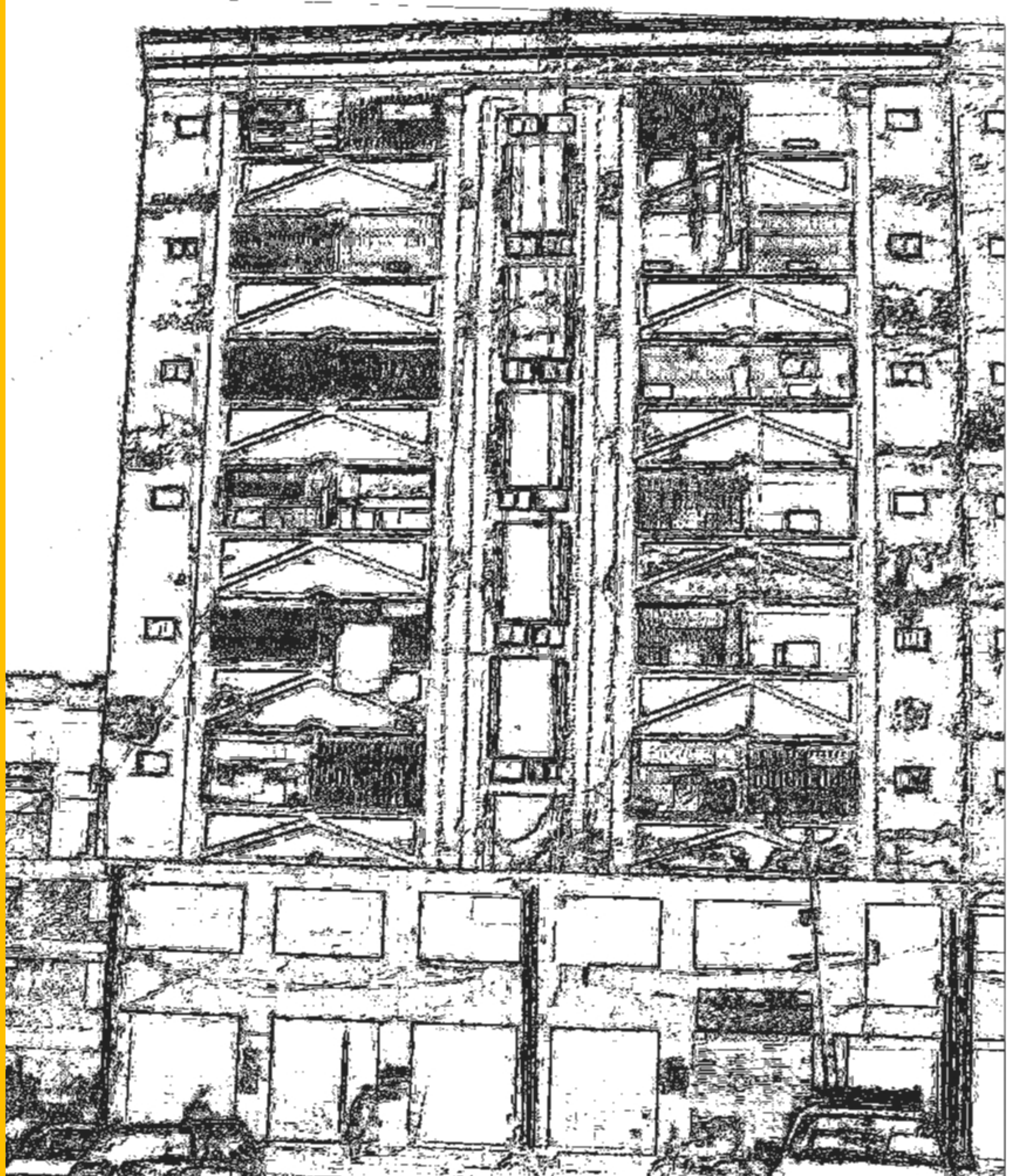

8-Storey Mixed Use Building in Karachi

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

The building is located in Gulistan-e-Johar, a densely populated area in Karachi. It is a reinforced concrete 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 building was constructed after 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 potential weak story created by open shop fronts at the ground floor, an eccentrically located reinforced concrete core, 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 Tier 3 nonlinear static pushover analyses showed that the building would be heavily damaged in the maximum considered earthquake (seismic Zone 4), but would be unlikely to collapse. Hand calculations determined that the beam-column joints have insufficient shear strength and are likely to experience significant damage. The case study team advisors considered it unlikely that the joints would deteriorate enough to cause collapse, however.

Because the building is a residential building in which it would be difficult to seismically retrofit the joints (joint retrofit schemes tend to be invasive), and because it is being evaluated for collapse prevention in the maximum considered earthquake, the case study team and advisors determined that the most practical course of action would be to leave the building as it is, and not attempt a retrofit of the beam column joints that would be disruptive to occupants. This case study illustrates the benefit of nonlinear analysis in capturing the existing strength and deformation capacity of a building to reduce, or in this case eliminate, potentially costly and disruptive seismic retrofit measures.

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 a case study team consisting of Ms. Tehmina Ayub, Assistant Professor, Mr. Aslam Faqeer Mohammed, Assistant Professor, Mr. Adnan Rais Ahmed, Lecturer, Department of Civil Engineering, NED University of Engineering and Technology, and Mr. Nadeem Manzoor Hassan, CEO and Partner, Tameeriat-e-Jadid and Times Construction.

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 by the team members gave them the opportunity to evaluate a real, commonly encountered building type 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 gave the case study team members the opportunity to gain experience using ETABS, computer analysis software from Computers and Structures, Inc. of Berkeley, California, and to better understand how to apply ASCE/SEI 31-03, ASCE/SEI 41-06 and other associated documents to buildings in Pakistan.

Building Information

The building, shown in Figure 1, is an eight storey (ground plus seven) mixed use apartment building with shops at the ground floor and offices at the mezzanine level. The building's overall dimensions are 55'-0" by 75'-0", and it is approximately 90 feet tall. The building has a reinforced concrete moment frame structural system with unreinforced concrete block infill walls. The concrete block infill walls are 4 inches thick. The foundations are reinforced concrete spread footings. The building is relatively new but some water damage is visible from the exterior. Some repairs have been made, but no condition assessments have been made.

Figure 1. Front elevation view (left) – the case study building is on the left; side elevation view (right)

The building's architectural and structural drawings are shown in Figure 2 through Figure 7. 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.

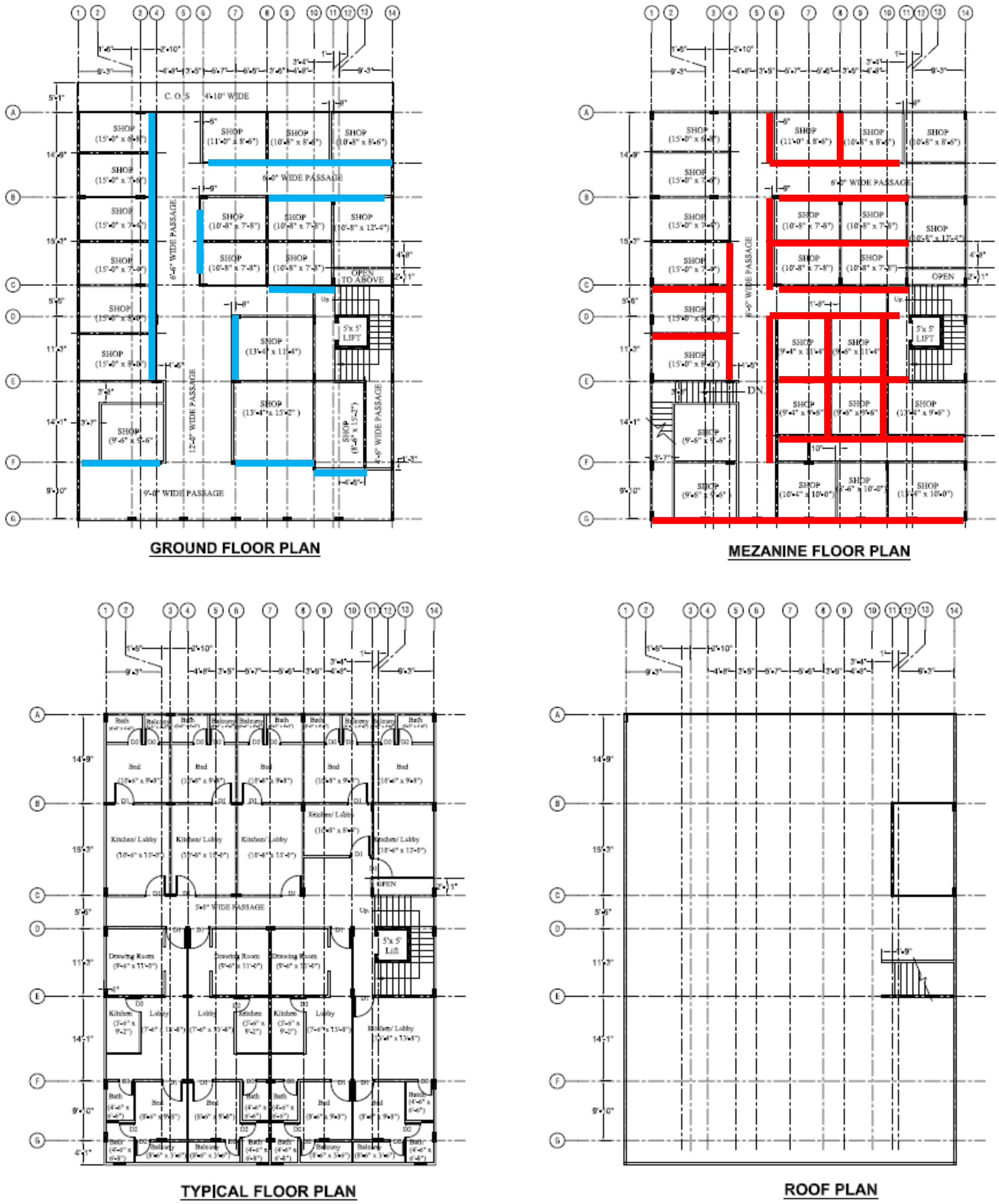


Figure 2. Architectural plans of ground floor, mezzanine, typical upper residential floor, and roof. Blue and red lines indicate locations where unreinforced masonry infill walls have been removed in commercial areas.

Figure 3. Architectural elevation (front) and typical section

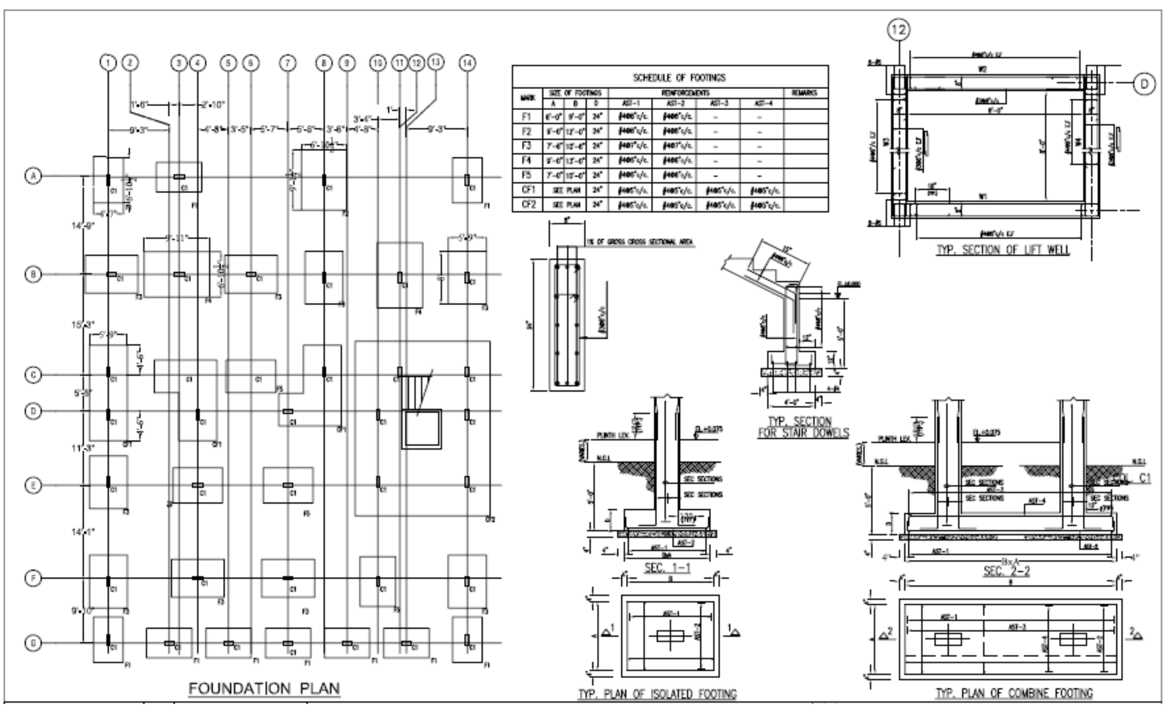


Figure 4. Structural drawings for foundation

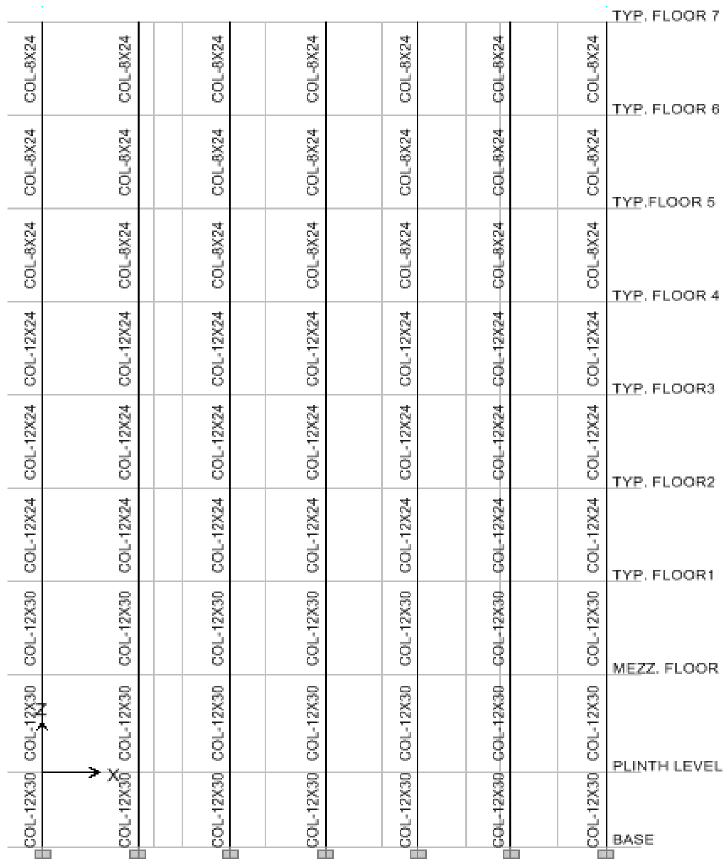


Figure 5. Column sizes from foundation to roof level

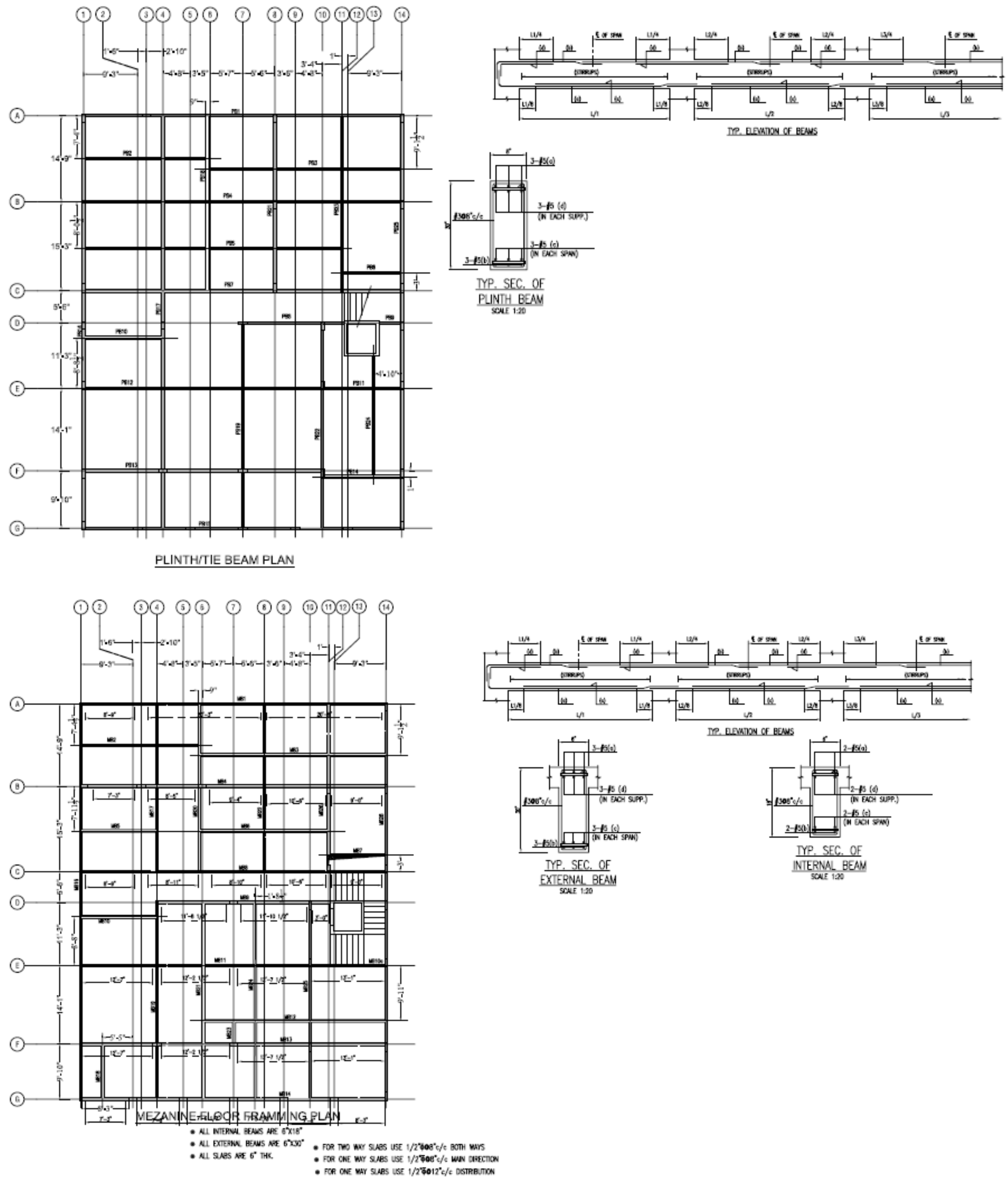


Figure 6. Structural framing for plinth level (top) and mezzanine level (bottom)

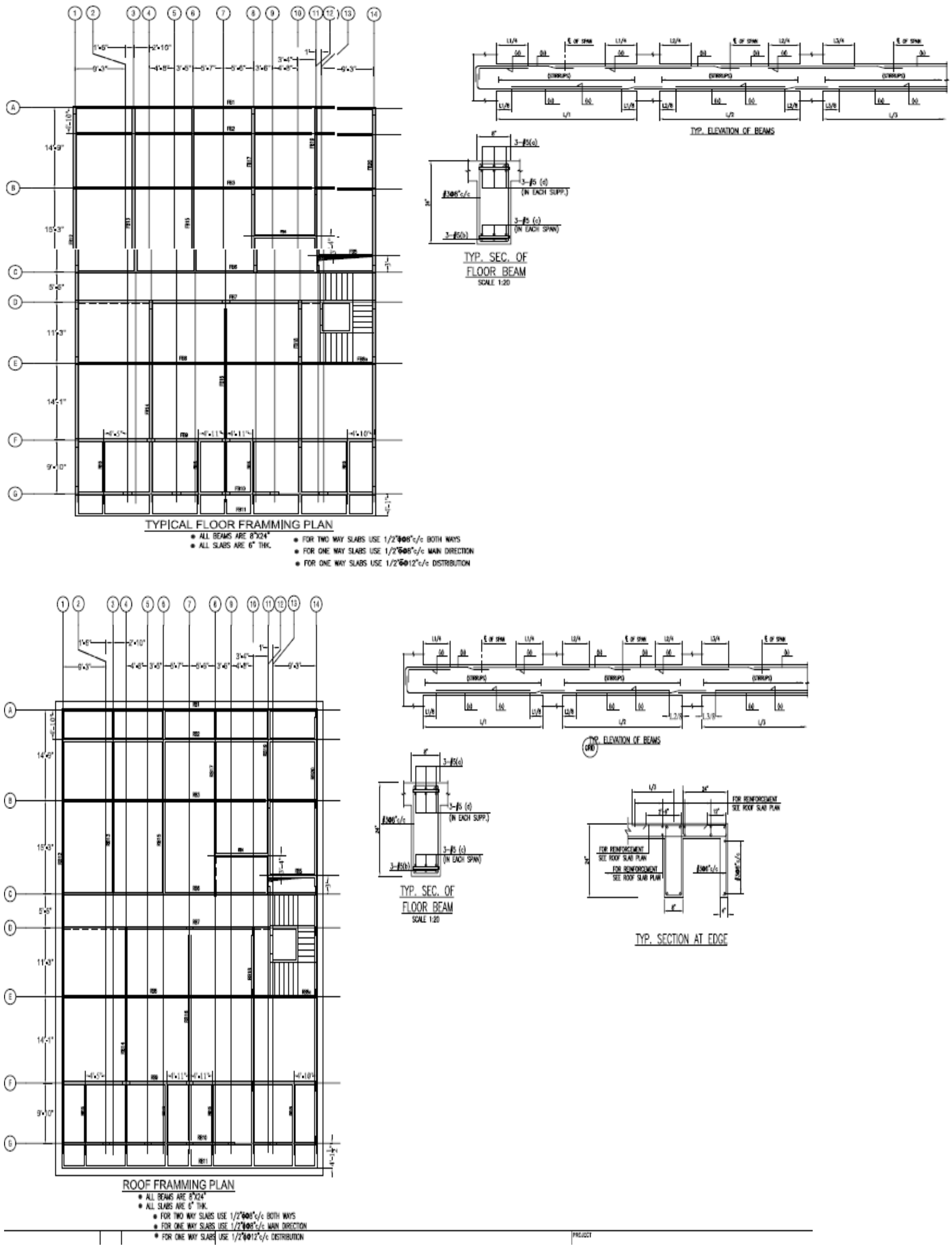


Figure 7. Structural framing for typical residential floor (top) and roof (bottom)

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	Adjacent building Soft storey Weak storey Vertical discontinuity Mass irregularity Torsion irregularity Deterioration
Lateral Force-resisting System	Interfering wall Shear stress check Axial stress check
Geologic Hazards and Foundation	None

Linear Evaluation

Figure 8 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.

