Ratios for Frame at Grid-5

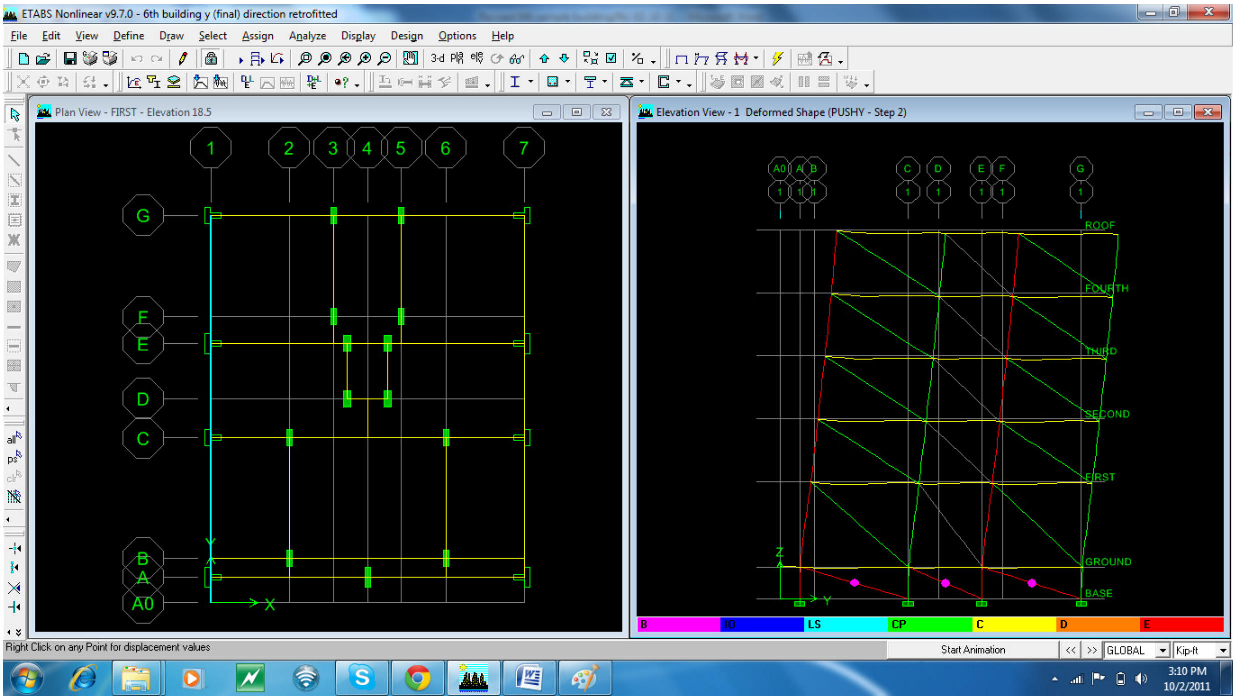


Demand/Capacity Ratios for Frame at Grid-6

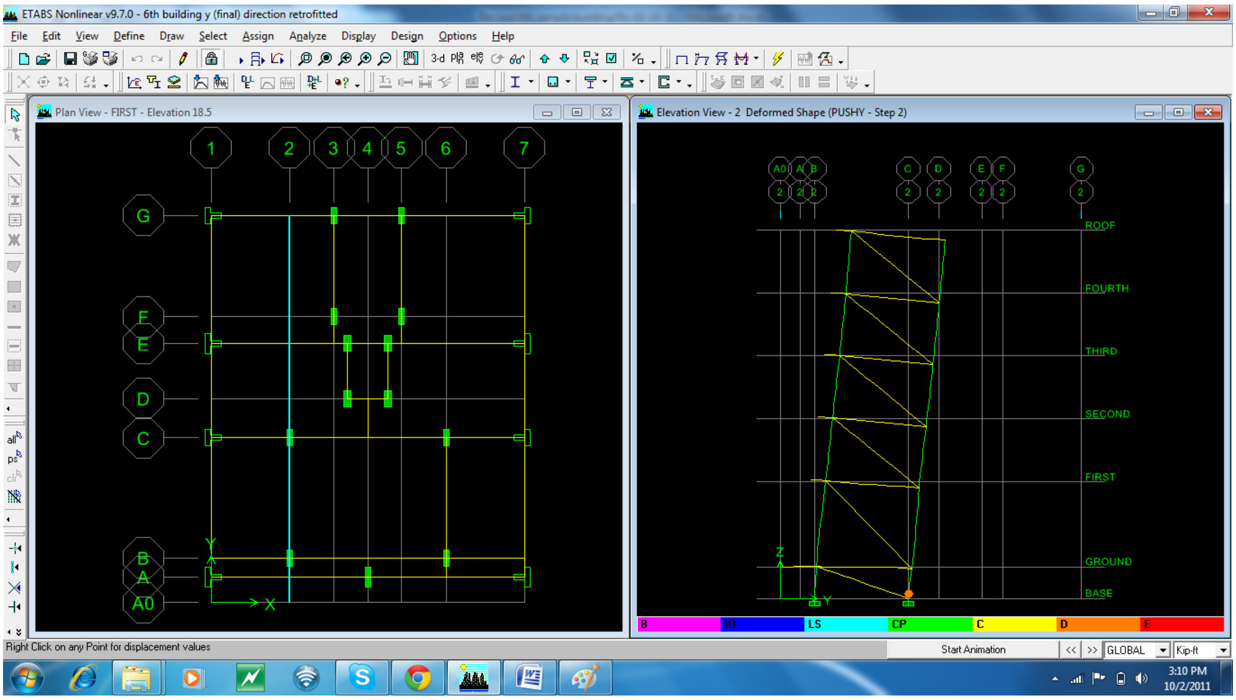


