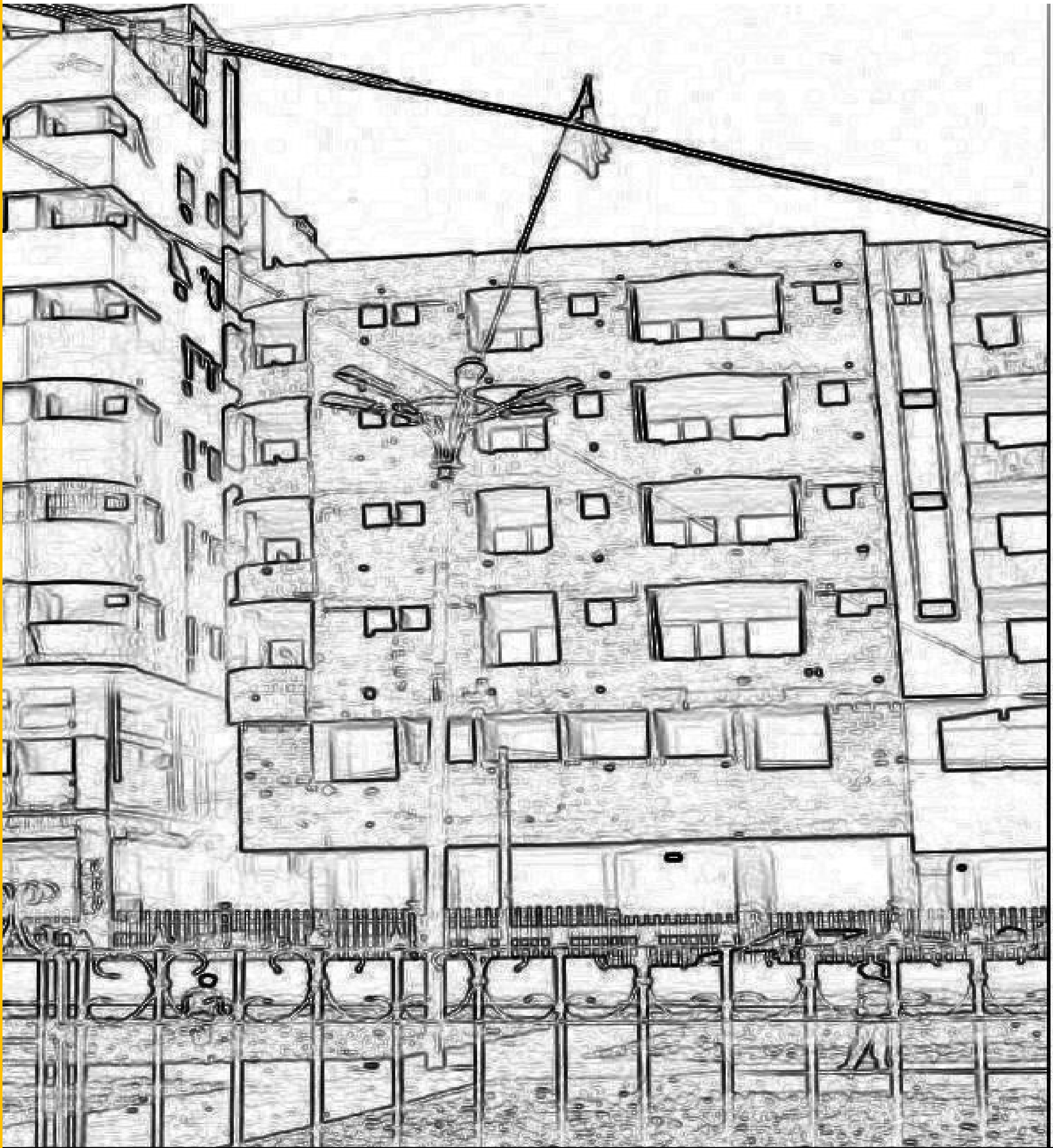

6-Storey Mixed Use Building in Karachi

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
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Summary

The building is located in Gulistan-e-Johar, a densely populated area in Karachi. An idealized two-dimensional frame from this building was studied earlier as the pilot case study. The results from that case study showed that the building needed retrofitting. It was then decided to do a non linear static analysis for the entire building. The building consists of reinforced concrete framed building with six storeys including the ground and mezzanine floors. The building has shops located at the ground and mezzanine floors, while the above floors are residential apartments. The building was constructed before the 2005 Kashmir Earthquake. Project participants selected this building 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 eccentrically located reinforced concrete core, and heavy, stiff unreinforced masonry infill walls that were not considered during the structural design of the building.