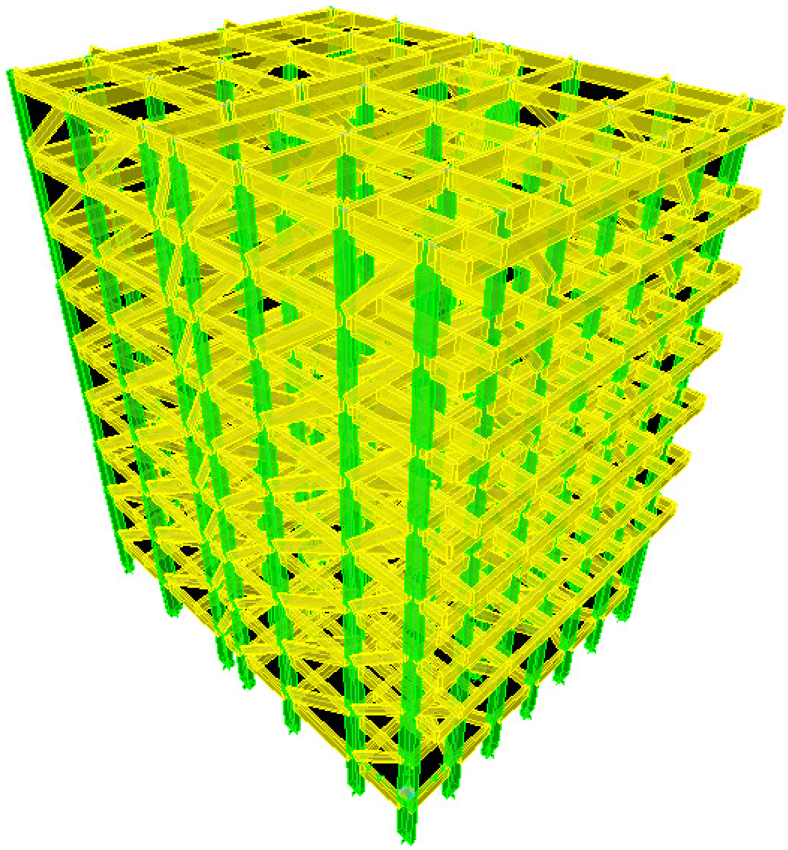


Figure 8. Rendering of linear ETABS model of the building

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), soft storey (shown in Table 2), and storey drift (shown in Table 3).

Table 1. Torsion irregularity check

Story	Diaphragm	XCM	YCM	XCR	YCR	% diff X (allow 20%)	% diff Y (allow 20%)
TYP. FLOOR 7	D1	337.307	402.057	351.286	400.033	2.1	0.2
TYP. FLOOR 6	D1	338.935	397.396	355.848	397.543	2.6	0.0
TYP.FLOOR 5	D1	338.935	397.396	358.62	394.476	3.0	0.3
TYP. FLOOR 4	D1	338.77	397.145	360.964	390.588	3.4	0.7
TYP. FLOOR3	D1	338.593	396.936	365.967	386.749	4.2	1.1
TYP. FLOOR2	D1	338.593	396.936	373	380.952	5.2	1.8
TYP. FLOOR1	D1	338.491	396.889	381.965	374.198	6.6	2.5
MEZZ. FLOOR	D1	337.331	417.815	388.753	380.266	7.8	4.2
PLINTH LEVEL	D1	335.343	441.55	394.478	350.073	9.0	10.2

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. 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
TYP. FLOOR 7	EX	252	2.654	94.95	----	11.6
TYP. FLOOR 6	EX	213	2.504	85.06	10.4	6.4
TYP.FLOOR 5	EX	183	2.29	79.91	6.1	4.0
TYP. FLOOR 4	EX	155	2.018	76.81	3.9	5.2
TYP. FLOOR3	EX	126	1.725	73.04	4.9	7.5
TYP. FLOOR2	EX	95	1.398	67.95	7.0	9.9
TYP. FLOOR1	EX	64	1.035	61.84	9.0	41.1
MEZZ. FLOOR	EX	26.53	0.6055	43.82	29.1	24.1
PLINTH LEVEL	EX	10	0.1733	57.70	31.7	----

Story	Load	storey force	Total Displacement	Stiffness	% diff in K (30% allow) % difference compare to	
		kips	inches	kip/in	Above storey	Below storey
TYP. FLOOR 7	EY	252	2.586	97.45	----	11.2
TYP. FLOOR 6	EY	213	2.43	87.65	10.0	5.9
TYP.FLOOR 5	EY	183	2.21	82.81	5.5	3.1
TYP. FLOOR 4	EY	155	1.929	80.35	3.0	3.4
TYP. FLOOR3	EY	126	1.622	77.68	3.3	4.7
TYP. FLOOR2	EY	95	1.28	74.22	4.5	5.2
TYP. FLOOR1	EY	64	0.907	70.56	4.9	35.5
MEZZ. FLOOR	EY	26.53	0.5096	52.06	26.2	21.9
PLINTH LEVEL	EY	10	0.15	66.67	28.1	----

Table 2 shows that a few stories do not comply with the stiffness criteria and may be soft storeys.

Table 3. Storey drift check

Story	Etab Drift X	Code Modified Drift	Etab Drift X	Code Modified Drift
	Δ_S	Δ_M	Δ_S	Δ_M
TYP. FLOOR 7	0.001288	0.00496	0.001367	0.00526
TYP. FLOOR 6	0.001889	0.00727	0.001926	0.00742
TYP.FLOOR 5	0.002442	0.00940	0.002445	0.00941
TYP. FLOOR 4	0.002648	0.01019	0.002701	0.01040
TYP. FLOOR3	0.002983	0.01148	0.003009	0.01158
TYP. FLOOR2	0.003259	0.01255	0.003279	0.01262
TYP. FLOOR1	0.003522	0.01356	0.003437	0.01323
MEZZ. FLOOR	0.00329	0.01267	0.002916	0.01123
PLINTH LEVEL	0.002089	0.00804	0.001609	0.00619

Table 3 shows that the calculated interstorey drifts in all storeys are less than the allowable drift limit of 0.02.

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 > 1 but less than 2. This required further non linear static analysis. Nonlinear static pushover static analysis based on performance-based seismic design was performed using hinge properties from ATC-40 and ASCE 41-06 criteria manually assigned to beams, columns, and struts in the 3-D model.

Analytical Models

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.

Loading and Performance Criteria

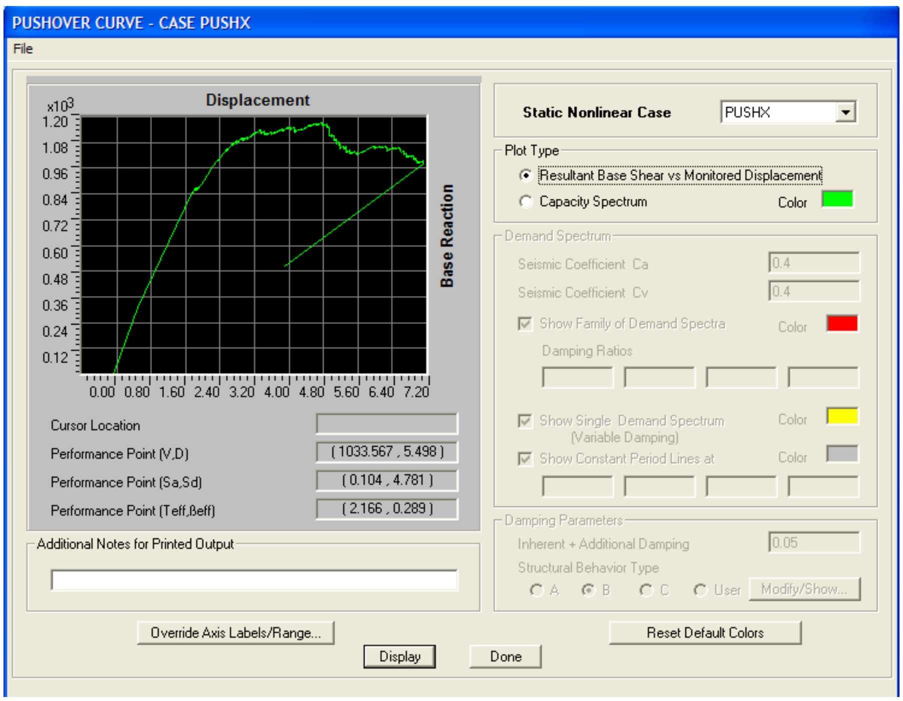
For pushover loading patterns, restart using secant stiffness for member unloading method with P-Delta effects for geometric nonlinearity was considered. A life safety performance criterion was selected for the study building. Table 4 shows ETABS modeling parameters.

Table 4. ETABS modelling parameters

Dead Loads:	Floor slab loads transferred to beam were manually calculated and assigned to each of the beams in the 3-D model. Floor Finishes load was taken as 30 psf and the 4 inch thick masonry infill wall of 50 psf was assigned to the beams where walls were present.
Live Loads:	For shops a load of 50 psf and for residential units a load of 40 psf was used.
Earthquake load:	
Z	0.4g.
R	5.5
C_a	0.4 (Ref: Table 16-Q (UBC 97)) with N_a = 1.0
C_v	0.4 (Ref: Table 16-R (UBC 97) with N_v = 1.0
Soil type	S_B (Ref: Table 16-J UBC-97)
Material properties:	Columns: f'c = 4000 psi, fy = 60,000 psi Beams and slabs: f'c = 3000 psi, fy = 60,000 psi

Analysis Results

Figure 9 shows the load-deformation curve, or *pushover* curve as well as 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. The deformed shapes and state of the nonlinear hinges at the performance point show that the building will be heavily damaged during the maximum considered earthquake, but that it is not likely to collapse. While the beams and infill panels have been pushed far into the nonlinear range and have lost most of their strength, the columns suffer minimal damage. This is a desirable seismic mechanism. Therefore the building as a whole can be considered to attain the collapse prevention performance level, despite individual members exceeding the collapse prevention acceptance criteria in ASCE 41-06. The deformed shape for the frame at the front of the building is shown in Figure 10, with the deformed shapes for the remaining frames shown in Appendix C.



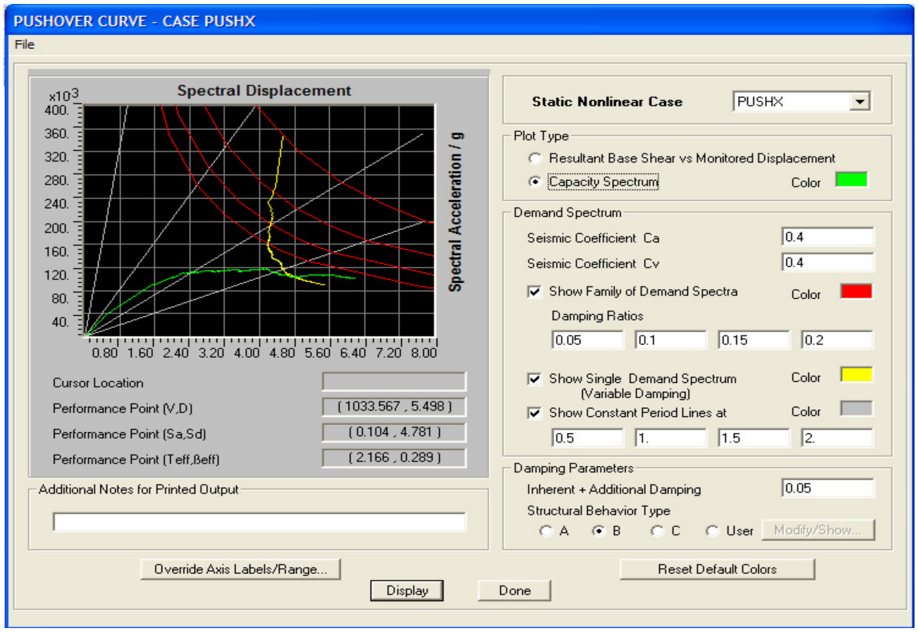


Figure 9. Pushover curve (top) and performance level (bottom) for seismic forces in X-direction

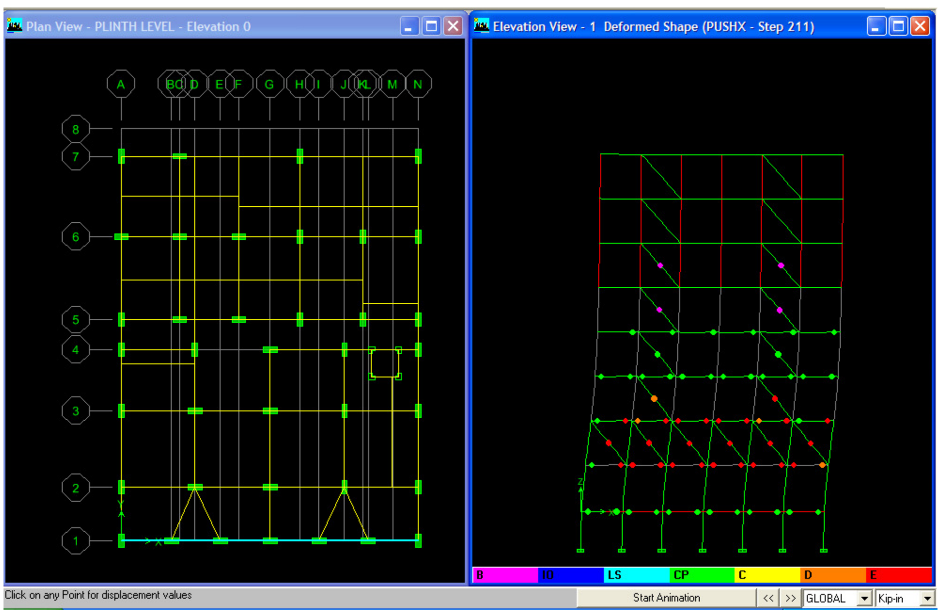


Figure 10. Deformed shape at performance point for frame at grid line 1, with frame location shown at left

Figure 11 and Figure 12 show the pushover curve and performance point for the Y direction, respectively.

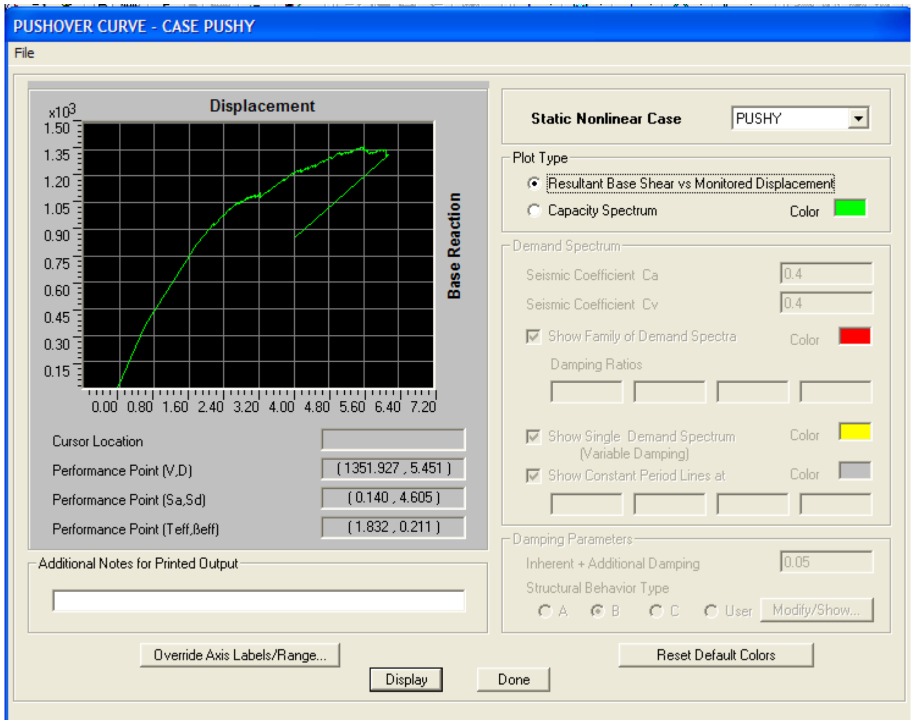


Figure 11. Pushover curve for seismic forces in Y-direction

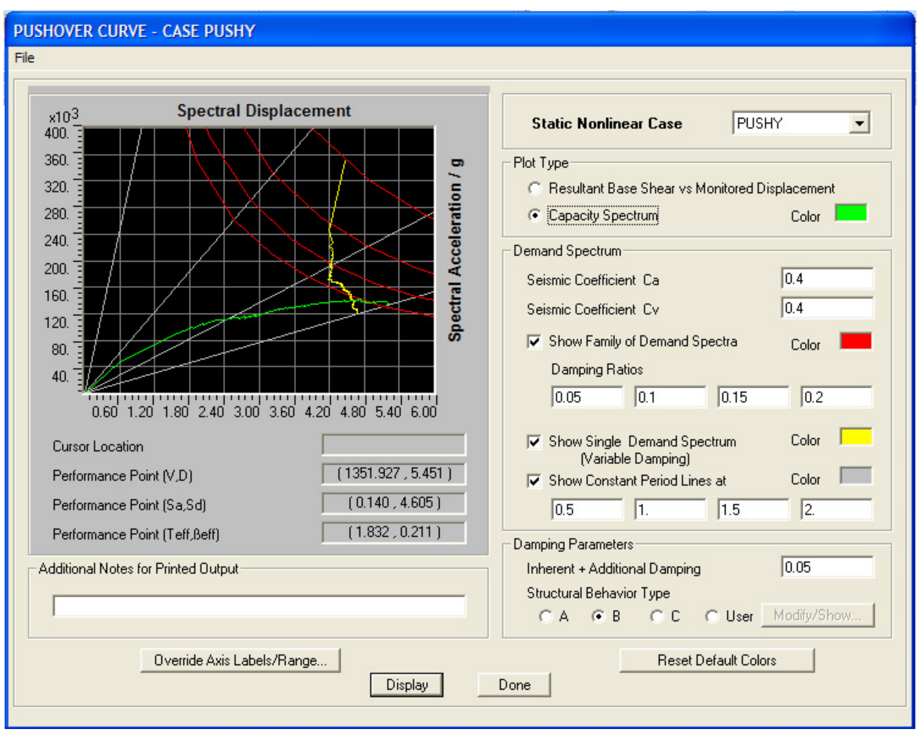


Figure 12. Performance level for seismic forces in Y-direction

Hand Calculation Checks

Joints have no reinforcing because although the column ties are closely spaced at the ends, they do not continue through the joint. In addition, the struts, columns, and beams are connected at the same node in the 3-D model. Therefore one critical joint was checked by hand using the American Concrete Institute (ACI) procedure outlined in ACI 352-02, such that the horizontal strut force is added to the shear force at the performance point and compared to the shear capacity of column, to see whether there will be a problem with the struts failing the columns or the joints failing in shear. The following equations from ACI 352-02 were applied to the joint shown in Figure 13, using the value of γ from ACI 352-02 Table 6-10, shown in Figure 14.