Demand/Capacity Ratios for Frame at Grid-7

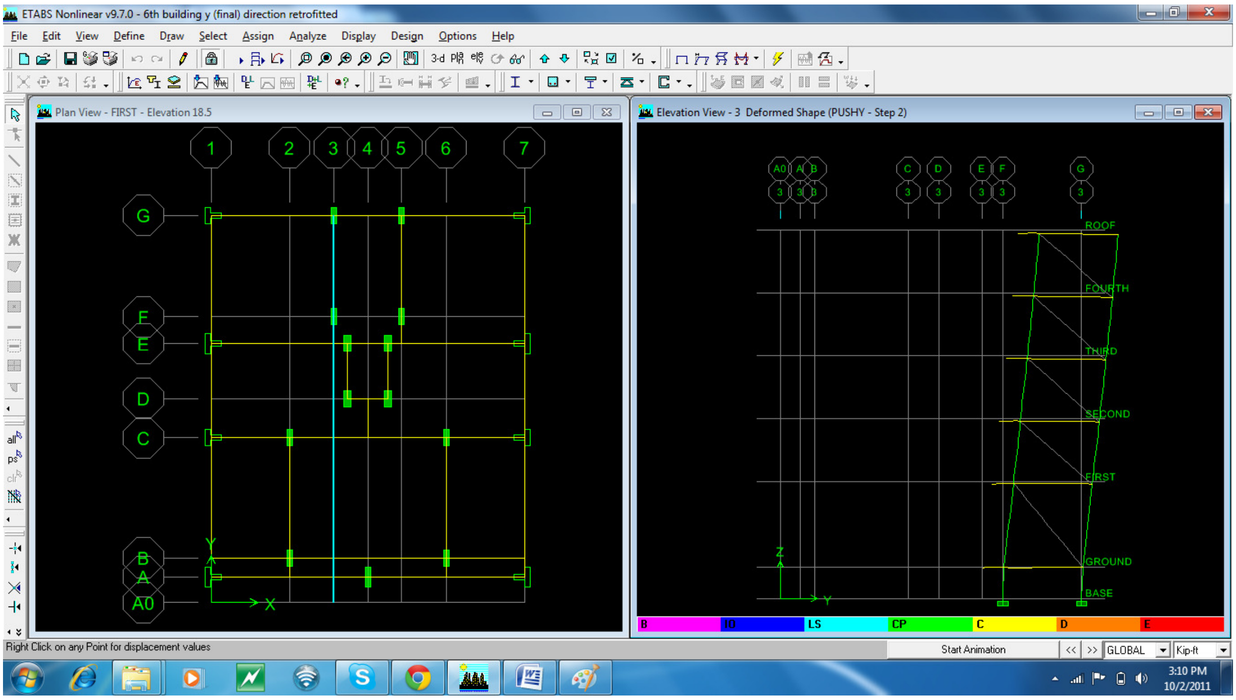
Appendix C: Non-linear Analysis (Tier 3) Results for Retrofitted solution