Moreover, 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 improve the capacity of the columns and the overall strength and deformation capacity of the structure. The columns were found to be marginal even under gravity loading, so the team decided to jacket a number of them, as well as replacing infill panels with reinforced concrete walls to form rocking spines. It was difficult to find locations to place spines due to the configuration of the building, but the team was able to obtain an acceptable solution by supplementing the base-to-roof spines with an additional shear wall in the weak ground and mezzanine storeys, and by making use of existing infill wall capacity in the upper storeys.

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. Adnan Rais, Lecturer, and Dr. Abdul Jabbar Sangi, Associate Professor, 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

This case study provided participants with the opportunity to expand the pilot case study to fully address the challenges posed by a building with major seismic vulnerabilities common in Karachi residential buildings, many of which have shops at the ground storey. The teams updated the Tier 1 analysis and performed Tier 2 (linear static analysis) and Tier 3 (nonlinear static analysis) on three dimensional models of the building. Team members were able to expand their capabilities with ETABS, a structural analysis software package from Computers and Structures, Inc. of Berkeley, California, and to better understand the ASCE/SEI 31-03 and ASCE/SEI 41-06 documents.

Building Information

Figure 1 shows the case study building (middle building). The building is a six storey (ground and mezzanine plus four typical floors with a basement) mixed use apartment building with shops at the ground and mezzanine floors. The building's overall dimensions 40' wide by 68' 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 an eccentrically located reinforced concrete core. The concrete block infill walls are 6 inch thick. The foundations are reinforced concrete spread footings. The building is recently constructed and no condition assessments have been made.

The beam sizes are 8"x24". The column heights were taken as 11 feet with two column sections from 12"x24" and 24"x24" from ground to roof floors. The slab thickness is 6 inches. Concrete f'c was taken as 3000 psi and f_y for steel was taken as 60000 psi.

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 1. Front view of the building (building in the middle) during construction and its neighbours

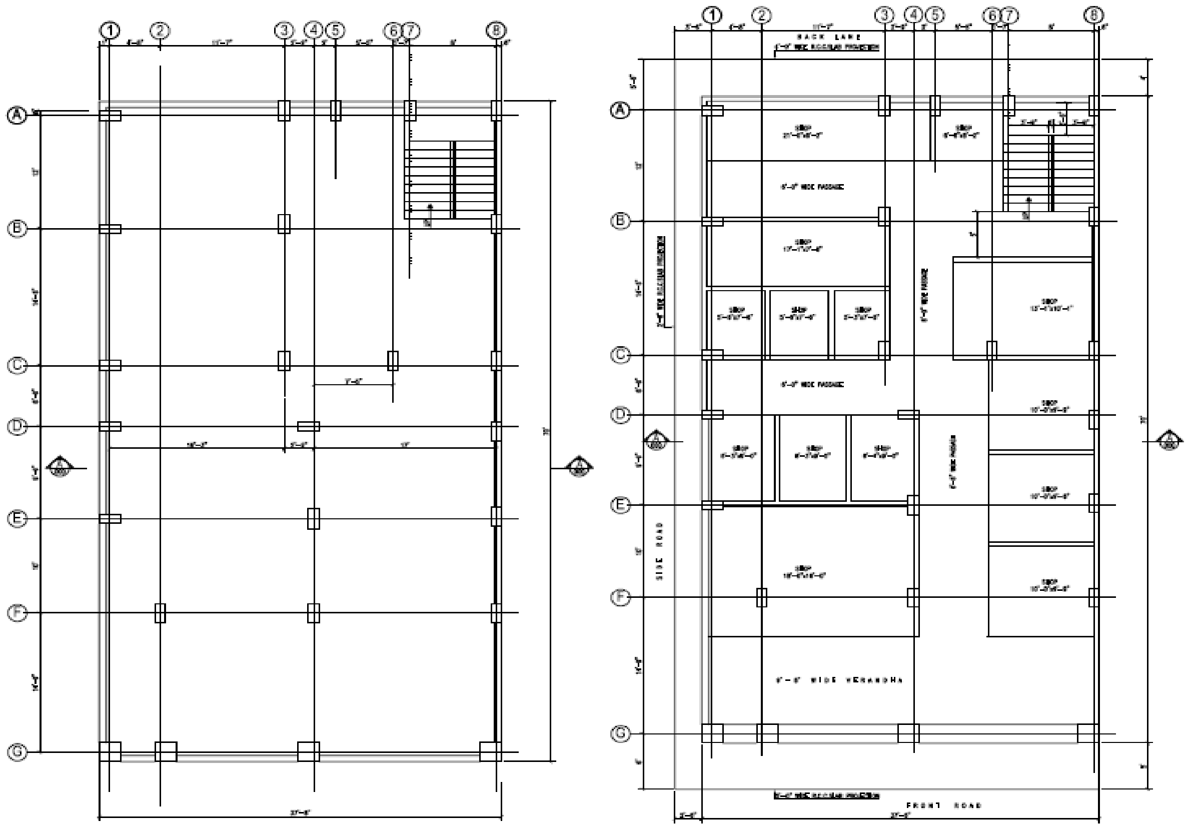

Figure 2. Architectural layout of basement (left) and ground floor (right)