$$M_{pr,b} = A_s \alpha f_y\left(d - \frac{a}{2}\right), \quad T_u = A_s \alpha f_y, \quad V_u = T_u - V_{col}, \quad V_n = \gamma\sqrt{f'_c}\, b_j h_c$$

$$b_j \le \begin{cases} \dfrac{b_c + b_b}{2} \\ b_b + \sum \dfrac{m \cdot h_c}{2} \\ b_c \end{cases}$$

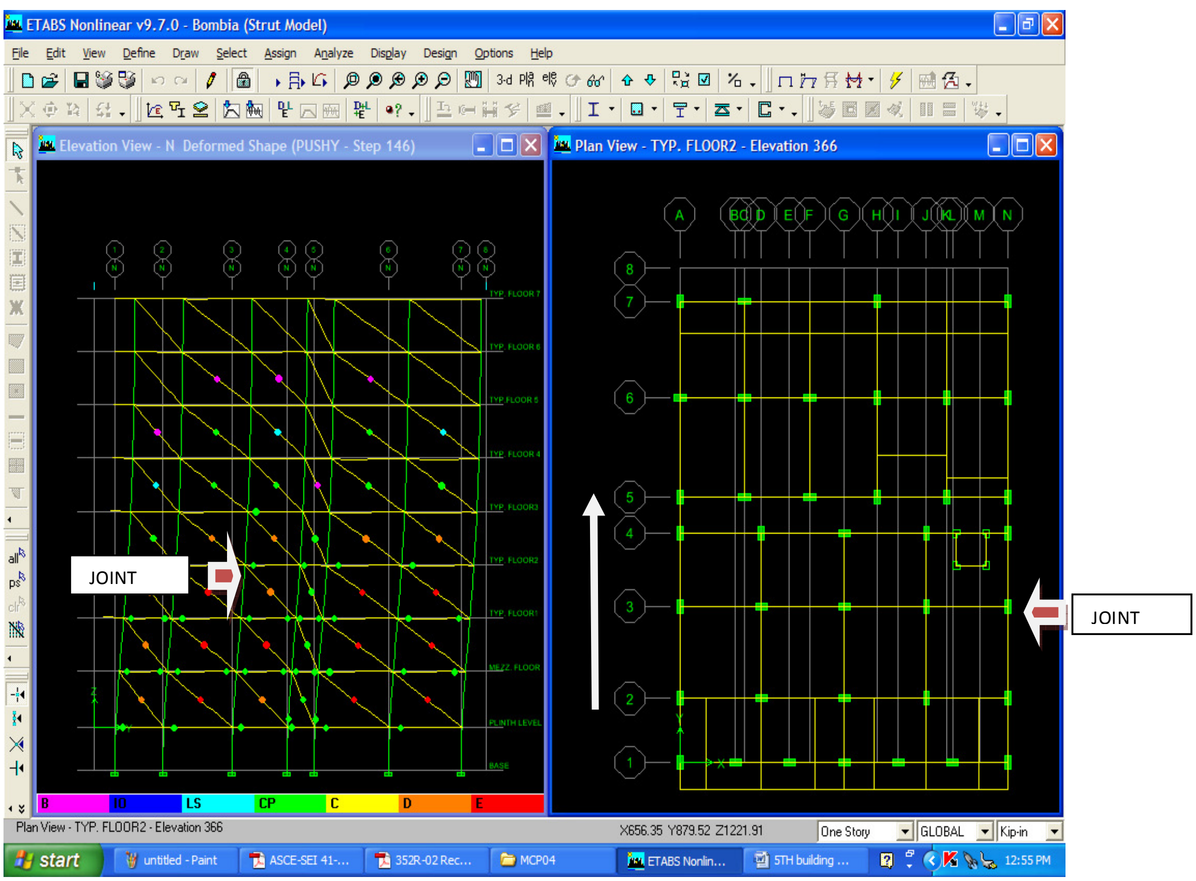


Figure 13. Location of the column joint

Table 6-10. Values of γ for Joint Strength Calculation

	Value of γ				
ρ''^1	Interior Joint with Transverse Beams	Interior Joint without Transverse Beams	Exterior Joint with Transverse Beams	Exterior Joint without Transverse Beams	Knee Joint with or without Transverse Beams
< 0.003	12	10	8	6	4
≥ 0.003	20	15	15	12	8

$^1\rho''$ = volumetric ratio of horizontal confinement reinforcement in the joint.

Figure 14. Value of γ used for joint shear check, from ACI 352-02

Joint capacity:

Column 12”x30” (from base to 1st)

Maximum shear = 65 kips

Added shear from strut = 3 cos(48) kips = 2kips (Because the strut fails at this point)

Total Shear in column = 64 kips

Capacity of column = 88.5 kips

Column 12”x24” (from 1st to 4th)

Maximum shear = 35 kips

Added shear from strut = 2 cos(48) kips = 1.4kips (Because the strut fails at this point)

Total Shear in column = 37 kips

Capacity of column = 70 kips

Column 8”x24” (from 4th to Roof)

Maximum shear = 15 kips

Capacity of column = 62 kips

In all the cases above, the joint shear capacity at the ground, first to fourth, and fourth to roof columns (88.5 kips, 70 kips and 62 kips respectively) is less than the demand, so a joint retrofit would normally be recommended. However, because this is a residential building, joint retrofits would be highly disruptive and therefore impractical. Also, because the demand-capacity ratios for the joints are less than 2, it is unlikely that the joints would be damaged enough to cause the building to collapse. Because the building is being evaluated for the Maximum Considered Earthquake, collapse prevention performance is acceptable and retrofit of the joints is not required.

Results Summary

Nonlinear static pushover analysis when combined with engineering judgment regarding the performance of the beam-column joints, leads to the expectation that the building would suffer major damage in the Maximum Considered Earthquake, but would be unlikely to collapse. This level of performance would be acceptable in most cases due to the expense and difficulty of retrofitting joints in a residential building. Because the joints are not reinforced and do not have adequate shear capacity, a retrofit either of the joints or to provide new elements that would provide additional capacity would improve the building's performance in a large earthquake.

In the event that the building owner would not be satisfied with collapse prevention only, the building could be retrofitted using the conceptual scheme shown in Appendix D. Though there is still significant damage in beams and infill walls, the retrofit improves building performance and reduces the tendency of deformations to concentrate in the lower stories, thus increasing the margin of safety against collapse.

Observations and Future Work

This case study building demonstrates the power of nonlinear analysis to make full use of the existing capacity of a building. The pushover analysis results show that the building can meet a minimum safety performance criterion – preventing collapse – in a very large earthquake.

However, there is some uncertainty regarding the likely performance of this building and its beam-column joints, which do not have proper reinforcing (there are no lateral ties in the joint). A retrofit of the joints would reduce damage in a major earthquake. However, joint retrofits are costly and invasive, and can be impractical for residential buildings such as this one. Innovative and lower cost retrofit schemes, such as the rocking spine concept used on other case study buildings considered as part of this project, need to be developed and tested in order to make retrofit of residential and mixed-use buildings in Pakistan more economical and practical.

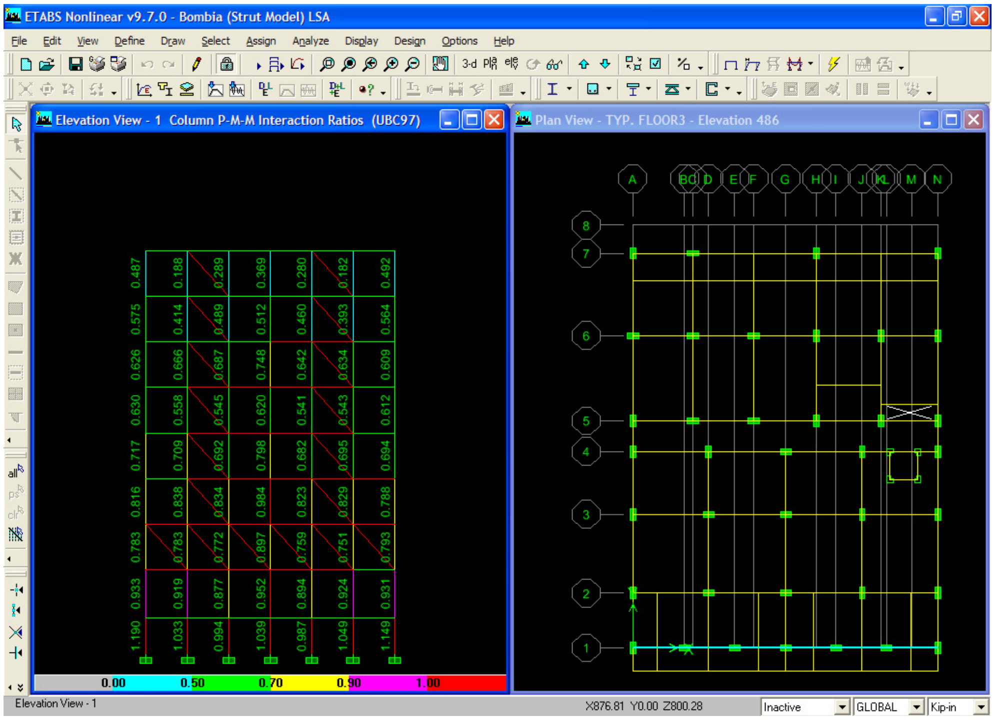
Appendix A: Tier 1 Checklists

SUMMARY OF CHECKLIST FOR BUILDING SYSTEM	
1. Load Path	C
2. Adjacent Building	NC
3. Mezzanine	C
4. Weak Story	NC
5. Soft Story	NC
6. Geometry	C
7. Vertical Discontinuities	NC
8. Mass Irregular	NC
9. Torsional Irregularity	NC
10. Deterioration	NC
11. Post Tensioning Anchors	N/A

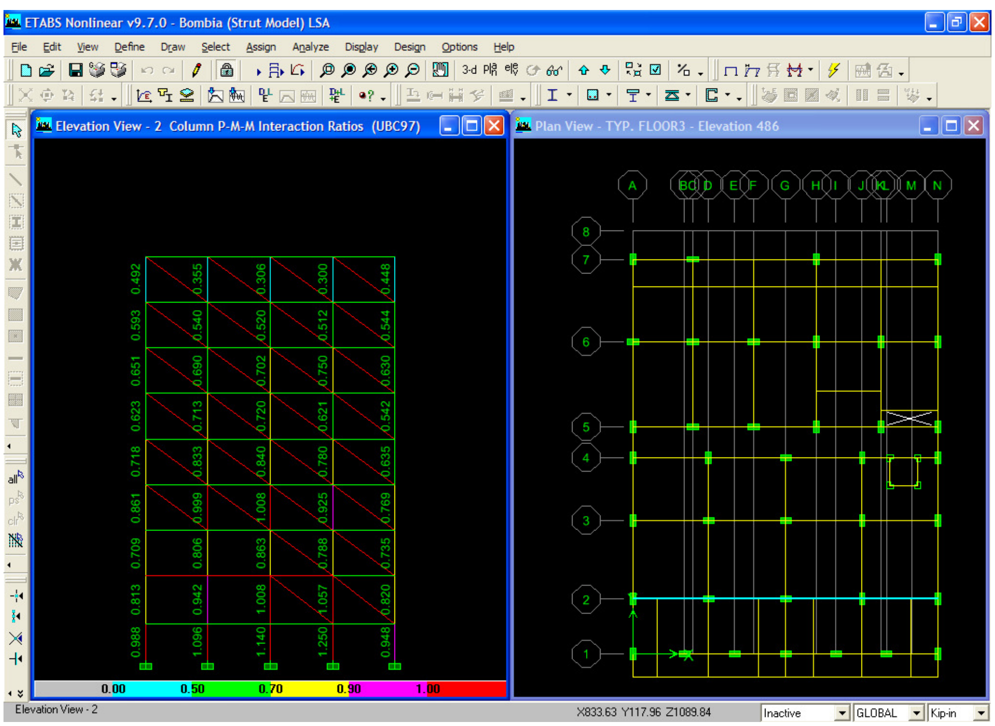
SUMMARY OF CHECKLIST FOR LATERAL – FORCE-RESISTING SYSTEM	
1. Redundancy	C
2. Interfering Wall	NC
3. Shear Stress Check	NC
4. Axial Stress Check	NC

Geological Site Hazards		
1.	Liquefaction	C
2.	Slope Failure	C
3.	Surface Fault rupture	C
Condition of Foundation		
1.	Foundation Performance	C
2.	Deterioration	C
Capacity of Foundation		
1.	Pole Foundation	N/A
2.	Overturning	C
3.	Ties between Foundation element	C
4.	Deep foundation	N/A
5.	Sloping Sites	C

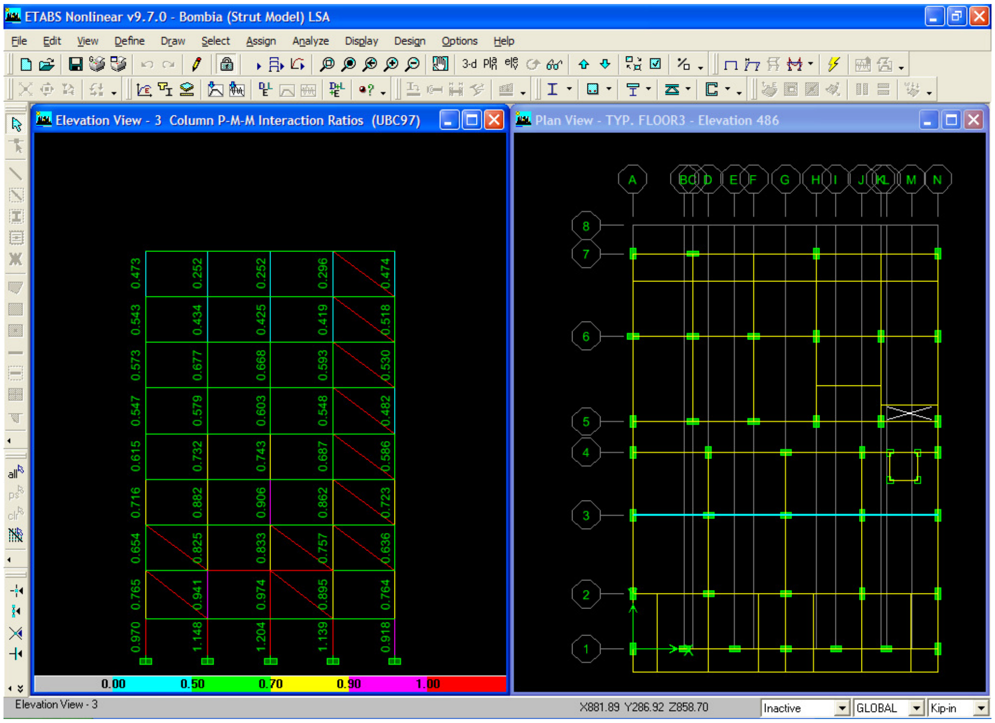
Appendix B: Linear Analysis (Tier 2) Results



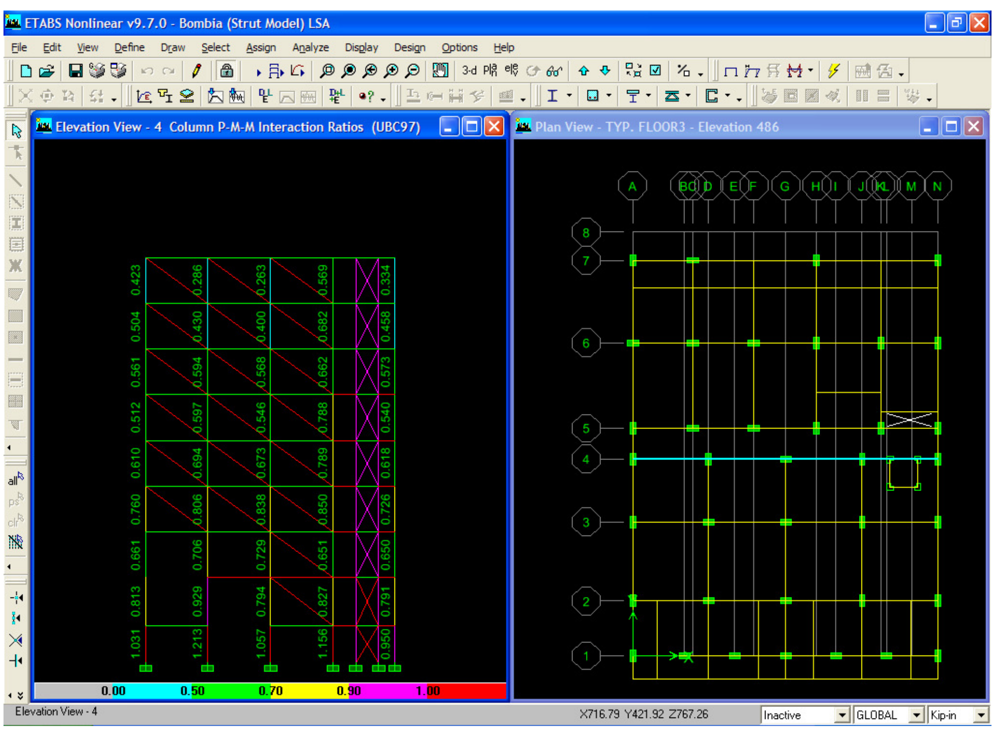
Demand/Capacity Ratios for Frame at Grid-1



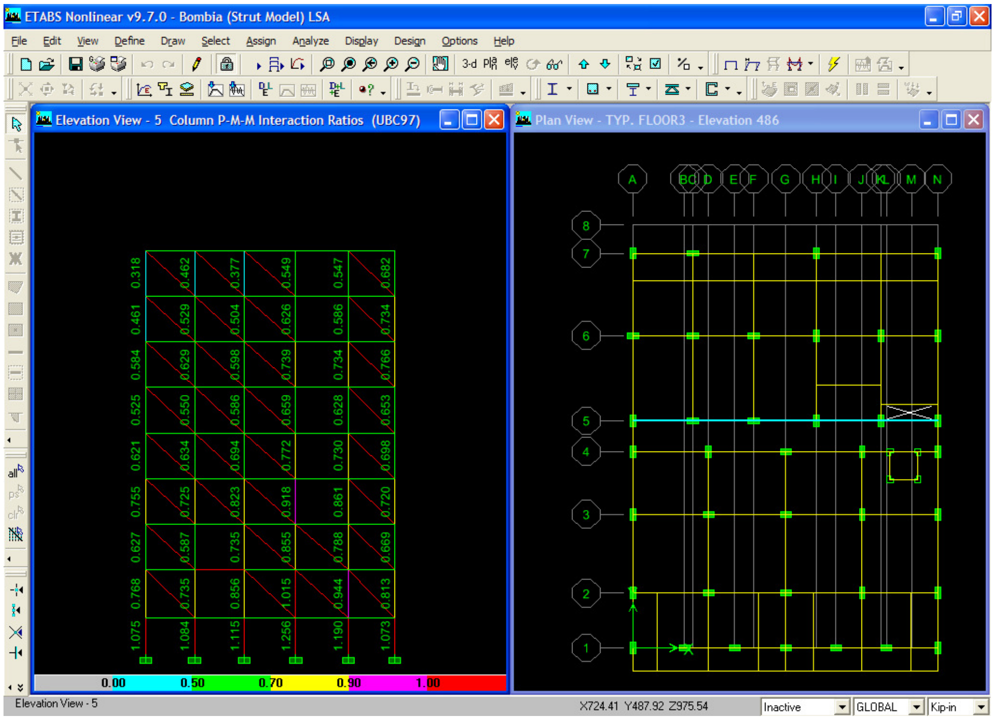
Demand/Capacity Ratios for Frame at Grid-2