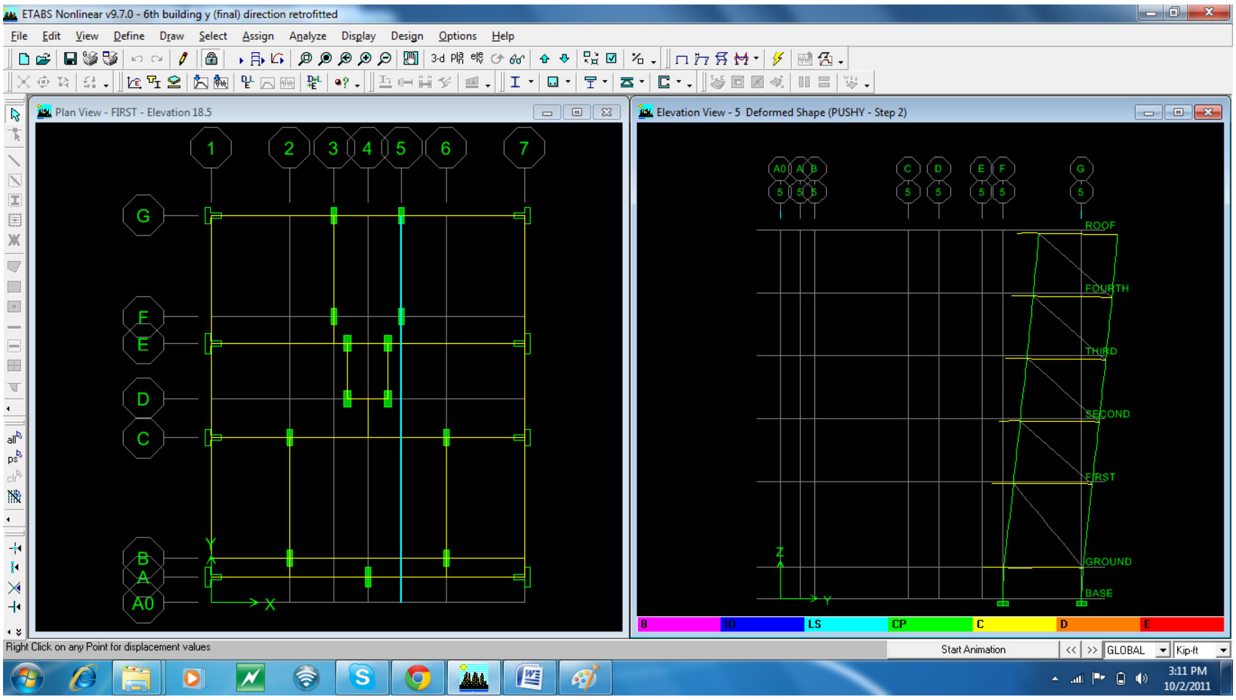
Mechanism or Deformed shapes at Performance Point for grid-1



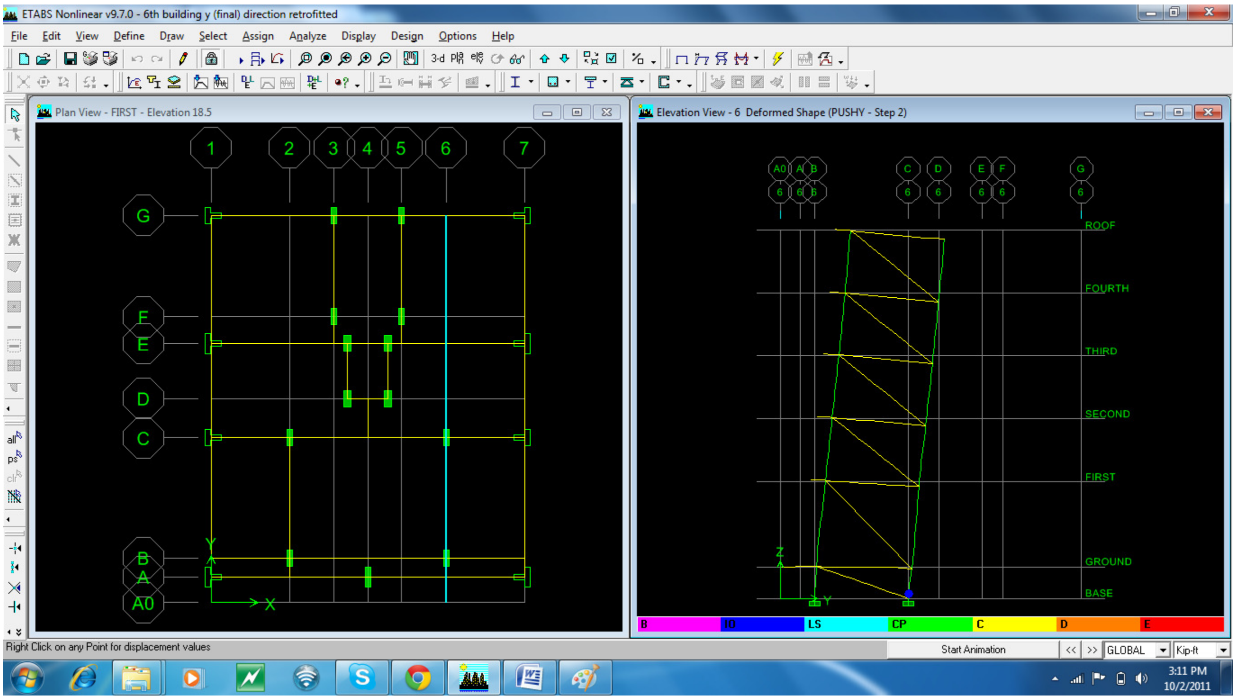
Mechanism or Deformed shapes at Performance Point for grid-2