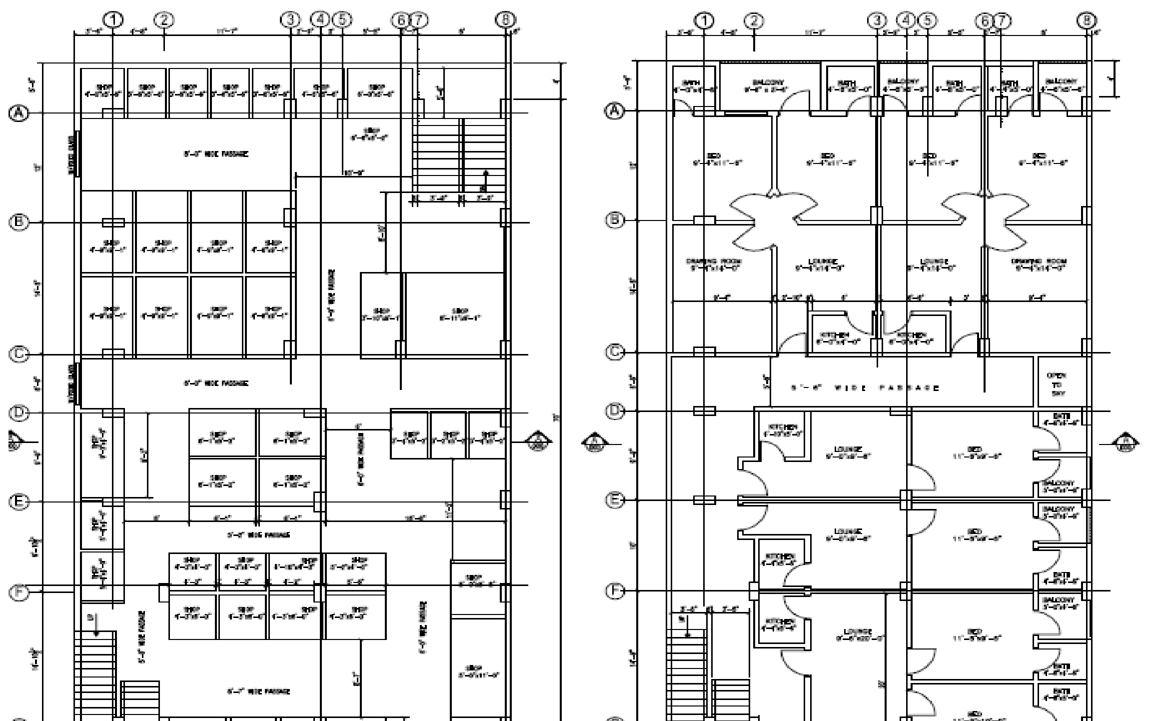

Figure 3. Architectural layout of mezzanine floor (left) and typical upper floors (right)