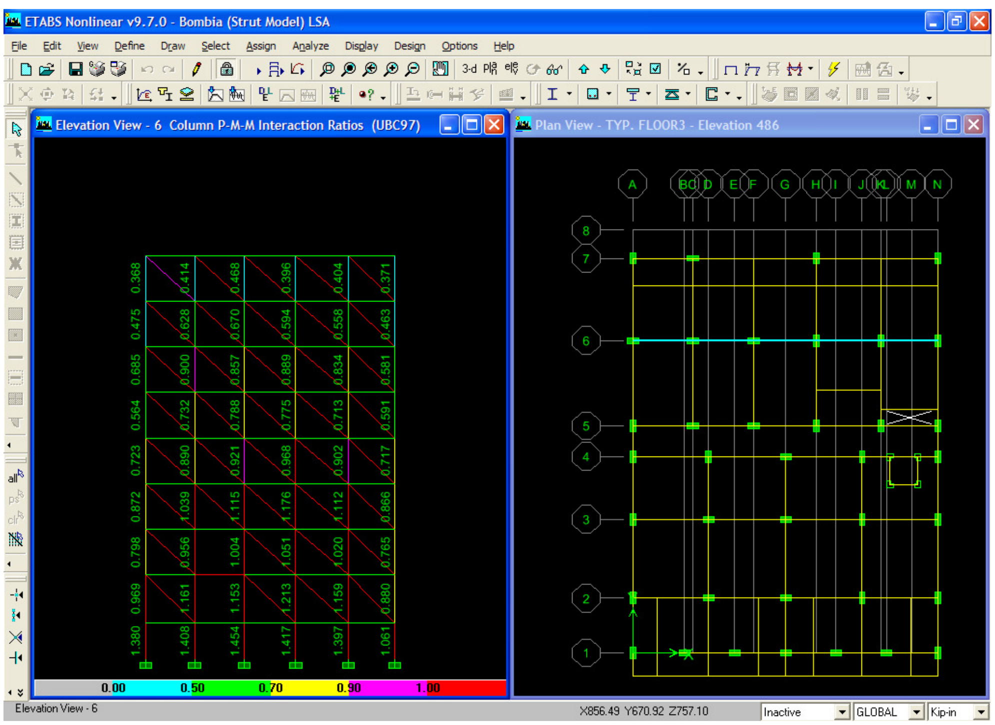
Demand/Capacity Ratios for Frame at Grid-3



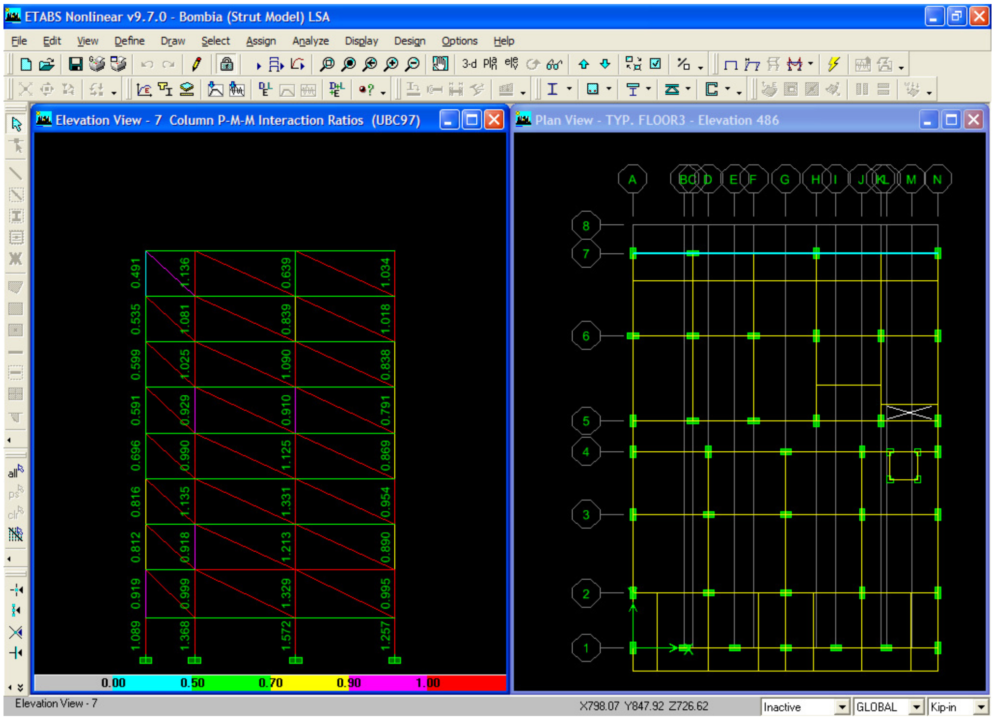
Demand/Capacity Ratios for Frame at Grid-4



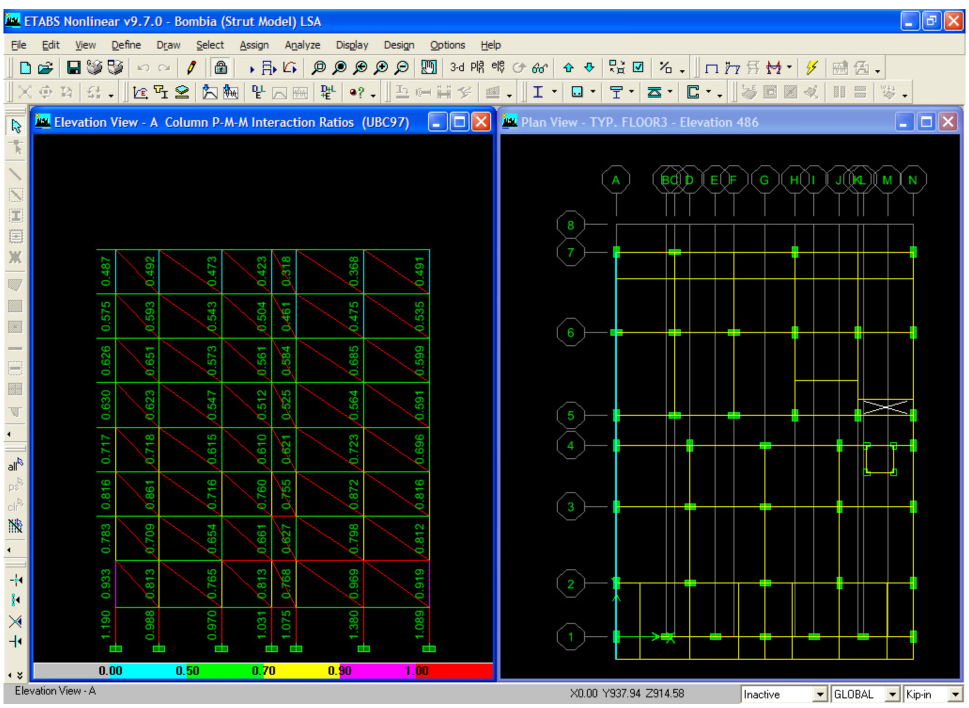
Demand/Capacity Ratios for Frame at Grid-5



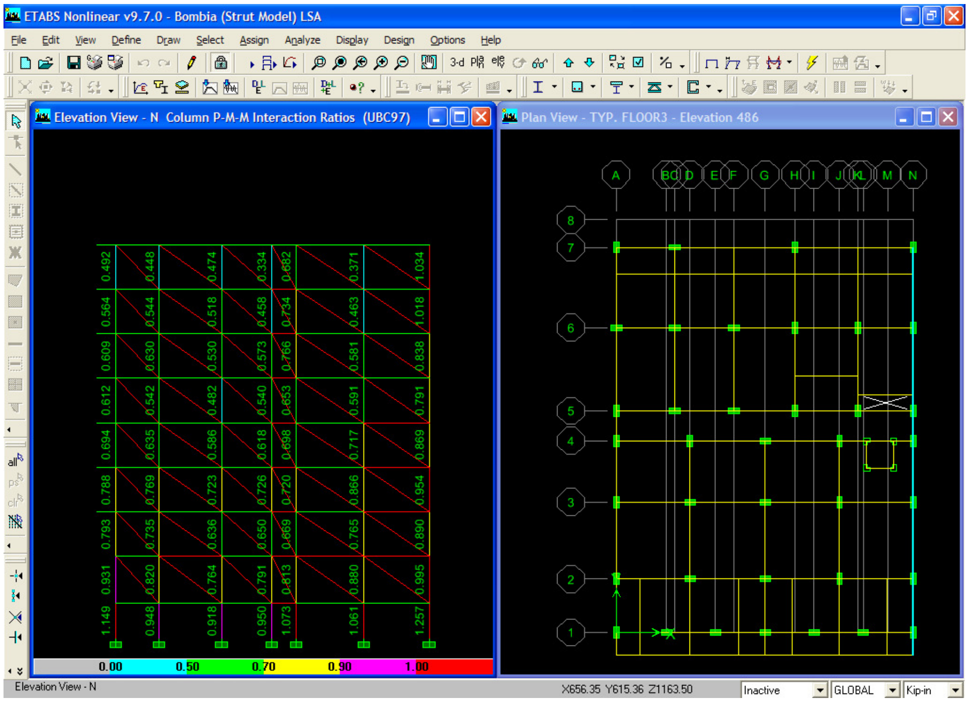
Demand/Capacity Ratios for Frame at Grid-6



Demand/Capacity Ratios for Frame at Grid-7



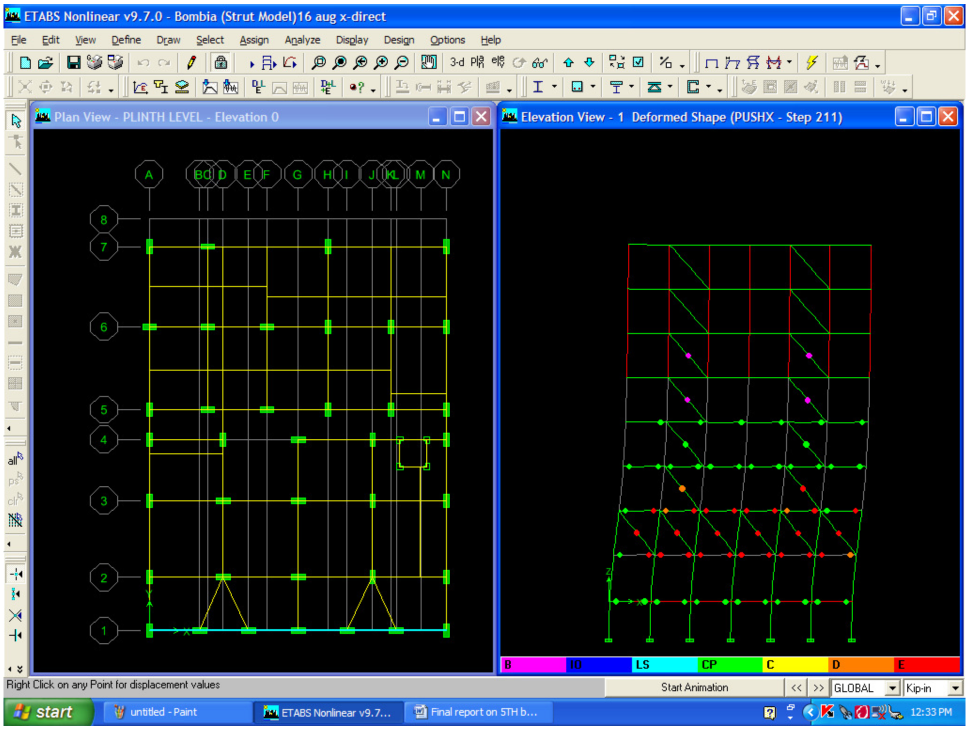
Demand/Capacity Ratios for Frame at Grid-A

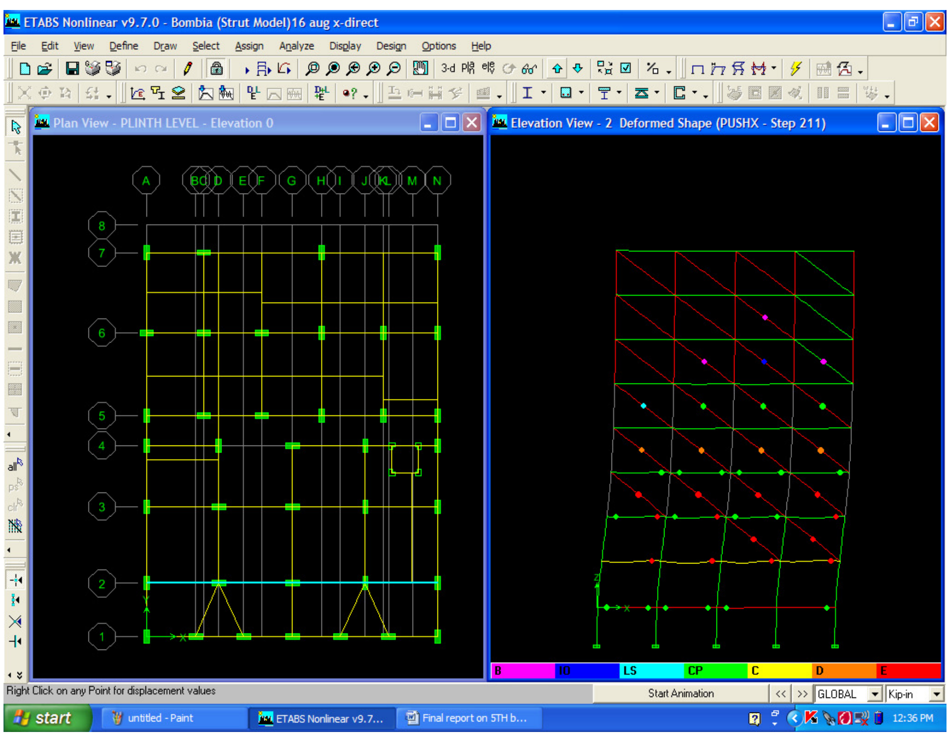


Demand/Capacity Ratios for Frame at Grid-N

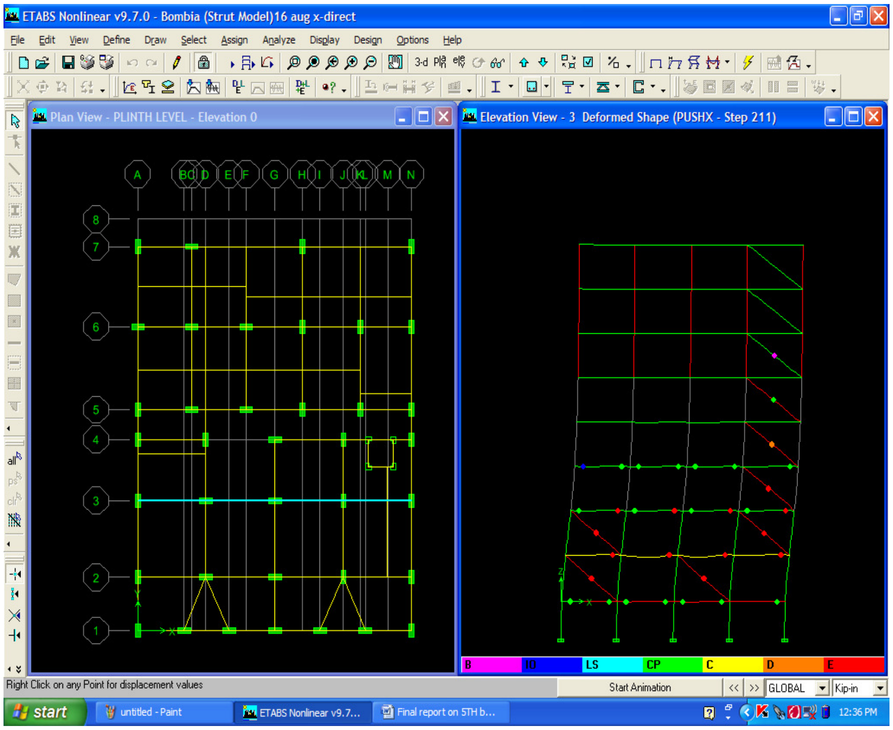
Appendix C: Non Linear Analysis (Tier 3) Results

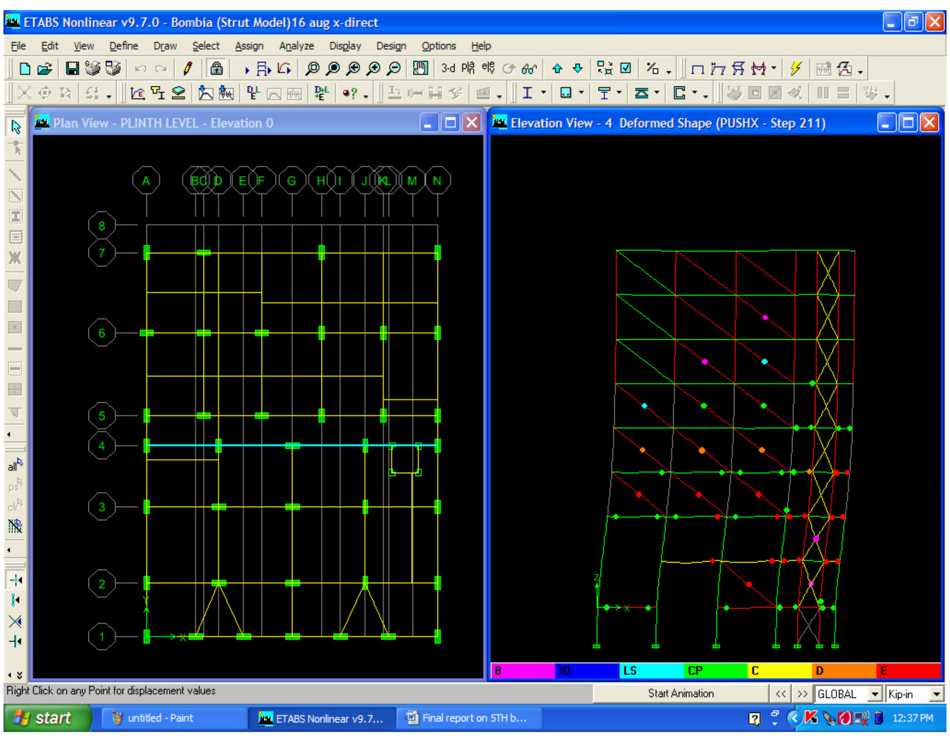
Results for non linear analysis in X-direction



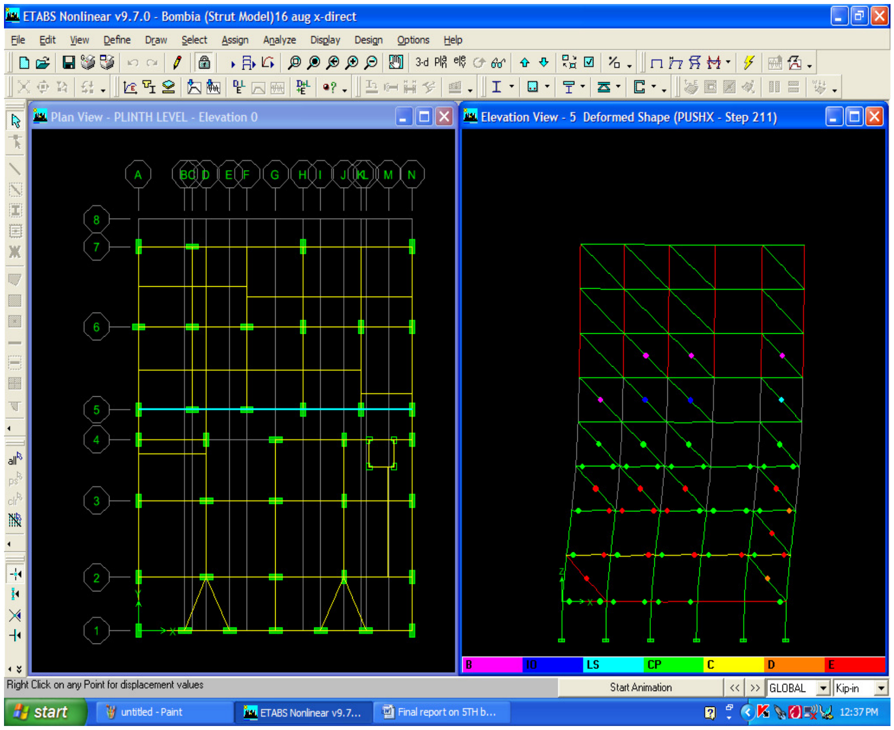


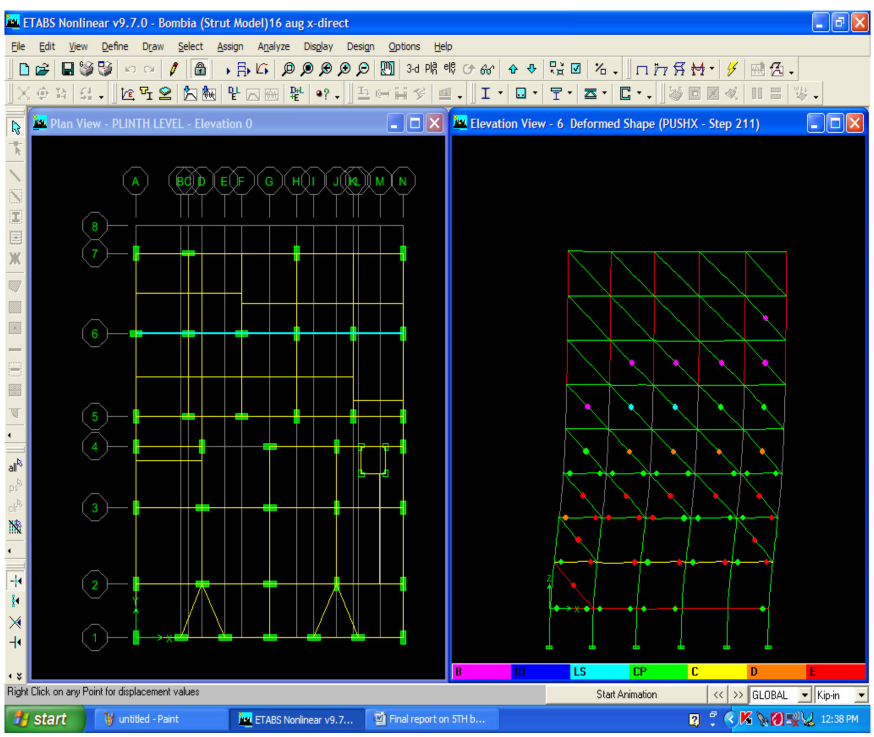
Mechanism or Deformed shapes at Performance Point - grid-1 (Top) grid-2 (bottom)