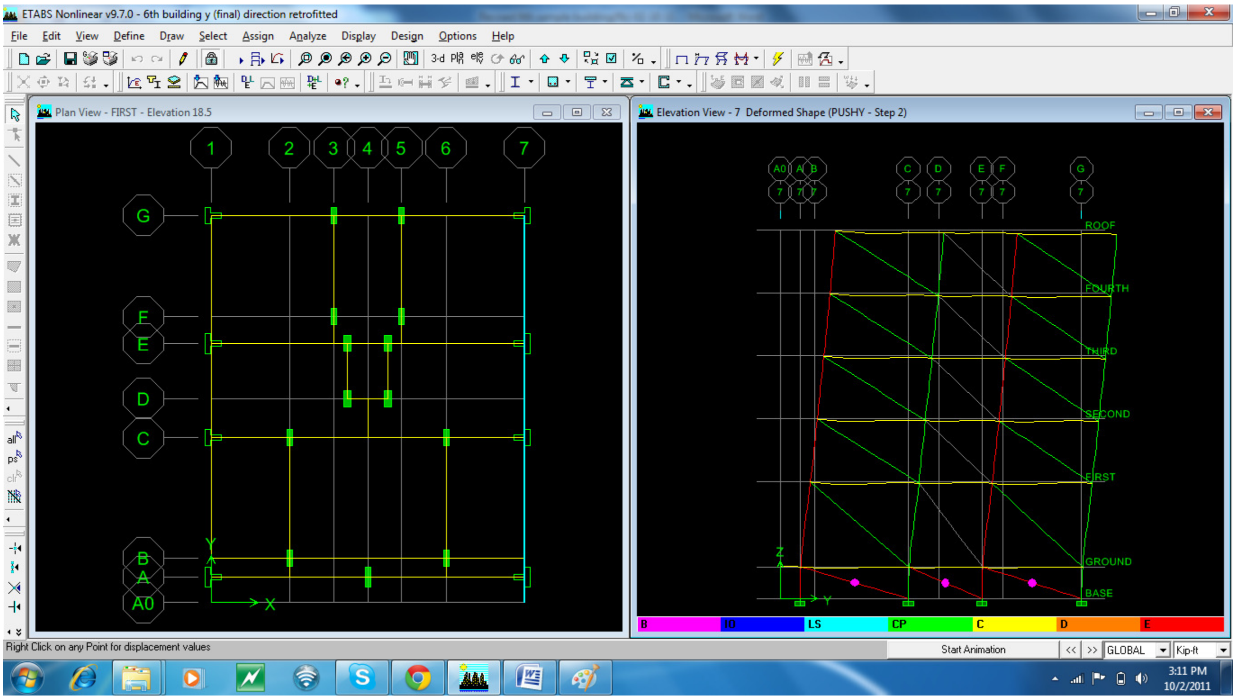
Mechanism or Deformed shapes at Performance Point for grid-3



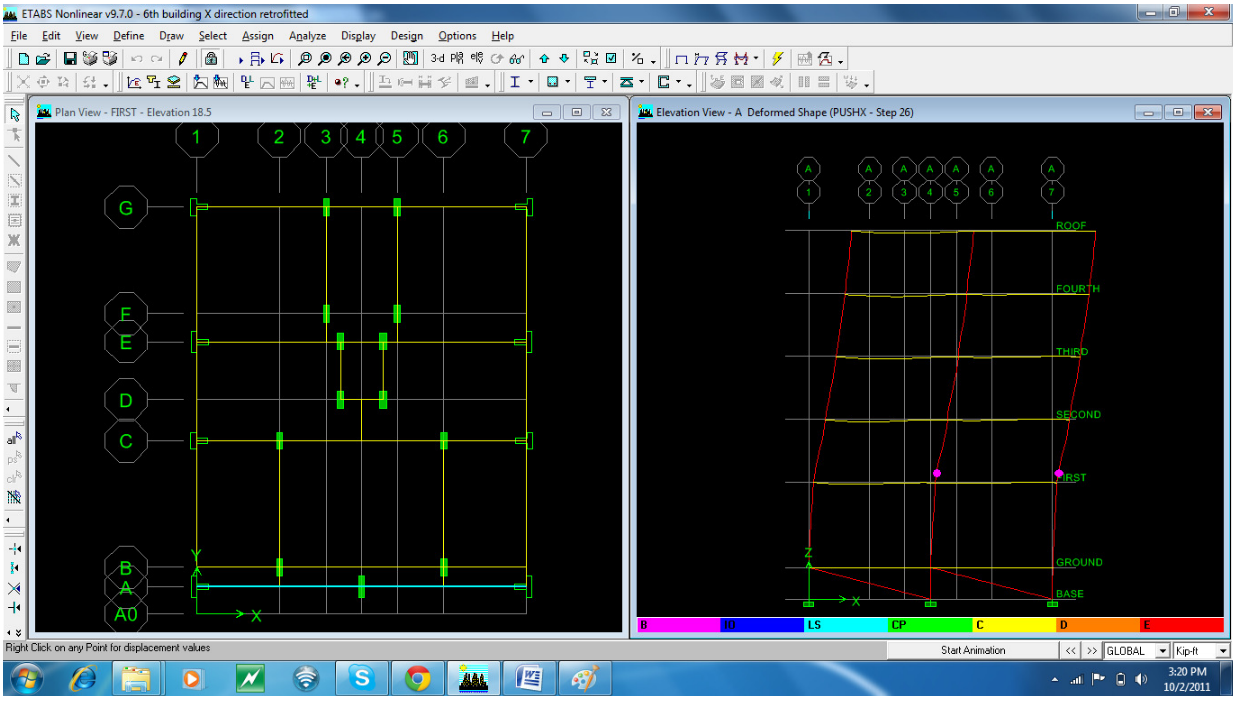
Mechanism or Deformed shapes at Performance Point for grid-5