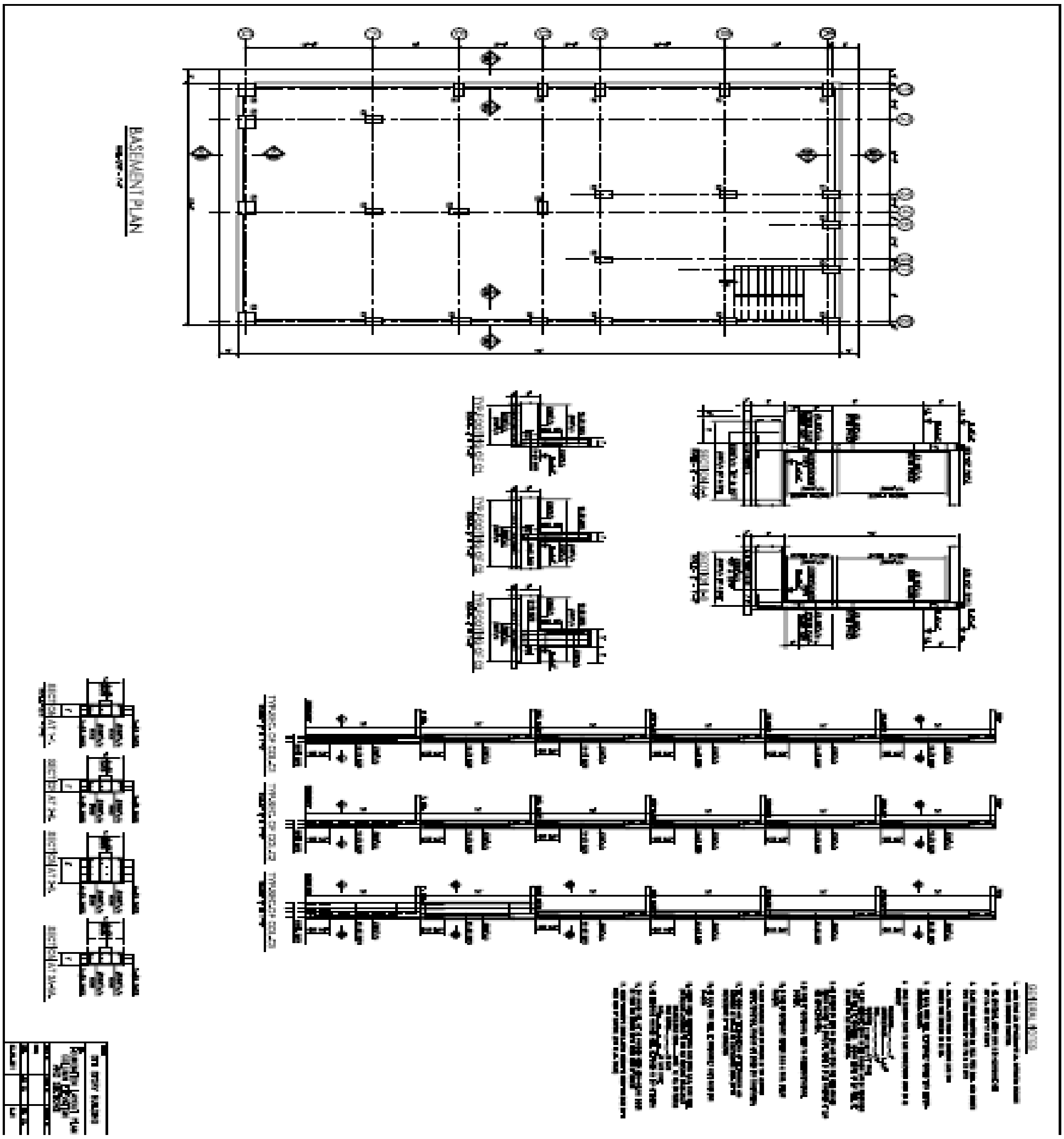

Figure 4. Structural layout of basement

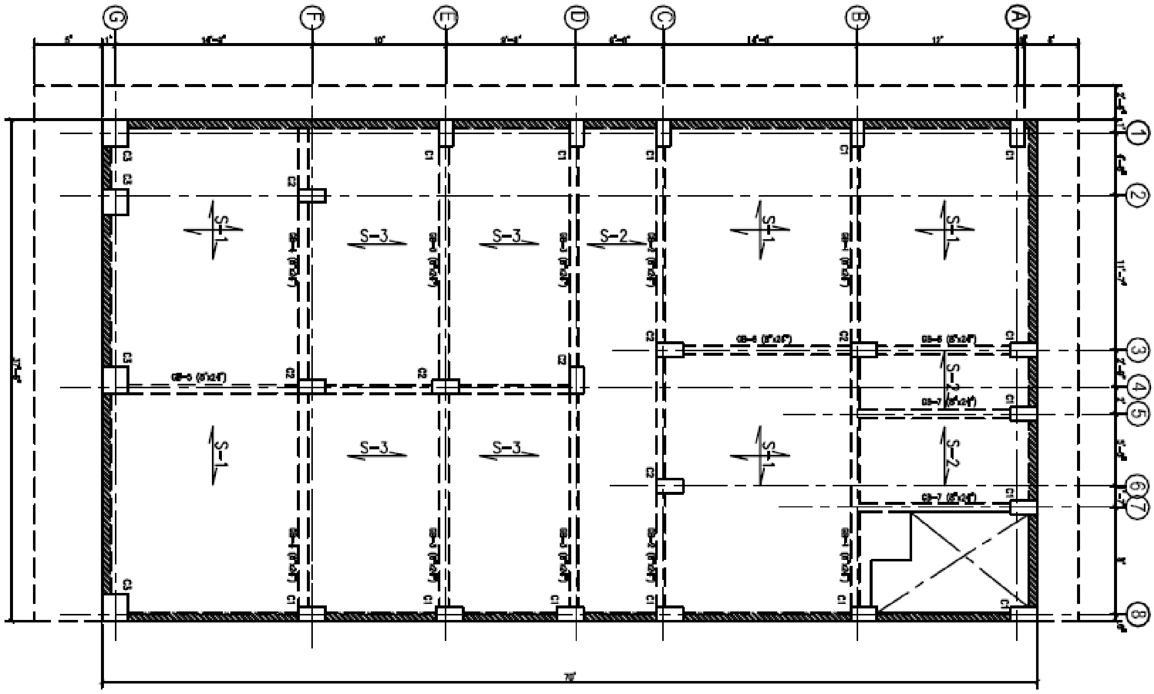

Figure 5. Structural layout of ground floor

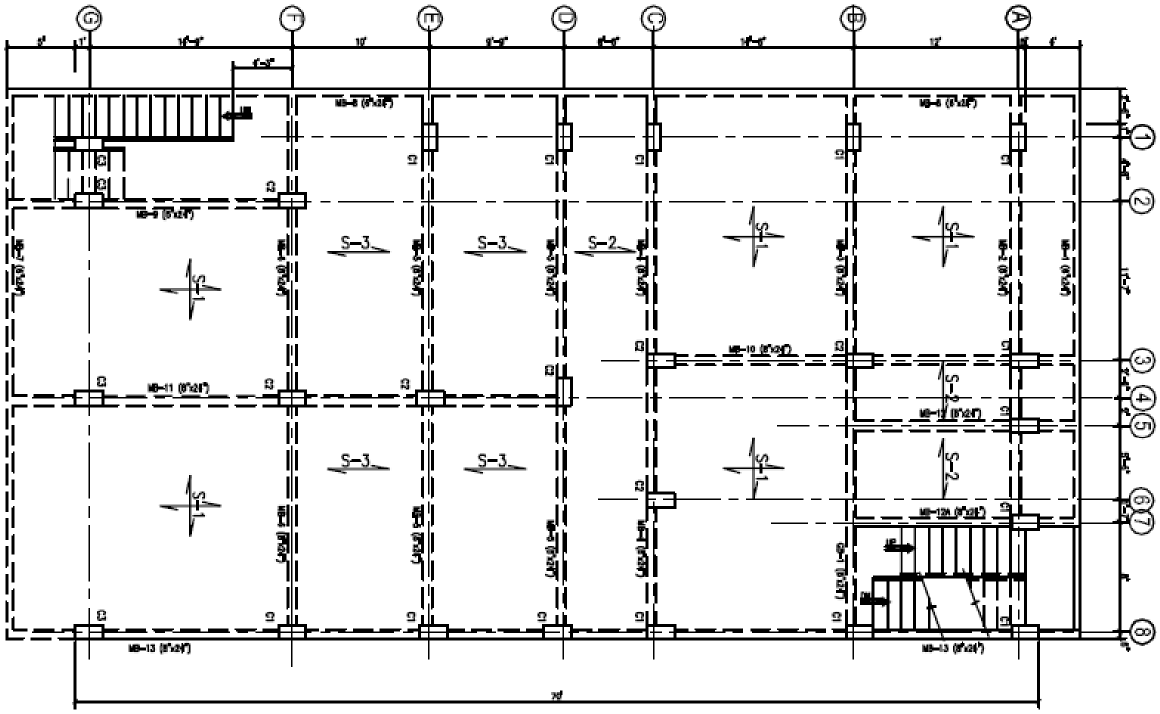

Figure 6. Structural layout of mezzanine floor

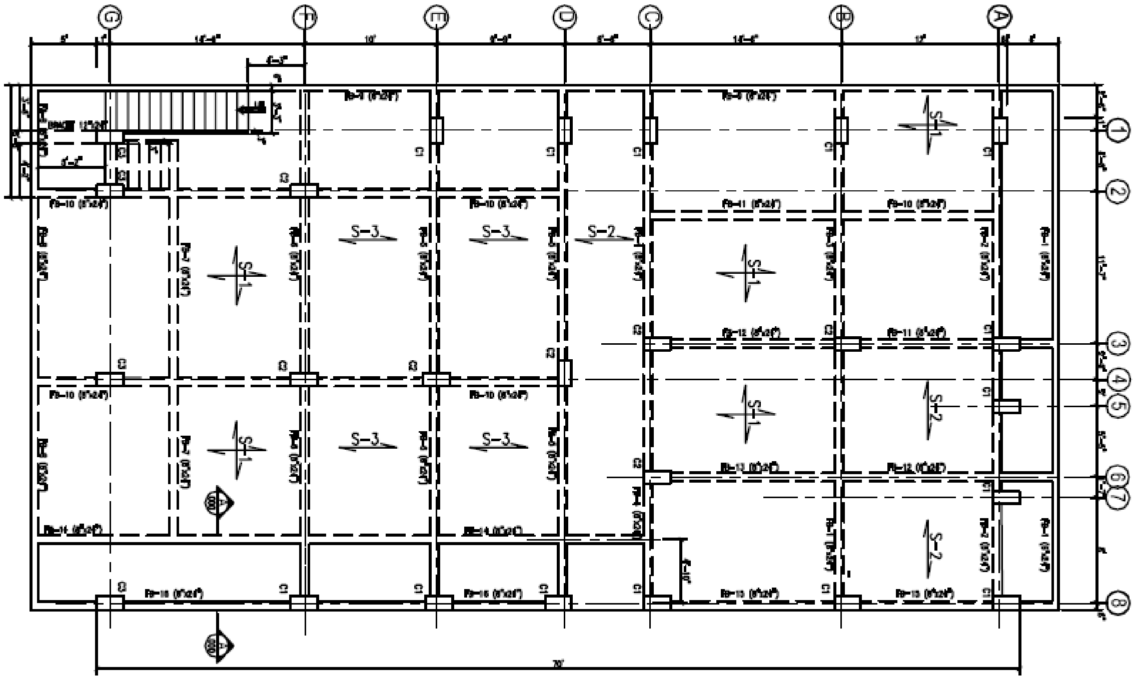

Figure 7. Structural layout of typical upper floors

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
Lateral Force-resisting System	Interfering wall Shear stress check Axial stress check Proportion of infill walls Over all construction quality