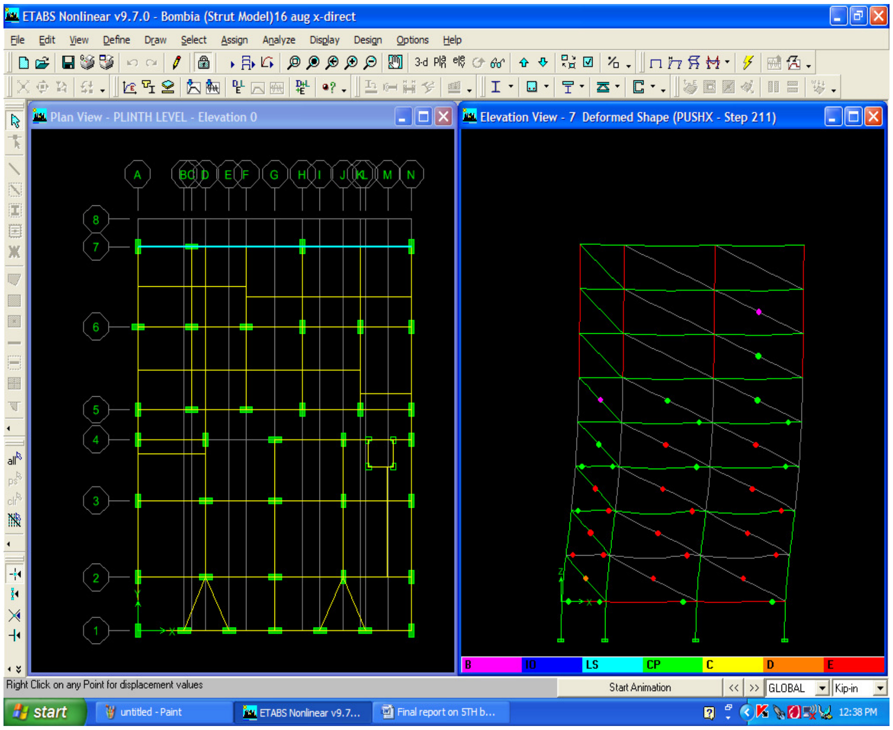


Mechanism or Deformed shapes at Performance Point - grid-3 (Top) grid-4 (bottom)



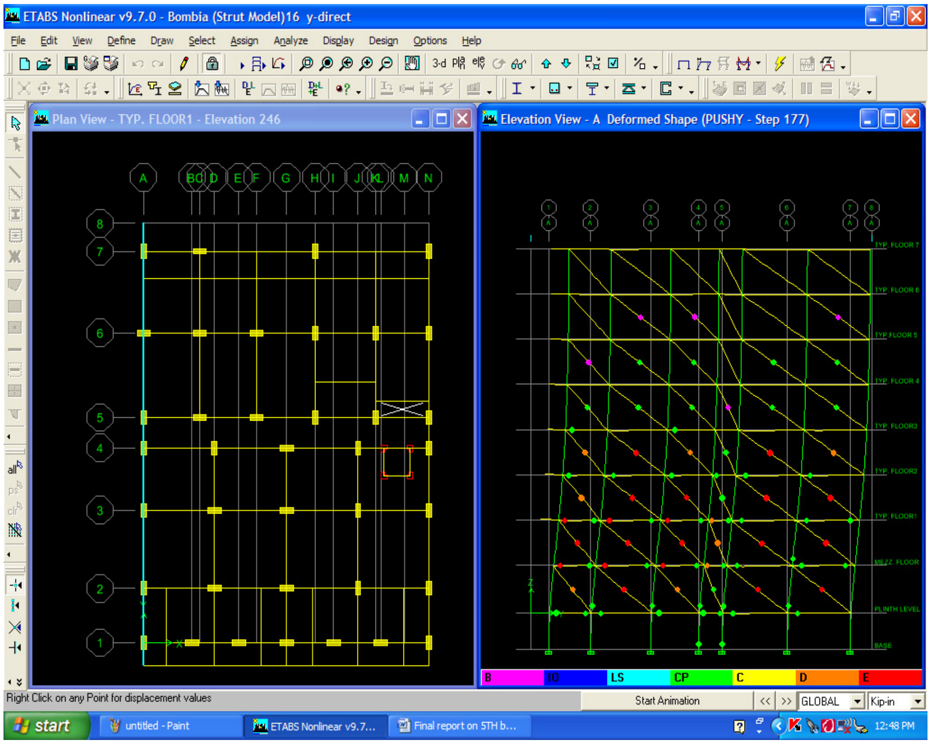


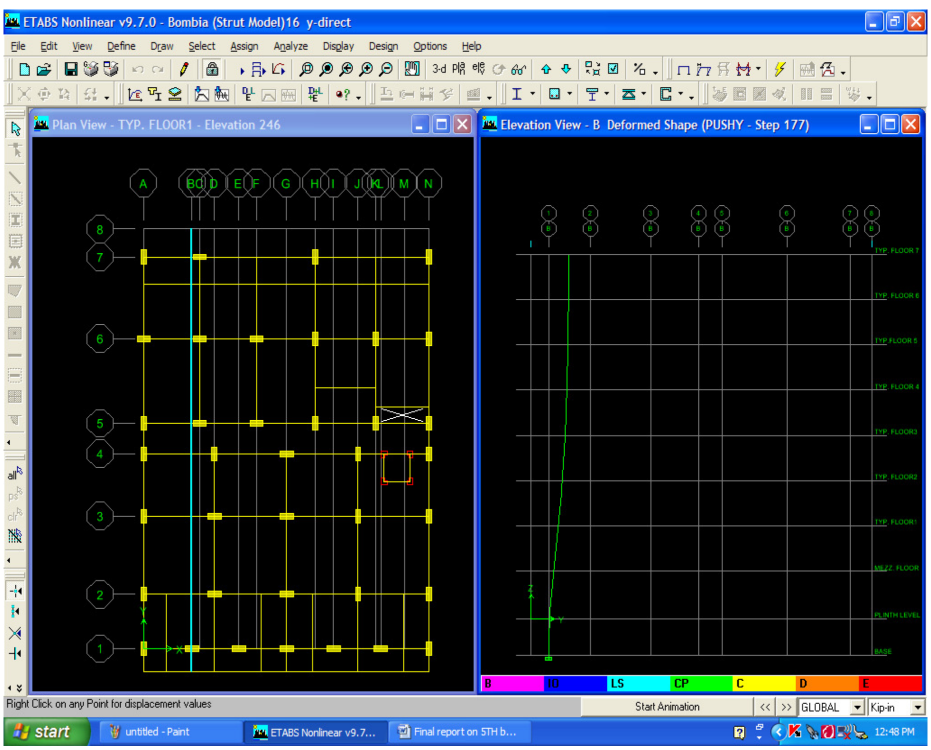
Mechanism or Deformed shapes at Performance Point - grid-5 (Top) grid-6 (bottom)



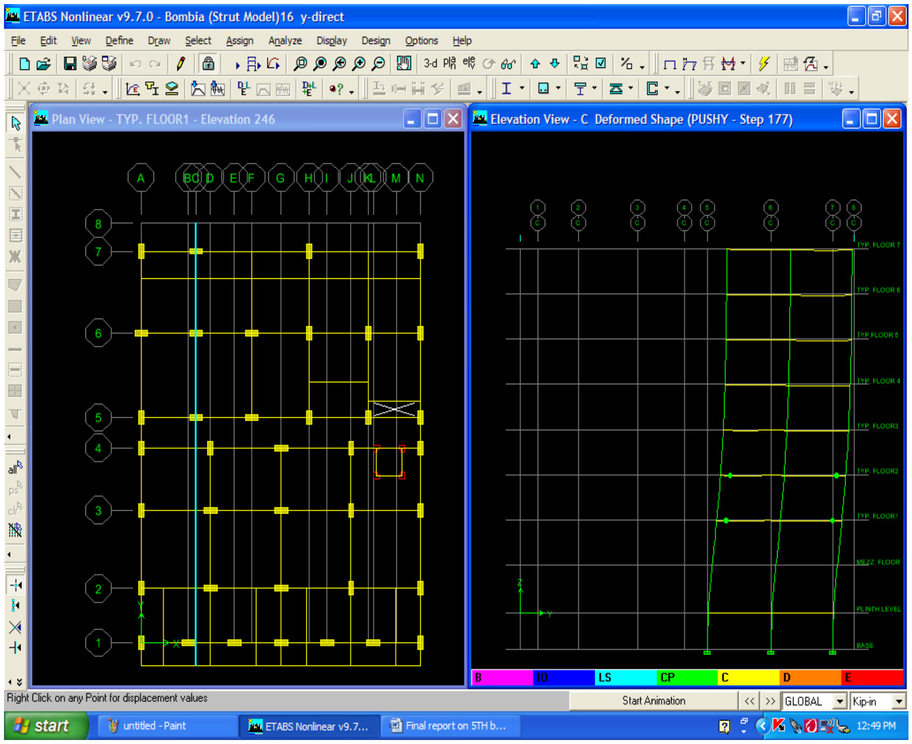
Mechanism or Deformed shapes at Performance Point - grid-7

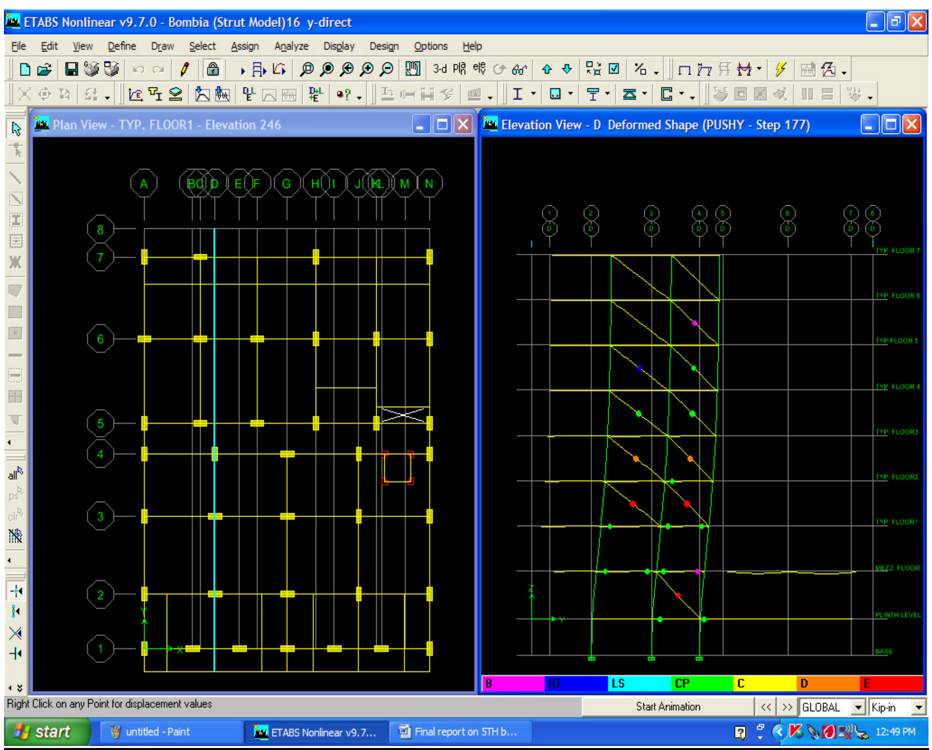
Results for non linear analysis in Y-direction



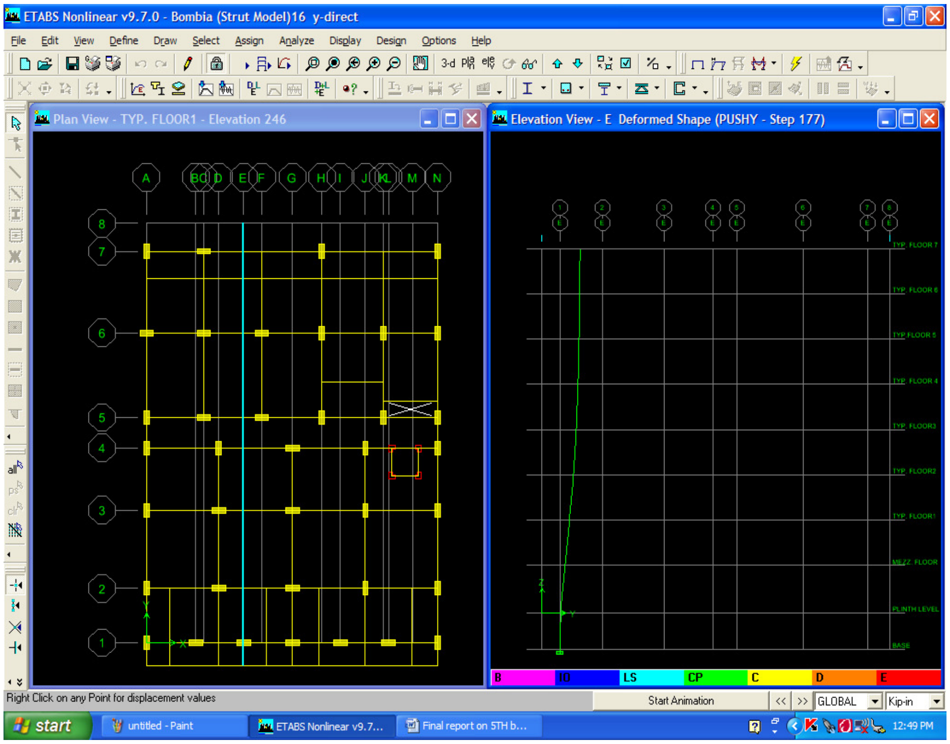


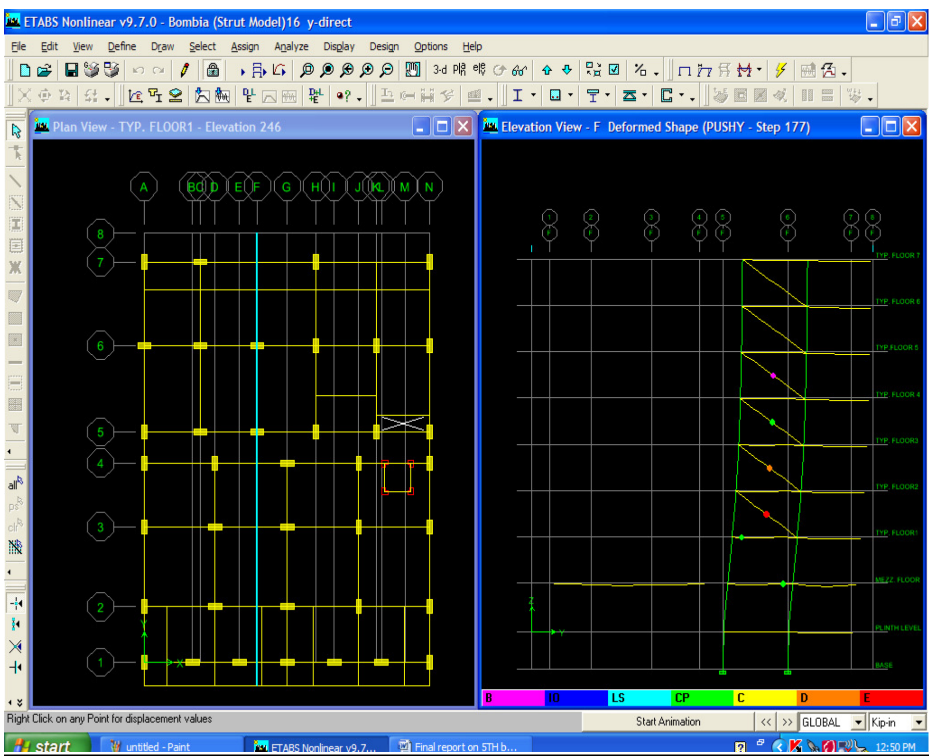
Mechanism or Deformed shapes at Performance Point - grid-A (Top) grid-B (bottom)



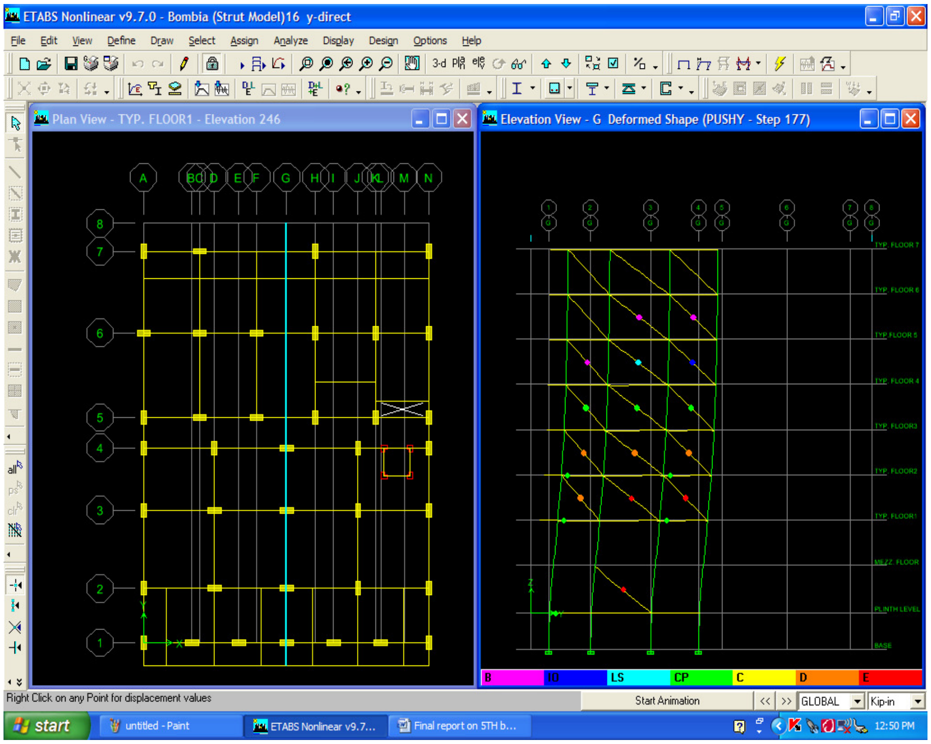


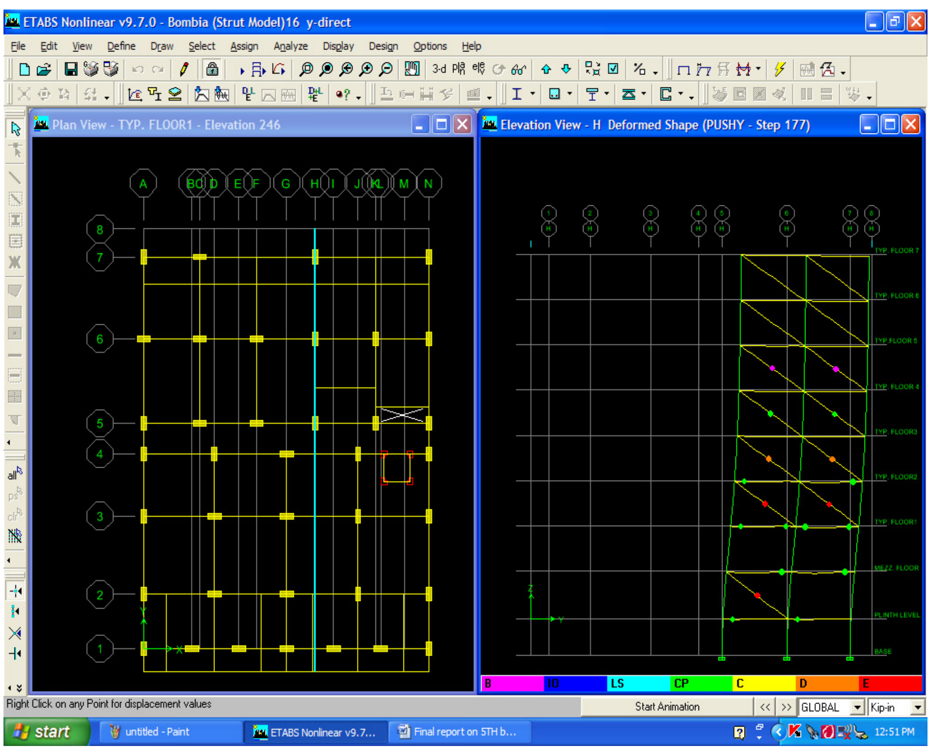
Mechanism or Deformed shapes at Performance Point - grid-C (Top) grid-D (bottom)