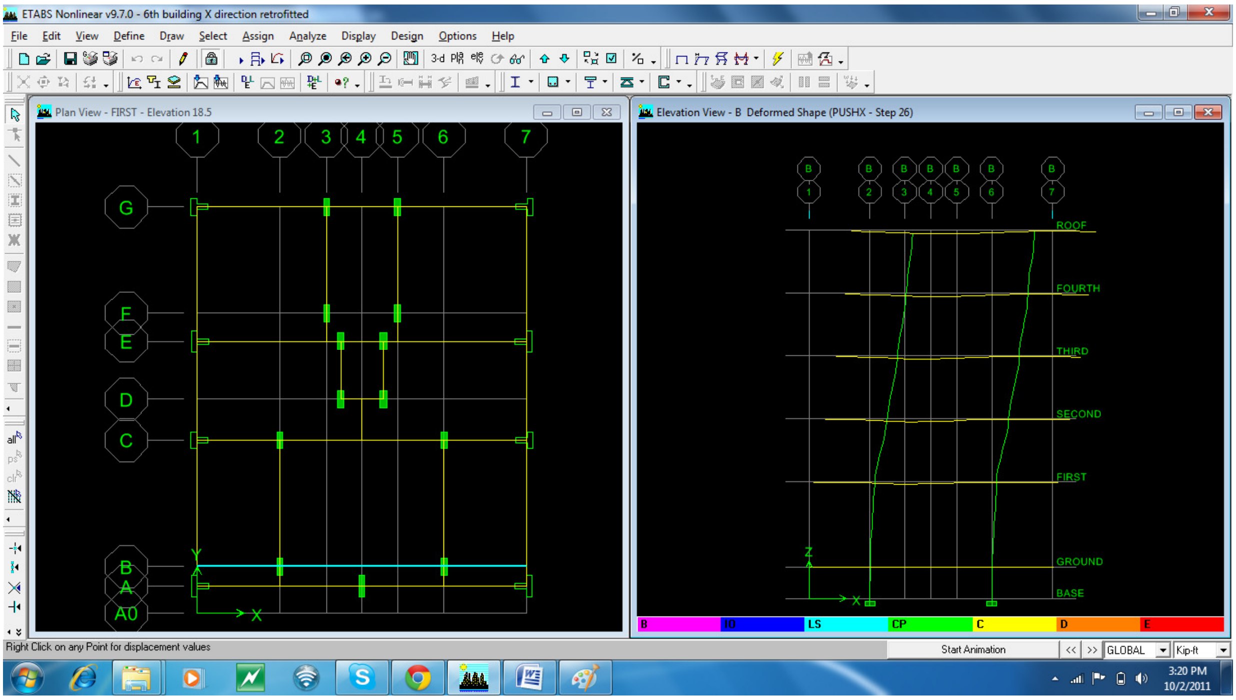
Mechanism or Deformed shapes at Performance Point for grid-6