Geologic Hazards and Foundation	None

Linear Evaluations of Existing Building

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 so the building is expected to respond in the nonlinear range. Please see Appendix B for linear analysis and Appendix C for nonlinear analysis results for the existing building.

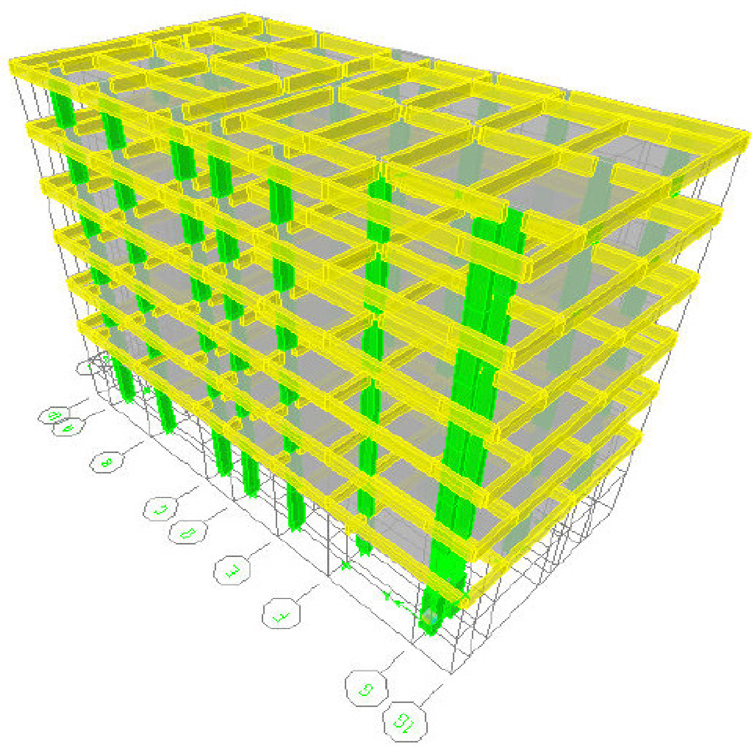

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Figure 8. 3-D finite element model in ETABS

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 IN X	%DIFF IN Y
RF	D1	16.691	34.654	25.19	37.136	21	3
4F	D1	16.784	34.784	25.074	37.051	21	3
3F	D1	16.784	34.784	24.972	37.012	20	3
2F	D1	16.784	34.784	24.809	36.98	20	3
1F	D1	16.784	34.784	24.473	36.945	19	3
MF	D1	16.001	33.845	23.243	36.365	18	3
GF	D1	16.983	32.61	16.868	31.848	0	-1

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 the fourth and roof floors have torsion irregularity as per ASCE 31, because the difference between centre of mass and centre of rigidity is greater than 20% for each storey.