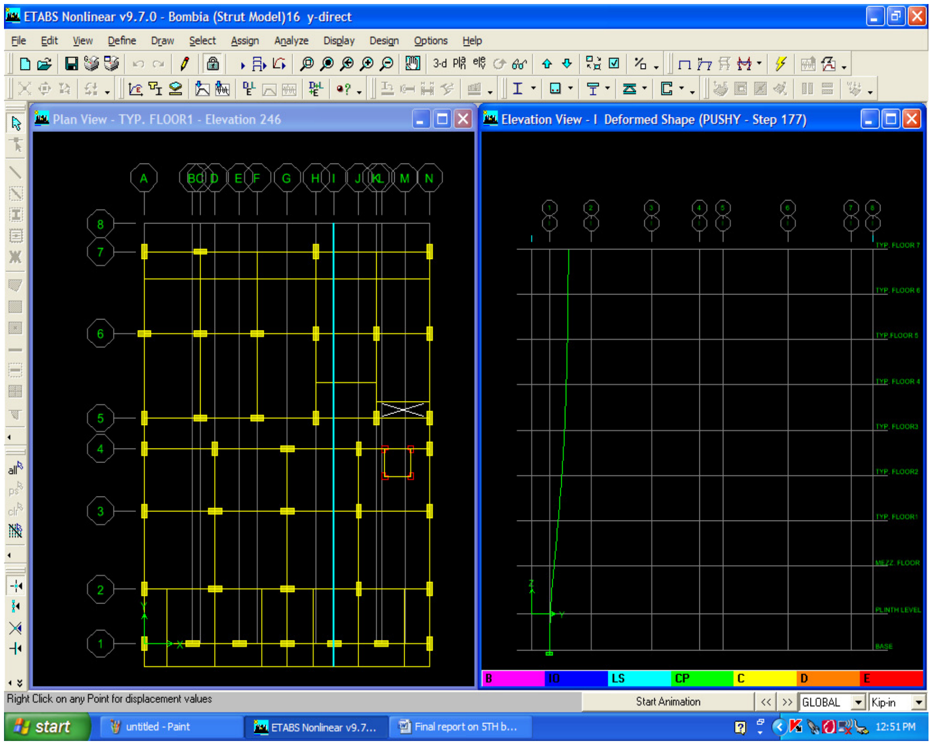


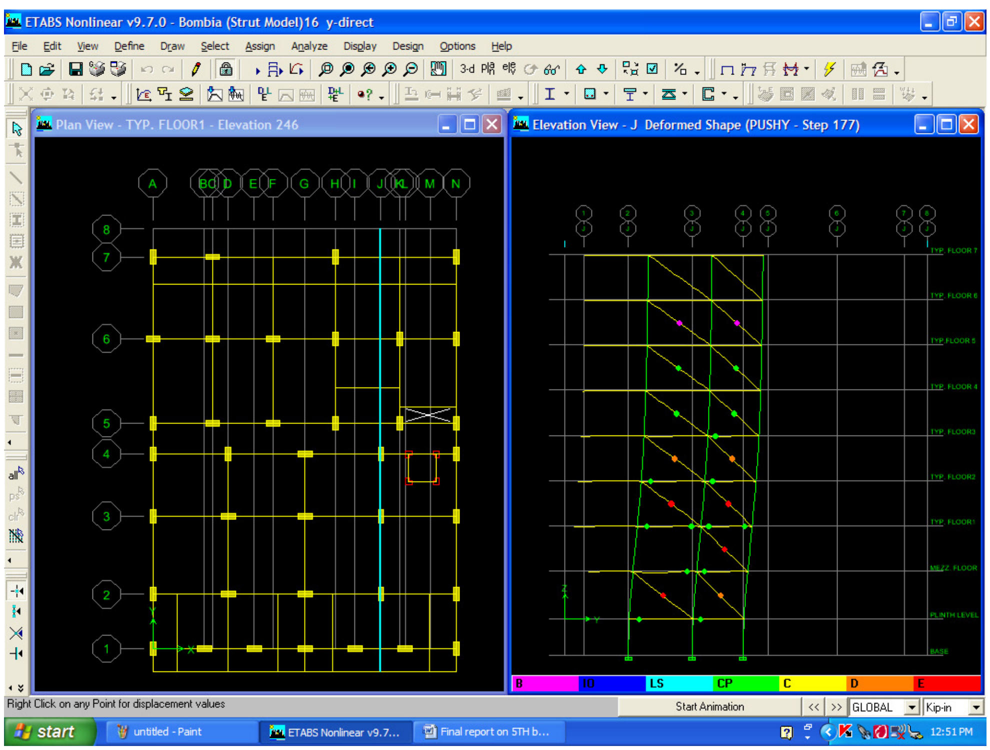
Mechanism or Deformed shapes at Performance Point - grid-E (Top) grid-F (bottom)



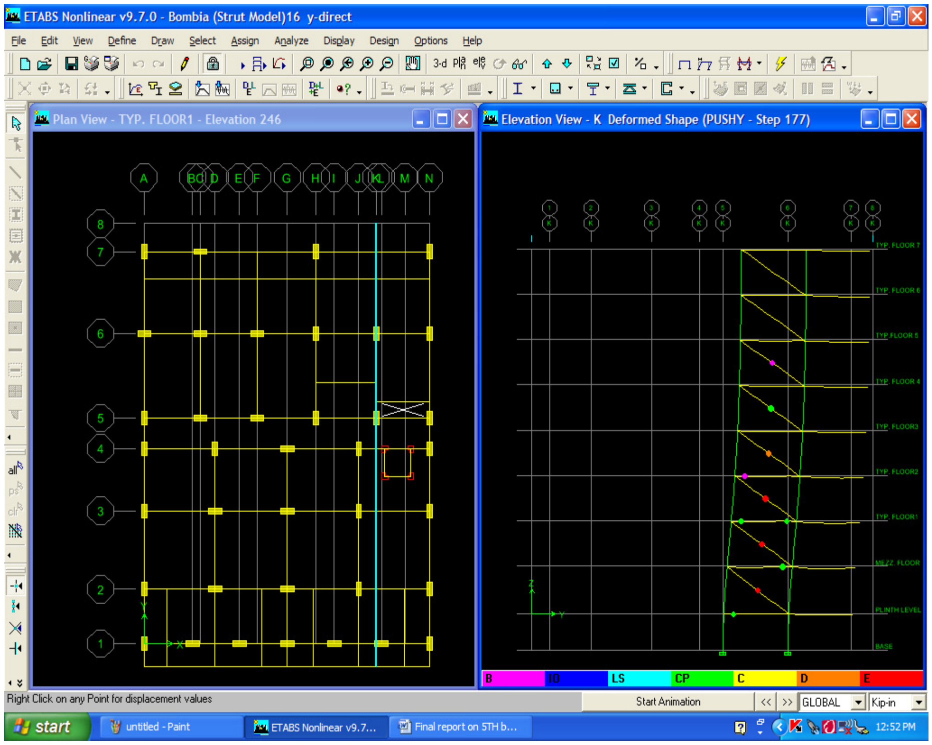


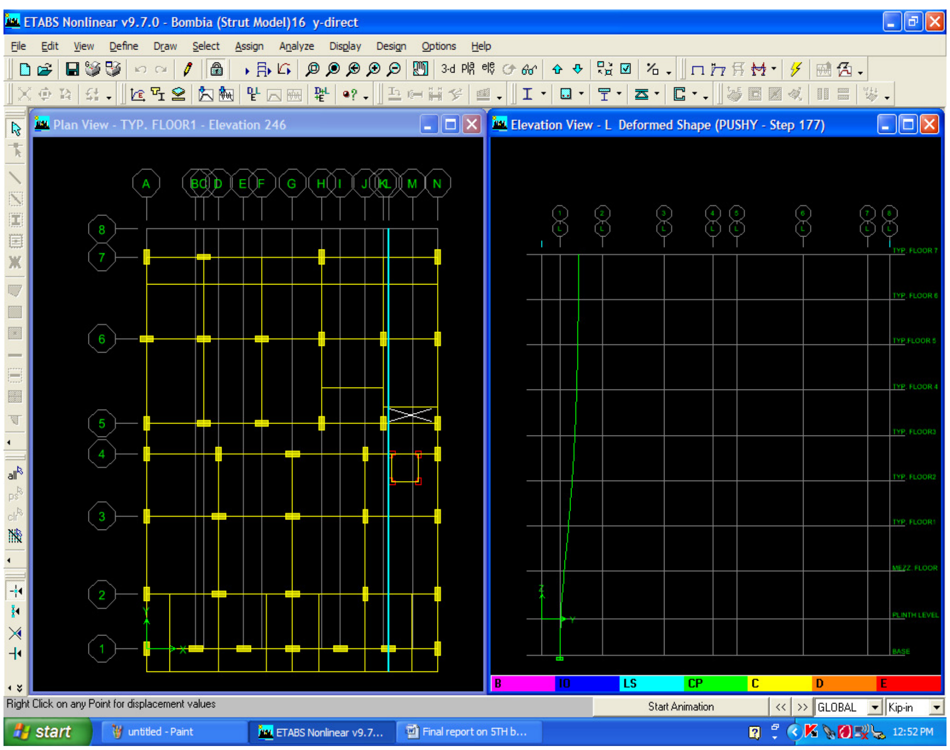
Mechanism or Deformed shapes at Performance Point - grid-G (Top) grid-H (bottom)



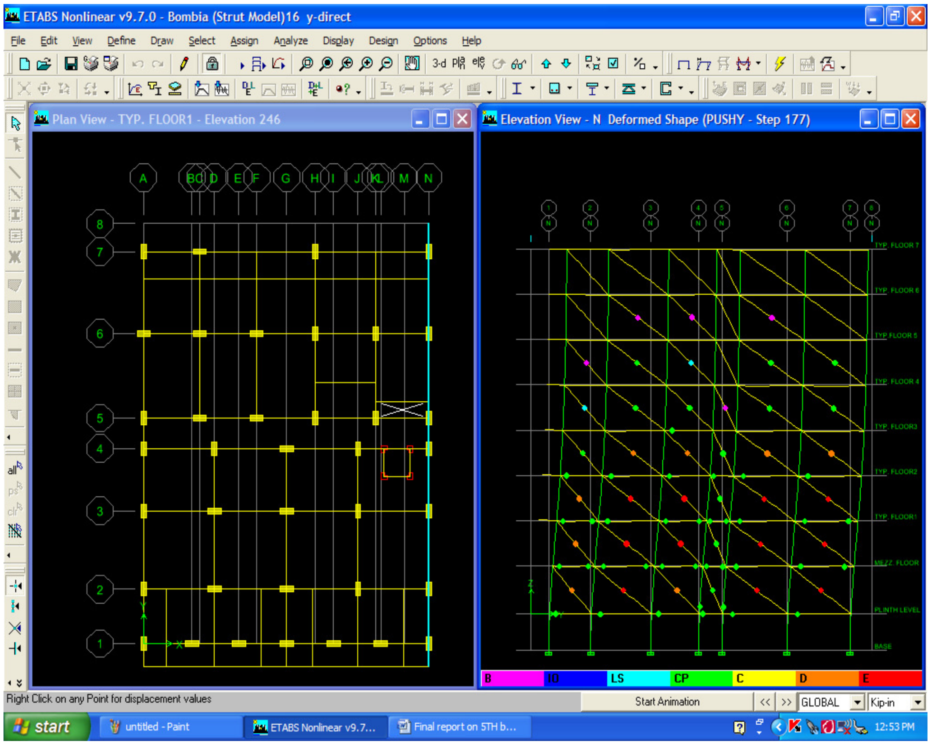


Mechanism or Deformed shapes at Performance Point - grid-I (Top) grid-J (bottom)

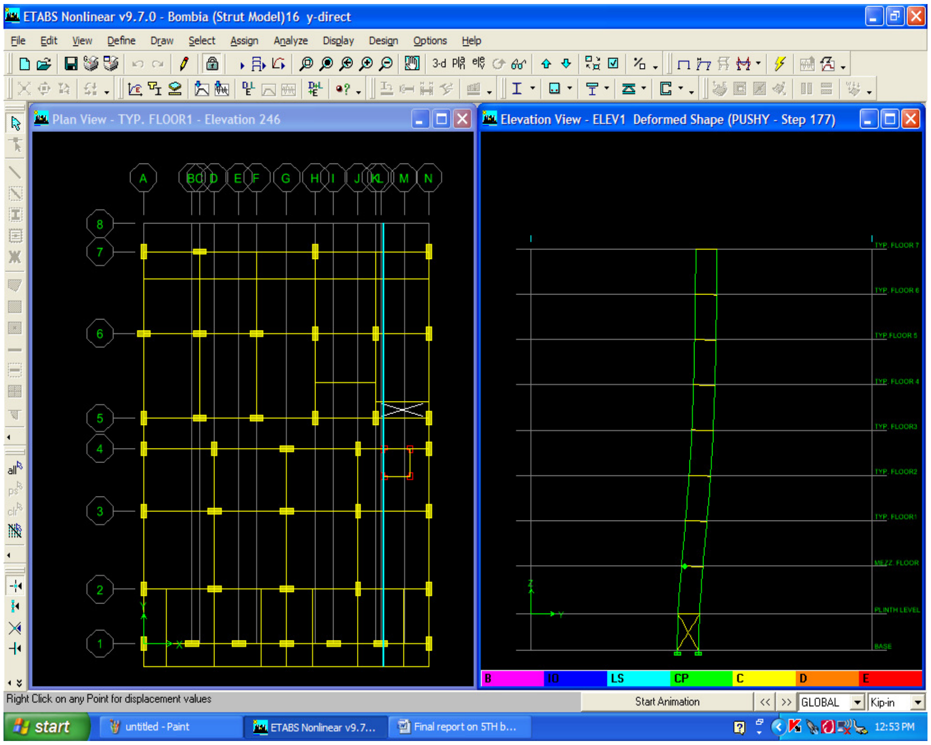


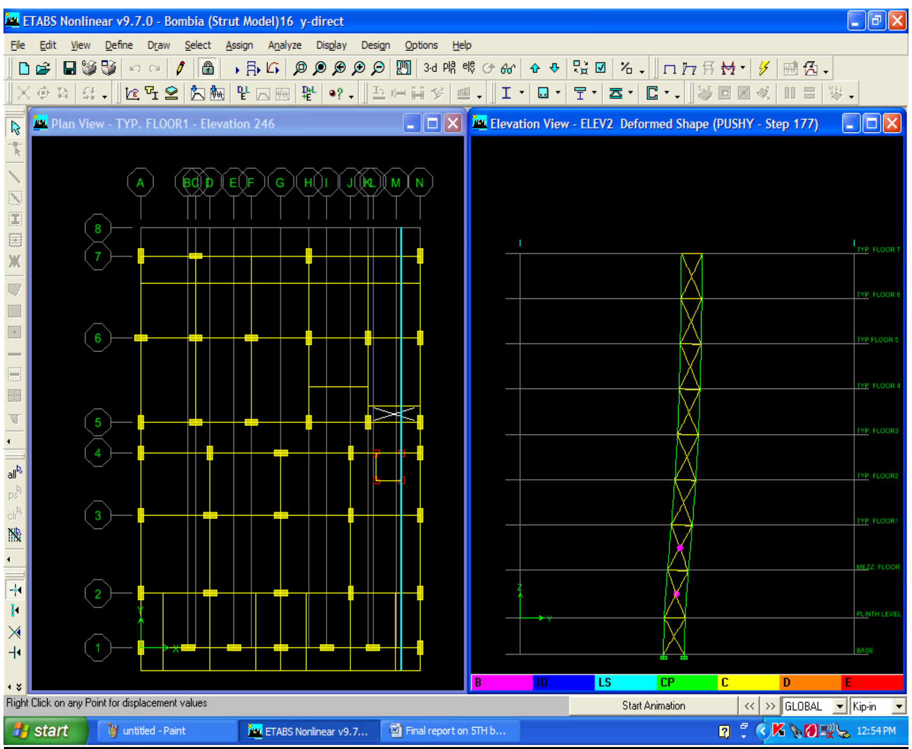


Mechanism or Deformed shapes at Performance Point - grid-K (Top) grid-L (bottom)



Mechanism or Deformed shapes at Performance Point - grid-N





Mechanism or Deformed shapes at Performance Point – LHS of Lift (Top) RHS of Lift (bottom)

Appendix D: Optional Conceptual Retrofit and Non Linear Analysis (Tier 3) Results after Retrofit

Conceptual Retrofit Option

The optional retrofit solution to improve seismic performance of the building and increase confidence against collapse would consist of adding RCC walls as shown in Figure 15. Figure 16 shows suggested details.