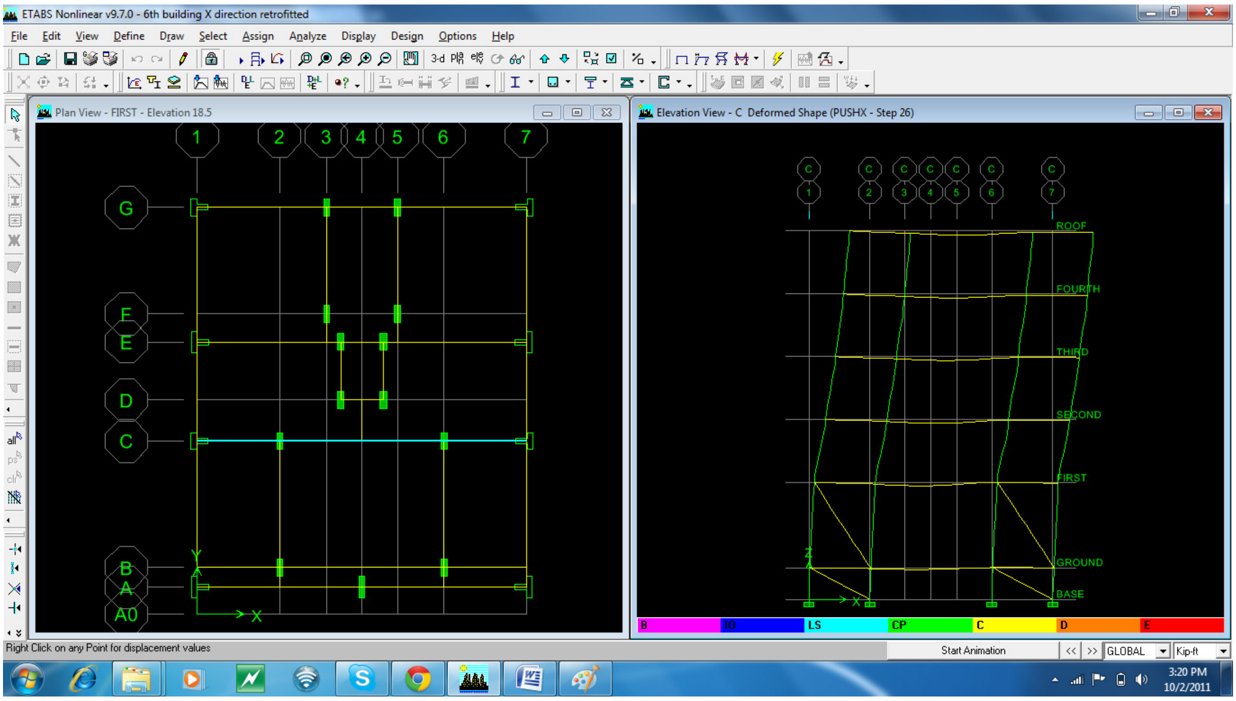
Mechanism or Deformed shapes at Performance Point for grid-7



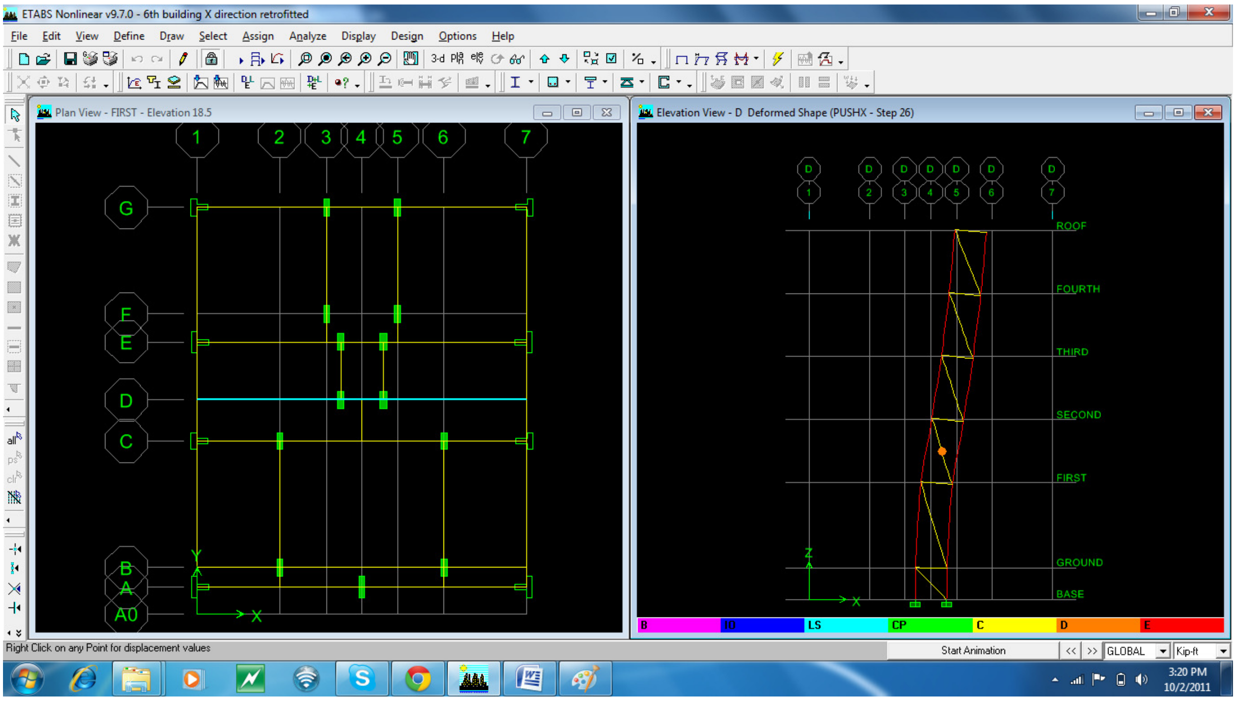
Mechanism or Deformed shapes at Performance Point for grid-A