Table 2. Soft storey check

Story	Load	storey force	Total Displacement	Stiffness	% diff in K
		kips	inches	kip/in	30% allow
ROOF	EX	87.56	1.696776	51.60	
4F	EX	70.24	1.635444	42.95	16.8
3F	EX	56.34	1.45863	38.63	10.1
2F	EX	41.7	1.112184	37.49	2.9
1F	EX	27.56	0.555978	49.57	32.2
MF	EX	12.93	0.251964	51.32	3.5

Table 2 shows that the first floor does not comply with the stiffness criteria and may be a soft storey.

Table 3. Storey drift check

	IN X direction			IN Y direction		
Story	Load	Etab DriftX	final drift	Load	Etab DriftY	final drift
RF	EX	0.014884	0.0573034	EY	0.000041	0.00015785
4F	EX	0.014346	0.0552321	EY	0.000096	0.0003696
3F	EX	0.012795	0.0492608	EY	0.000144	0.0005544
2F	EX	0.009756	0.0375606	EY	0.000181	0.00069685
1F	EX	0.004877	0.0187765	EY	0.000186	0.0007161
MF	EX	0.002333	0.0089821	EY	0.000141	0.00054285
GF	EX	0.000061	0.0002349	EY	0.000014	0.0000539

Table 3 shows that the calculated interstorey drift for second to roof floors exceeds the allowable drift limit of 0.02 in the X-direction.

Detailed Evaluations of Existing Building

Through linear static analysis of this building, the checks for building system (soft storey, 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 some columns were failing under gravity loads and many had DCR > 1. This required further non linear static analysis. The nonlinear static pushover analysis based on performance-based seismic design was adopted and hinge properties according to ATC-40 and ASCE 41-06 criteria were evaluated and manually assigned to beams, columns, and struts in the 3-D model.

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. Figure 9 shows how plastic hinge force-deformation relations are defined in ASCE/SEI 41-06. IO, LS and CP are the Immediate Occupancy, Life Safety and Collapse Prevention performance levels, respectively.

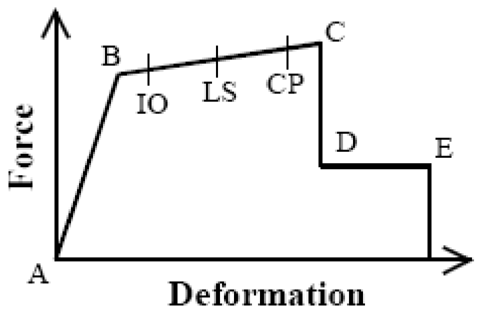


Figure 9. Force-deformation relation for hinges (reprinted from public domain document FEMA 356, the precursor to ASCE/SEI 41-06) showing the definition of acceptance criteria and performance states

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 4 gives the geometric and material properties used in the model.

Table 4. Properties of nonlinear model

Geometric Properties			
Beam	Width = 12 in		
	Depth = 24 in		
	@ Support	Top R/F	As' = 2.64 in^2

		Bottom R/F	As = 0.88 in^2
	Length = 24 ft		
Column	Width = 18 in		
	Depth = 18 in		
	Exterior Column	from base to 2^{ND} floor from 3^{RD} floor to roof	As = 8 # 8 = 6.3 in^2 As = 8 # 6 = 3.5 in^2
	Interior Column	from base to 2^{ND} floor from 3^{RD} floor to roof	As = 16 # 8 = 12.6 in^2 As = 8 # 8 = 6.3 in^2
	Height = 12 ft		
Ordinary Infill Wall Strut	Width = 6in		
	Depth = 36.6 in		
Material Properties			
	fc' = 3000psi for beam and column		
	E_{con} = 3144 ksi for beam and column		
	For ordinary strut fc' = 300 psi		
	E_{mas} = 214.5 ksi		

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. The building is being evaluated for life safety. Table 5 shows ETABS loading input parameters.

Table 5. ETABS loading input parameters

Gravity load: **Dead load** **Live load**	Loads from slab floors were manually calculated and assigned to the slab supporting beams in the 3-D model. Self wt of frame + 6" thick slab + 2" thick finishes + 50psf wall load 50psf on floor and 30psf on roof
Earthquake load: **Z** **R** **C_a** **C_v**	 0.4g 5.5 $0.4N_a$ (Ref: Table 16-Q (UBC 97)) with N_a = 1.0 $0.4N_v$ (Ref: Table 16-R (UBC 97) with N_v = 1.0
Soil type	S_B (Ref: Table 16-J UBC-97)

Analysis Results

Figure 10 shows the load-deformation curve, or *pushover* curve, and Figure 11 shows the pushover curve converted into a capacity spectrum and compared with the estimated demand using the capacity spectrum method. This figure 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.