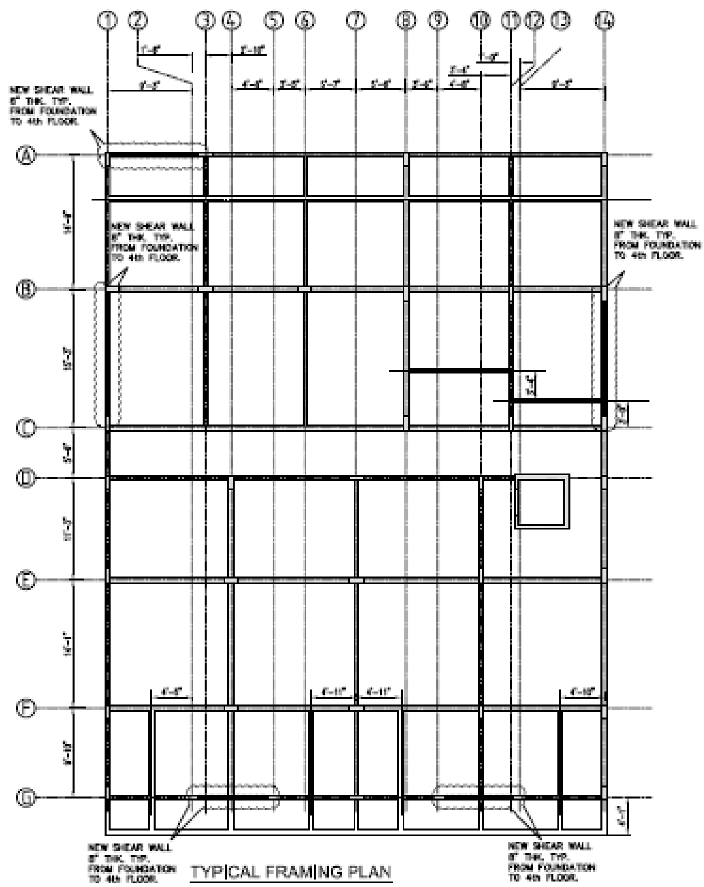


Figure 15. Proposed location of RCC infill walls

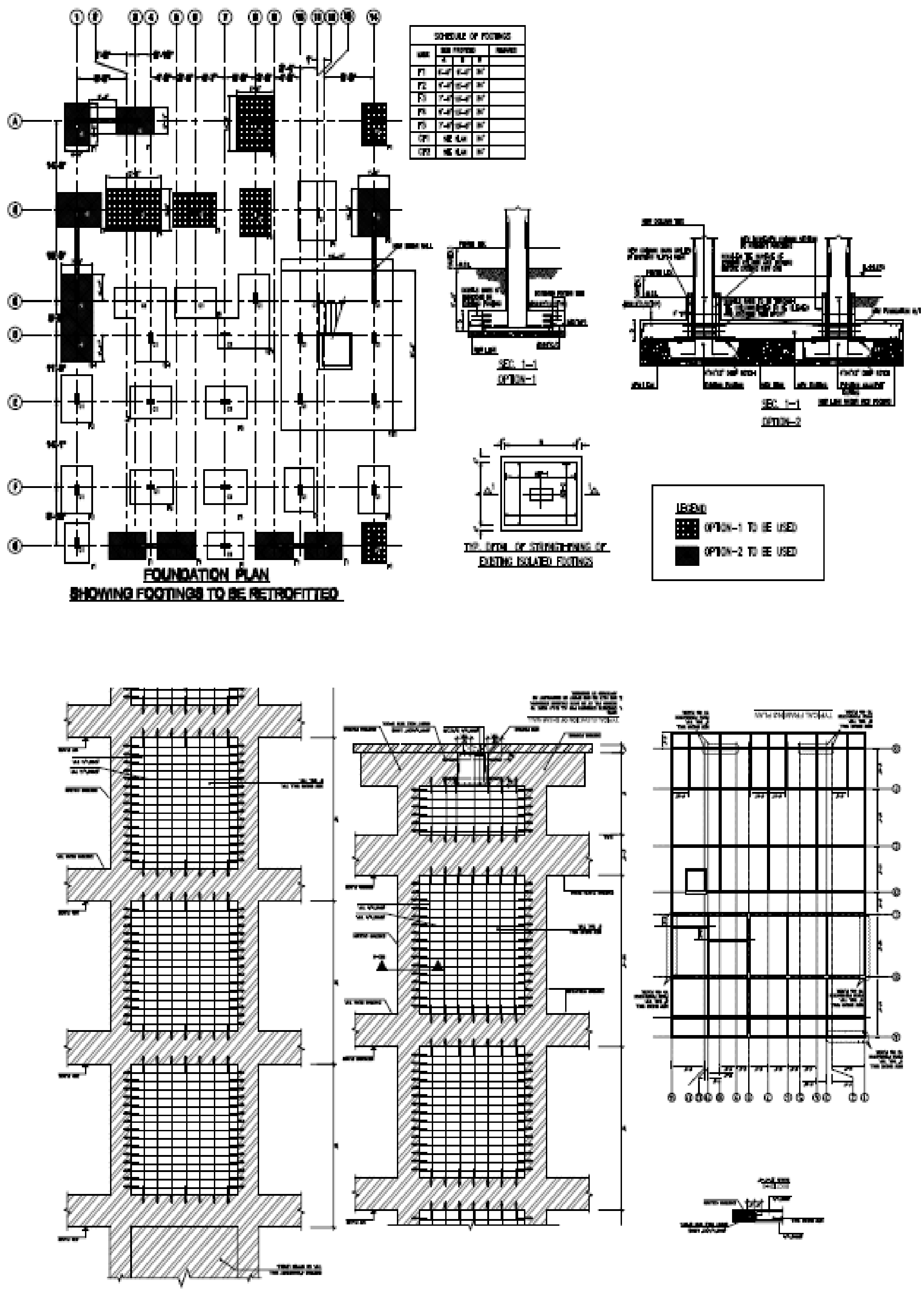
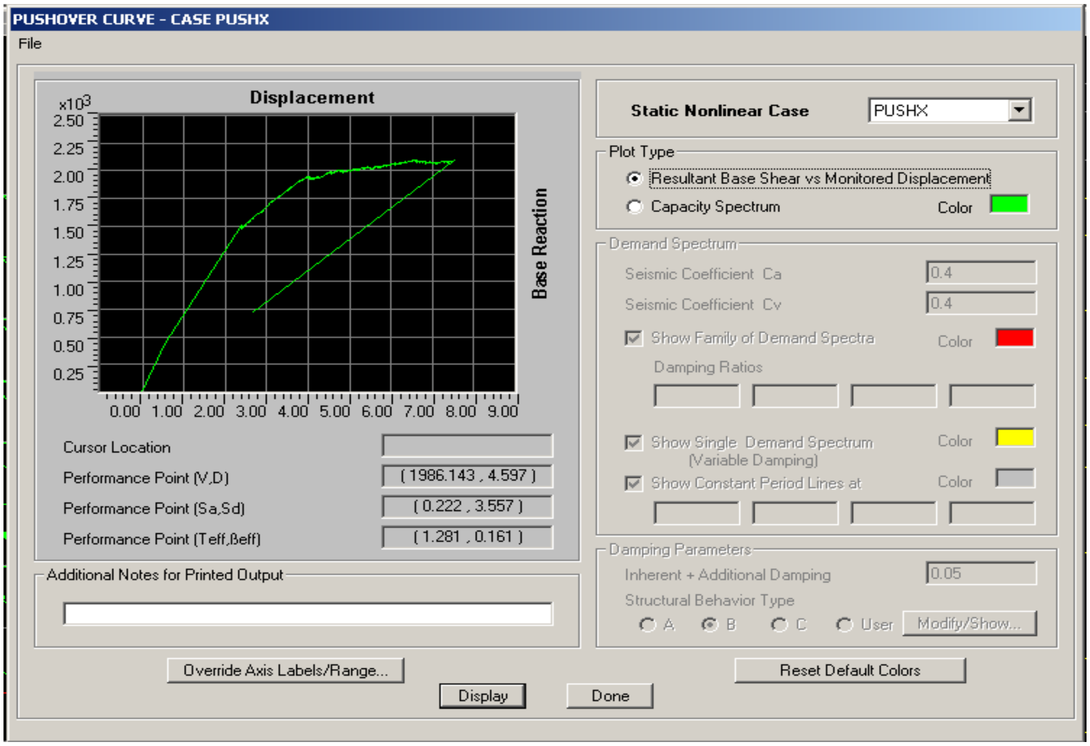
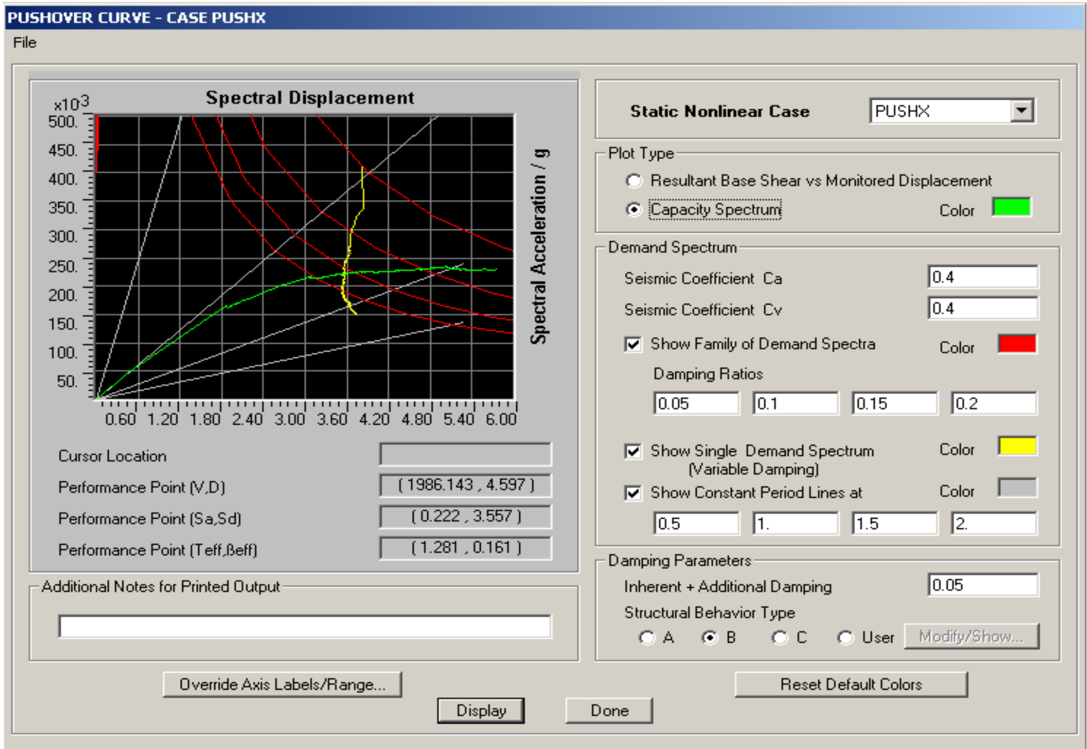


Figure 16. Proposed retrofitting details – foundations (top) RCC infill walls (bottom)

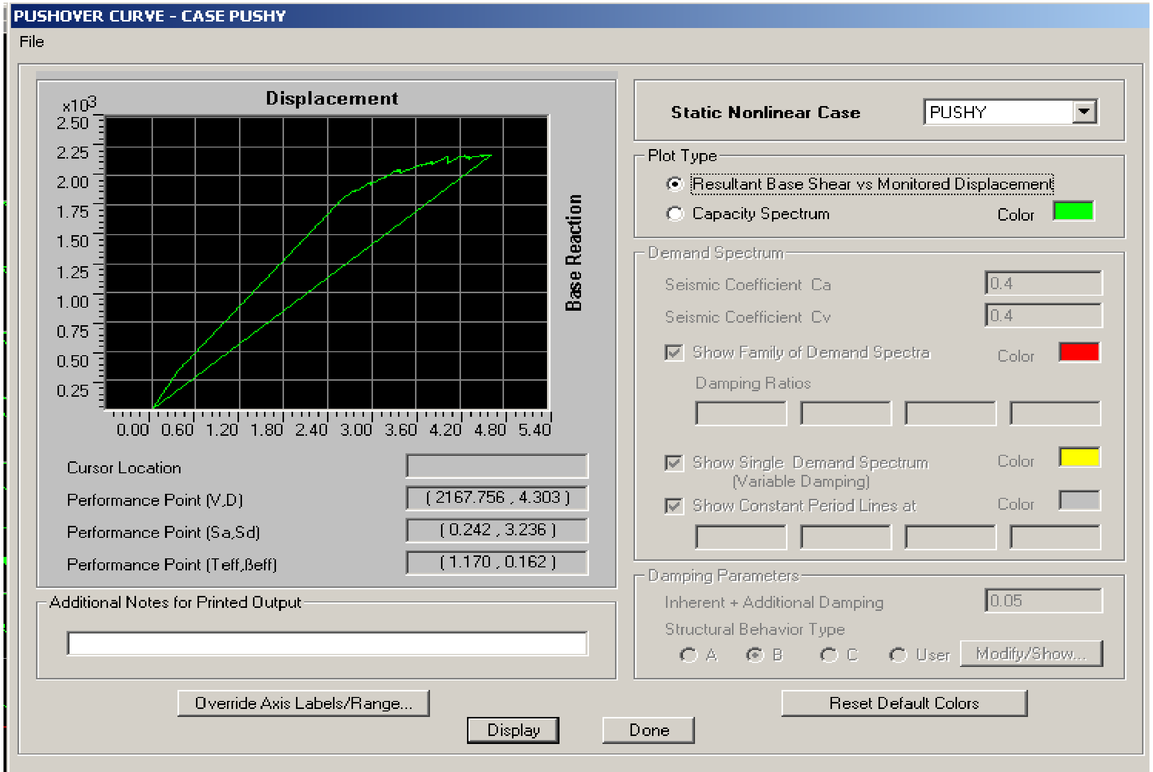
Analytical Model and Results

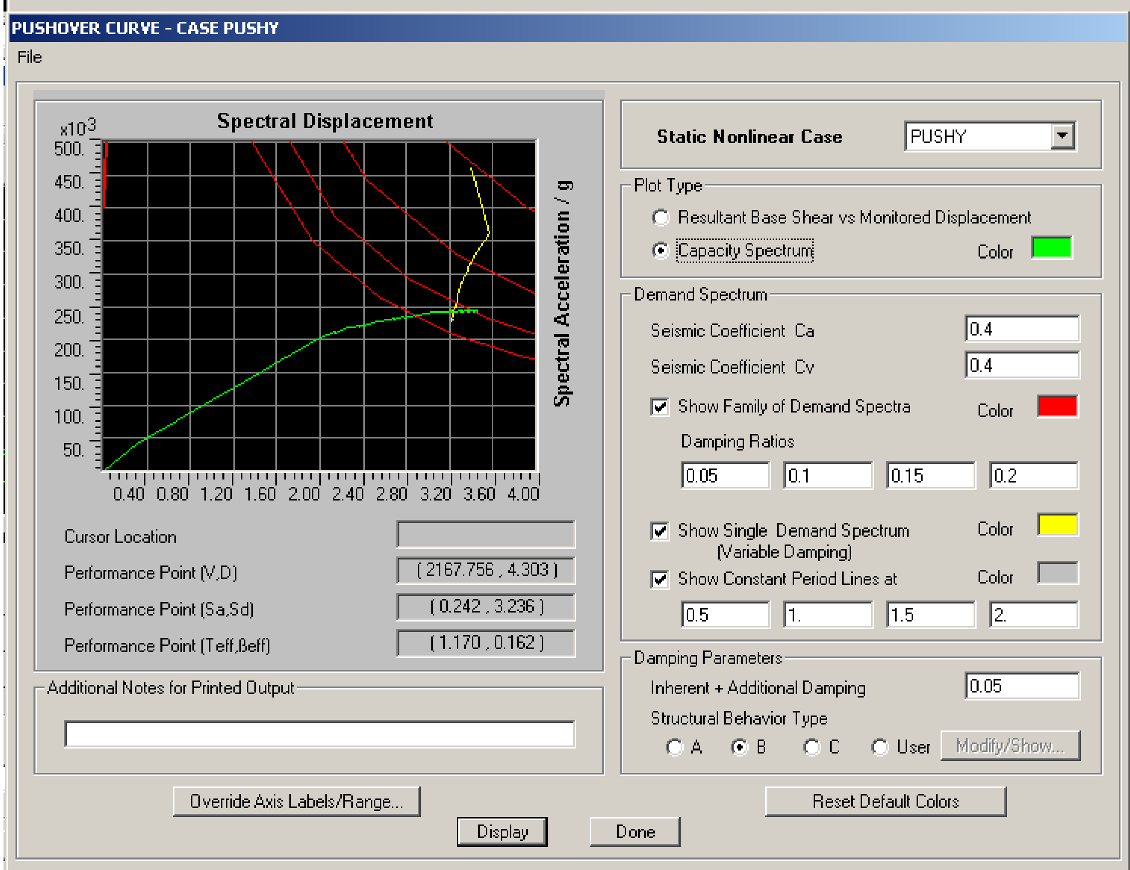
The same 3-D model is used with strengthened infill walls modeled with linear compression struts and tension ties. Results for the retrofitted building are showing in the following plots.





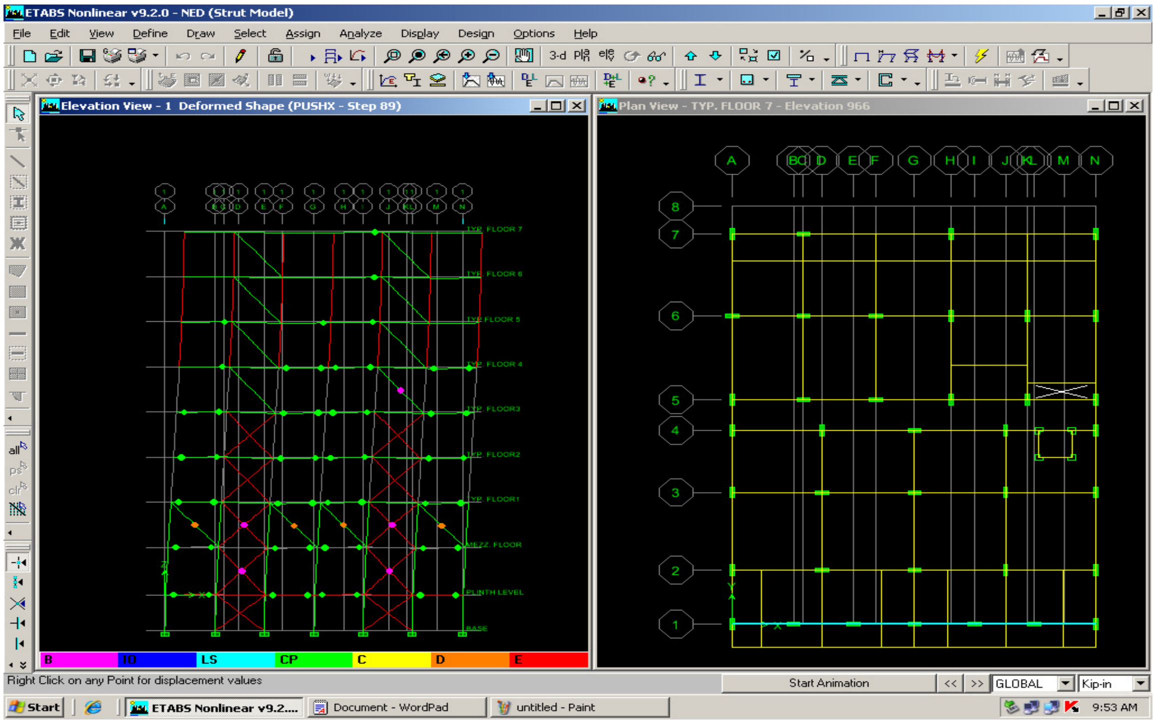
Pushover curve (top) and performance level (bottom) for seismic forces in X-direction

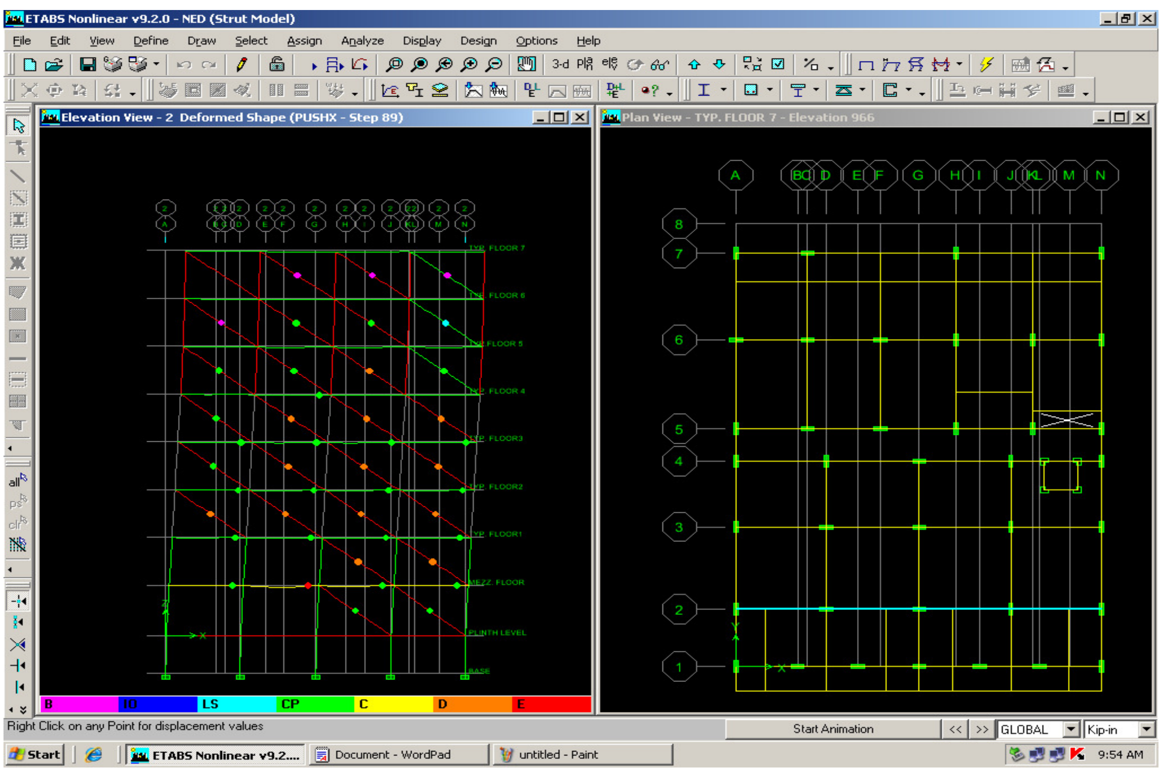

PUSHOVER CURVE - CASE PUSHY
File
Displacement
Base Reaction
x10^3
Static Nonlinear Case
PUSHY
Plot Type
Resultant Base Shear vs Monitored Displacement
Capacity Spectrum
Color
Demand Spectrum
Seismic Coefficient Ca
Seismic Coefficient Cv
Show Family of Demand Spectra
Damping Ratios
Show Single Demand Spectrum (Variable Damping)
Show Constant Period Lines at
Damping Parameters
Inherent + Additional Damping
Structural Behavior Type
Cursor Location
Performance Point (V,D) (2167.756 , 4.303)
Performance Point (Sa,Sd) (0.242 , 3.236)
Performance Point (Teff,Beff) (1.170 , 0.162)
Additional Notes for Printed Output
Override Axis Labels/Range...
Display
Done
Reset Default Colors
Modify/Show...


PUSHOVER CURVE - CASE PUSHY
File
Spectral Displacement
Spectral Acceleration / g
x10^3
Static Nonlinear Case
PUSHY
Plot Type
Resultant Base Shear vs Monitored Displacement
Capacity Spectrum
Color
Demand Spectrum
Seismic Coefficient Ca 0.4
Seismic Coefficient Cv 0.4
Show Family of Demand Spectra
Damping Ratios 0.05 0.1 0.15 0.2
Show Single Demand Spectrum (Variable Damping)
Show Constant Period Lines at 0.5 1. 1.5 2.
Damping Parameters
Inherent + Additional Damping 0.05
Structural Behavior Type
Cursor Location
Performance Point (V,D) (2167.756 , 4.303)
Performance Point (Sa,Sd) (0.242 , 3.236)
Performance Point (Teff,Beff) (1.170 , 0.162)
Additional Notes for Printed Output
Override Axis Labels/Range...
Display
Done
Reset Default Colors
Modify/Show...