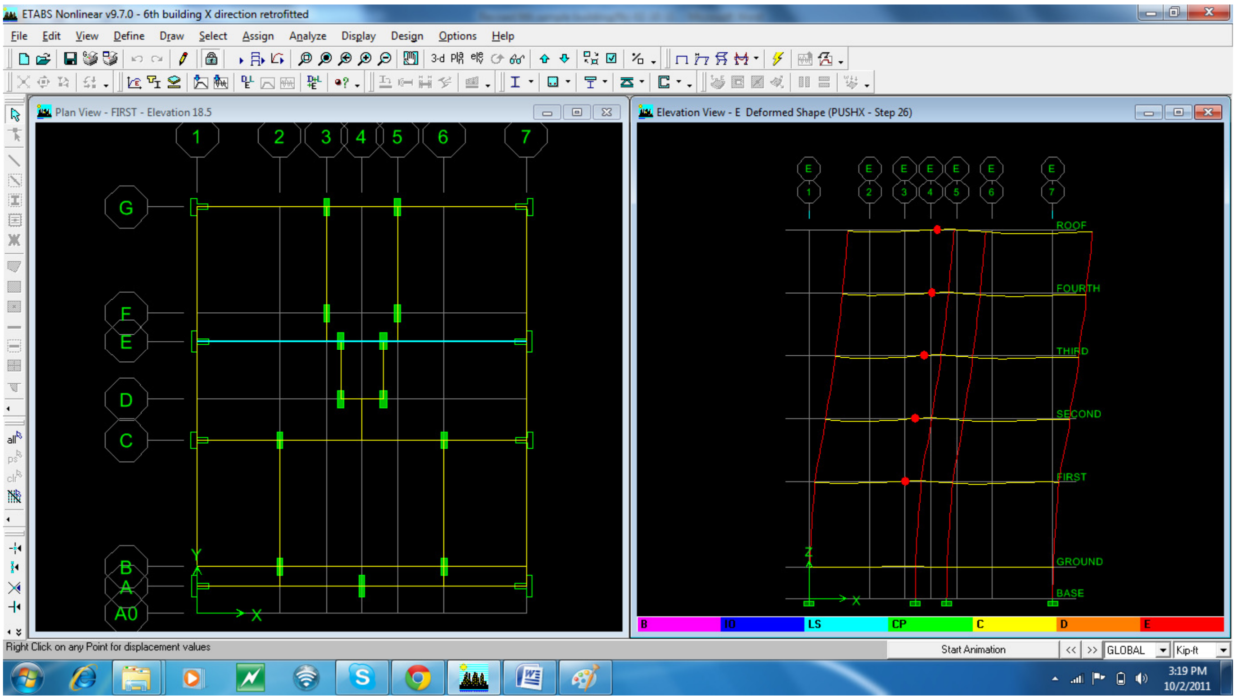
Mechanism or Deformed shapes at Performance Point for grid-B