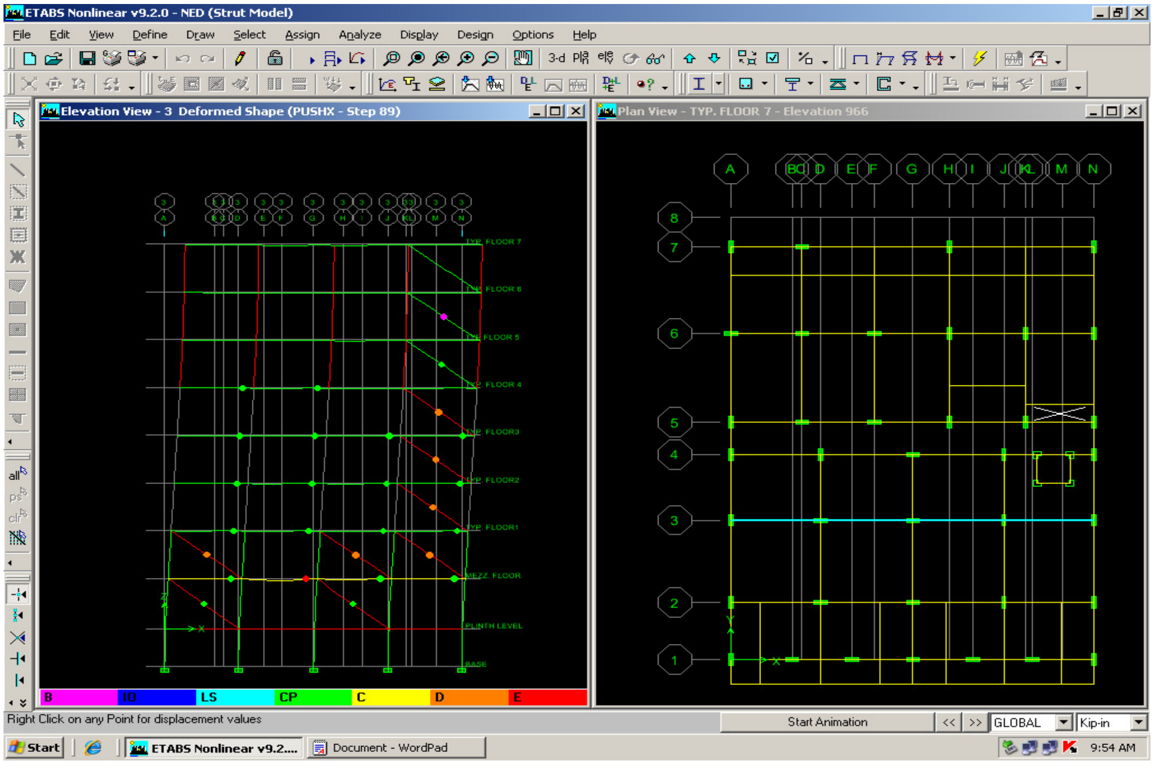
Pushover curve (top) and performance level (bottom) for seismic forces in Y-direction

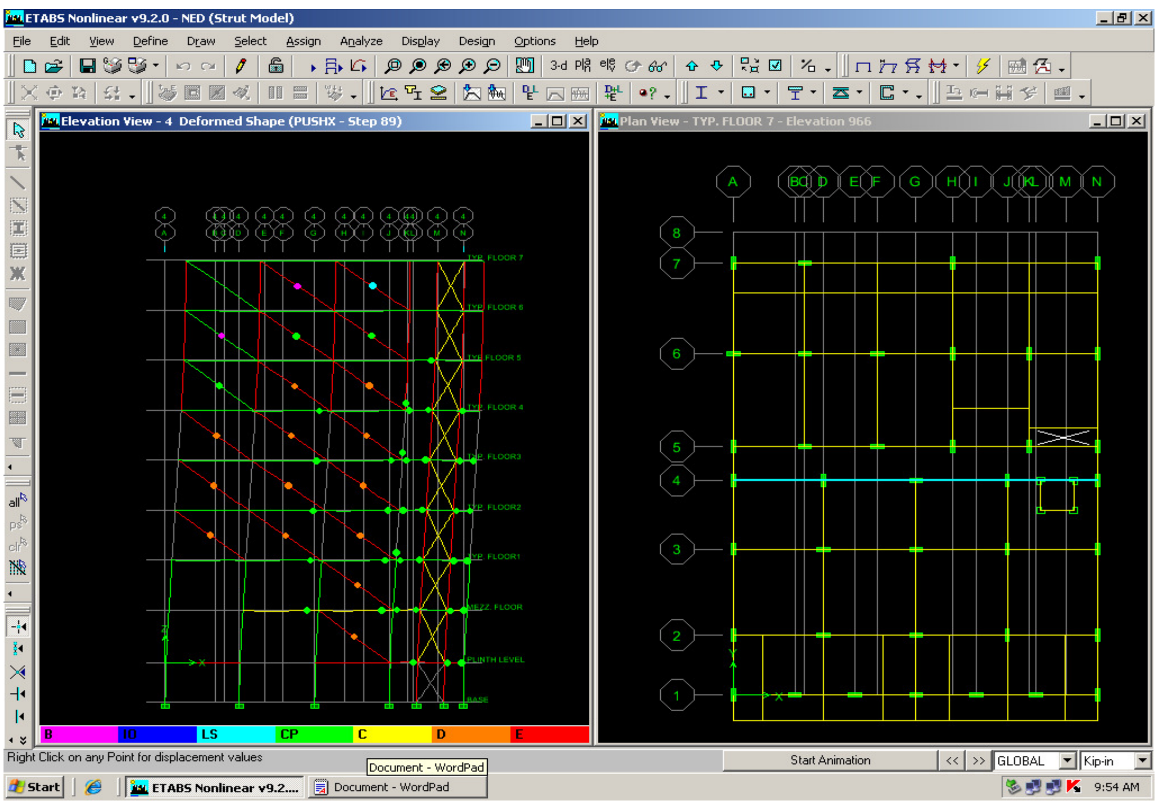
Results for non linear analysis along X-direction



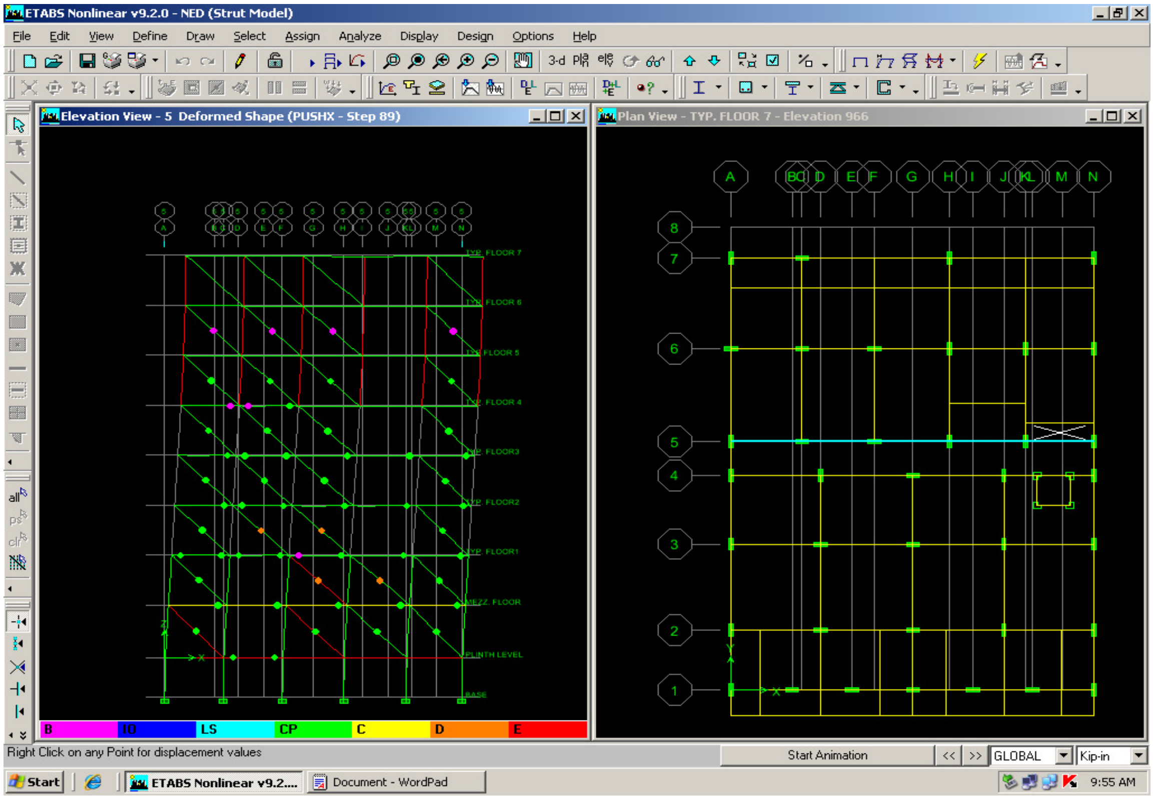


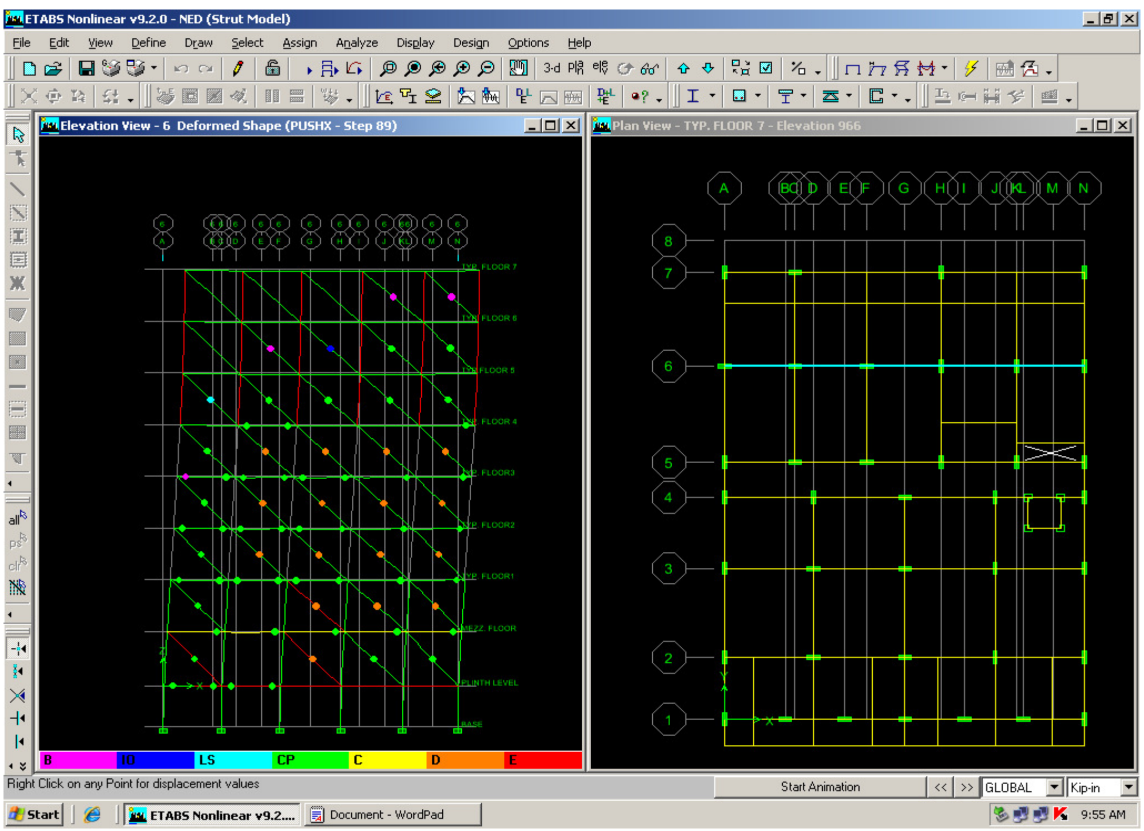
Mechanism or Deformed shapes at Performance Point - grid-2 (Top) grid-3 (bottom)



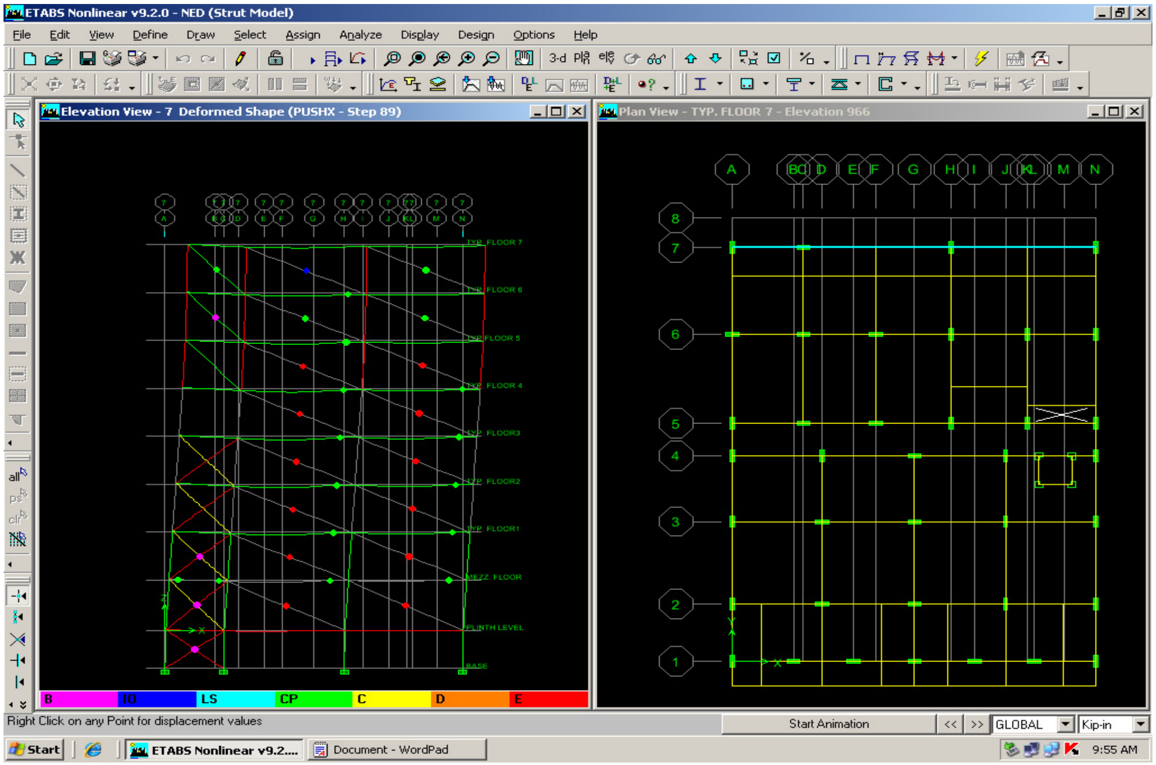


Mechanism or Deformed shapes at Performance Point - grid-3 (Top) grid-4 (bottom)



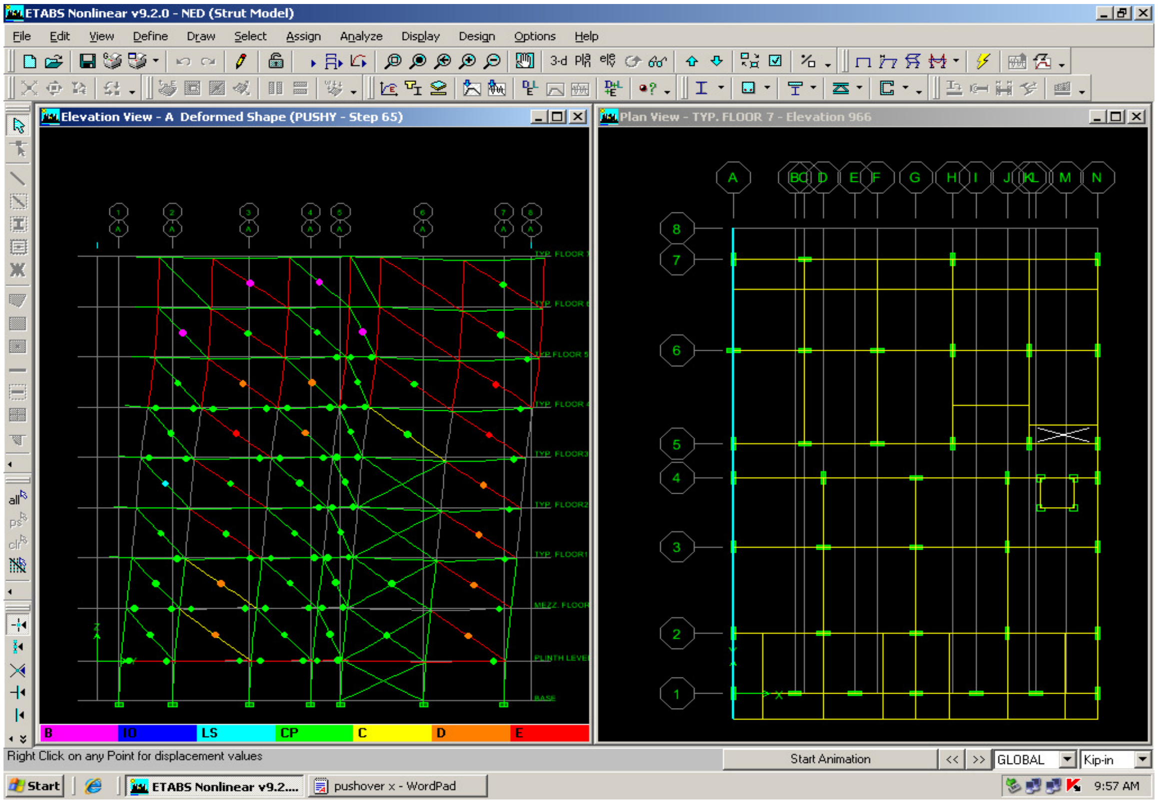


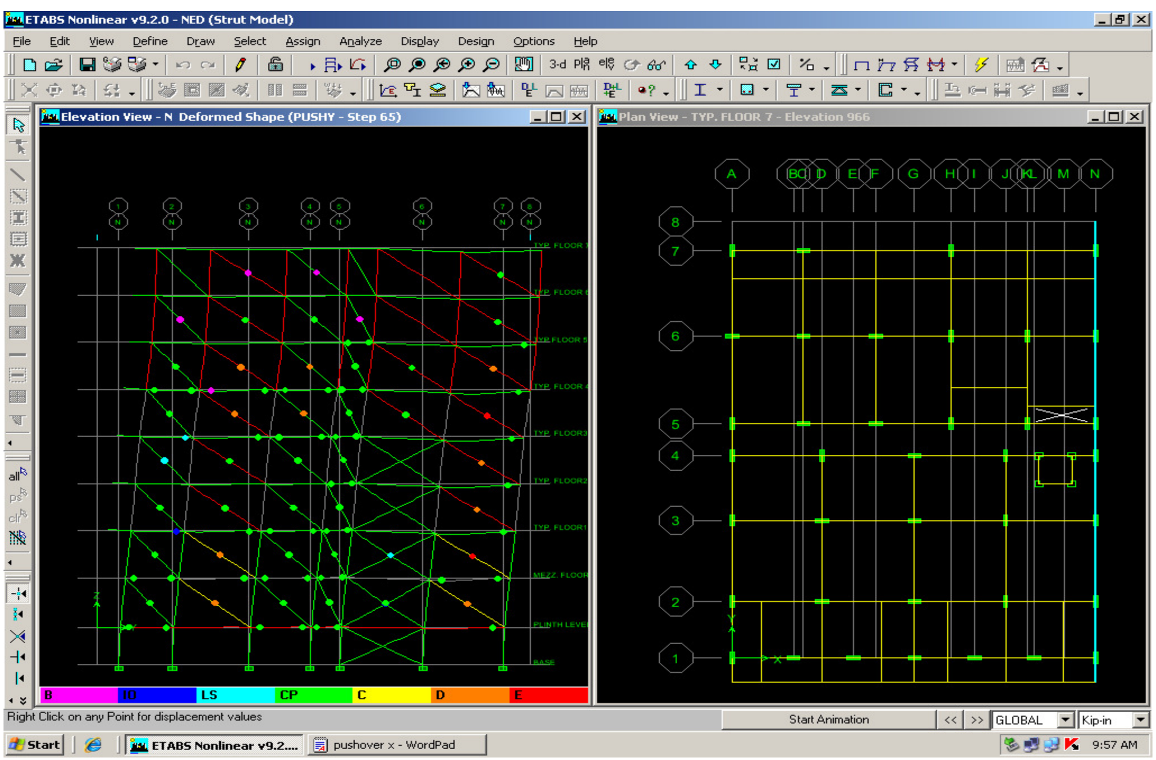
Mechanism or Deformed shapes at Performance Point - grid-5 (Top) grid-6 (bottom)



Mechanism or Deformed shapes at Performance Point - grid-7

Results for non linear analysis along Y-direction





Mechanism or Deformed shapes at Performance Point - grid-A (Top) grid-N (bottom)