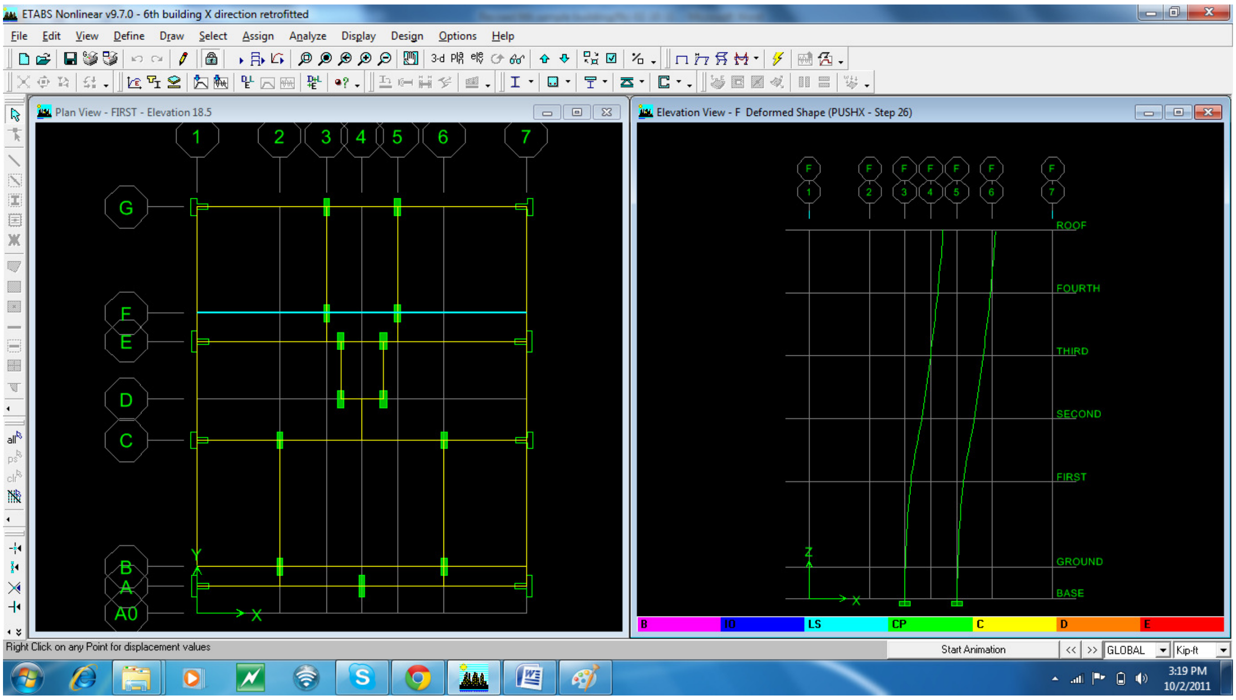
Mechanism or Deformed shapes at Performance Point for grid-C



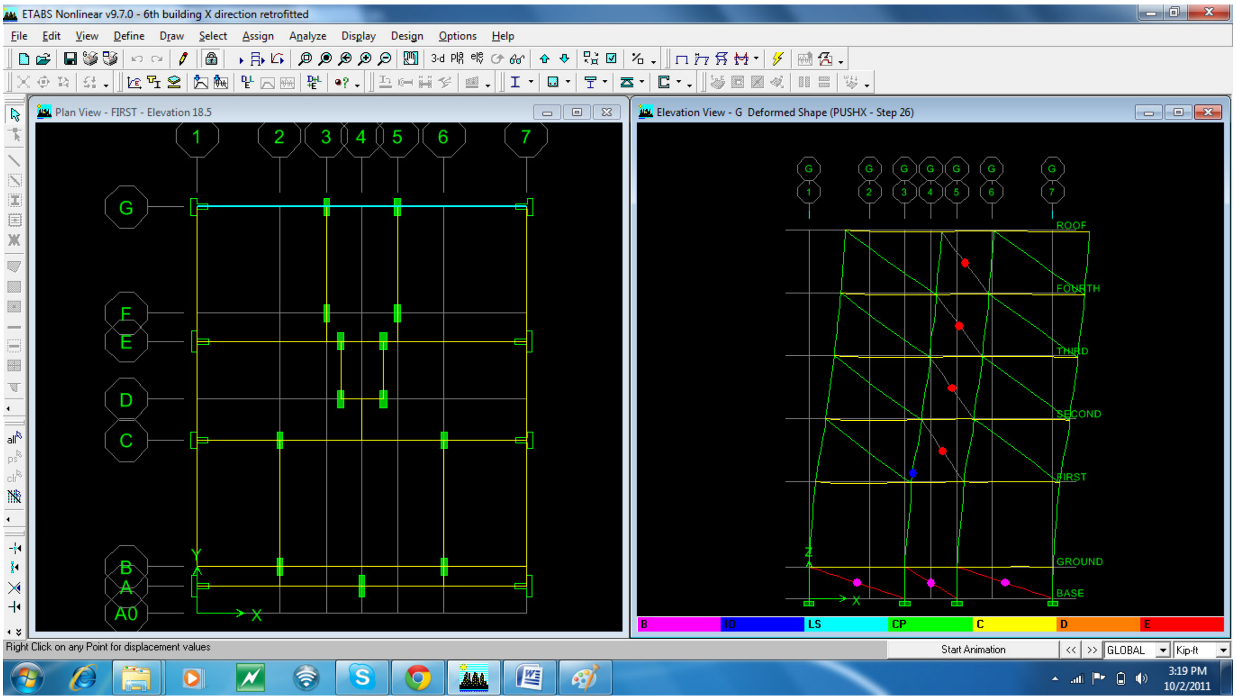
Mechanism or Deformed shapes at Performance Point for grid-D



Mechanism or Deformed shapes at Performance Point for grid-E



Mechanism or Deformed shapes at Performance Point for grid-F



Mechanism or Deformed shapes at Performance Point for grid-G

Appendix D: Retrofit Drawings