The results of the non linear analysis shown in Figure 12 and Appendix C show the state of the nonlinear hinges at the performance point. These results confirm that retrofitting is needed to achieve stability and to prevent failure at the acceptance level.

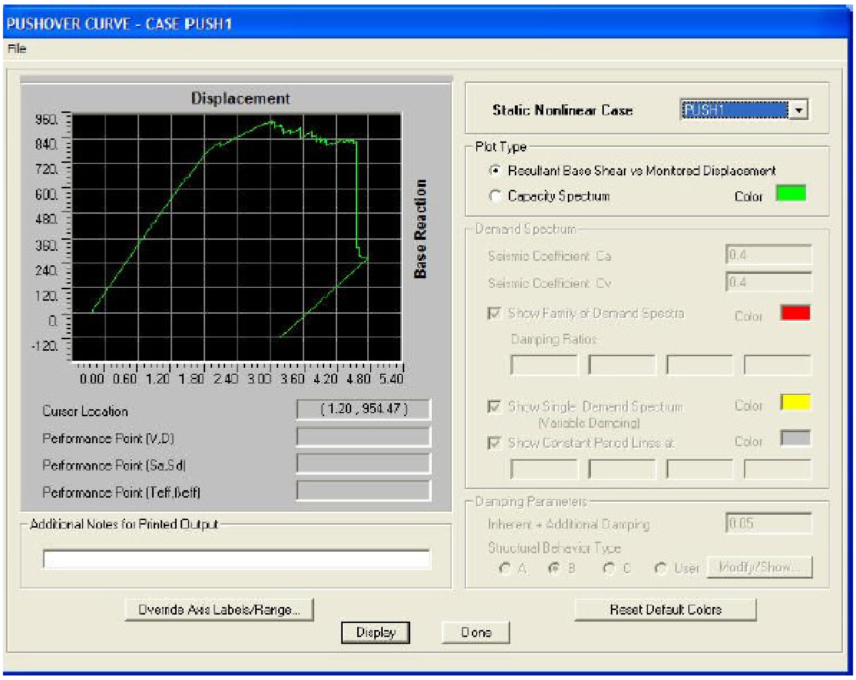


Figure 10. Pushover curve for Seismic forces in X-direction

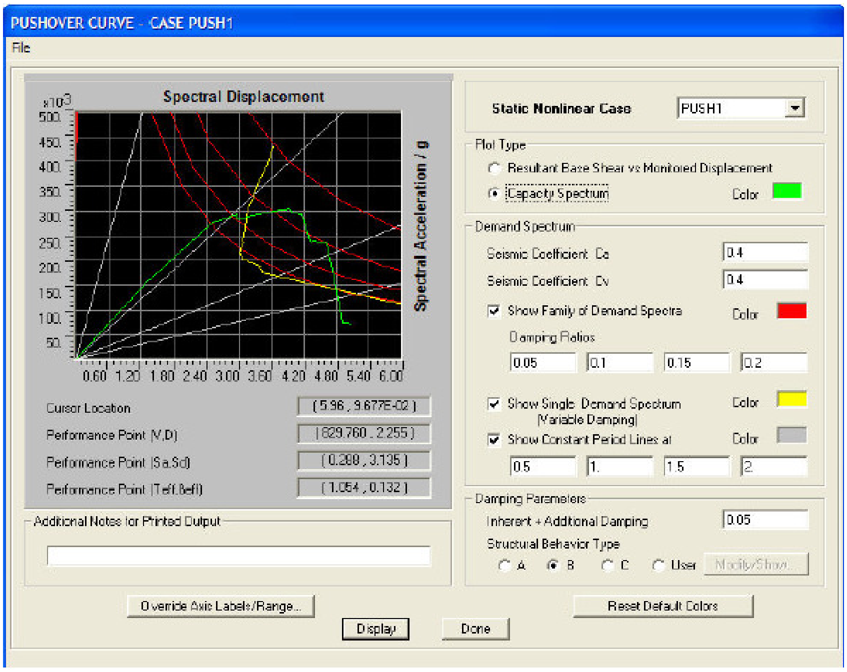


Figure 11. Performance level for seismic forces in X-direction

Figure 12 shows the state of the nonlinear hinges at the performance point for X-direction analysis. Note the column hinges in the weak ground storey.

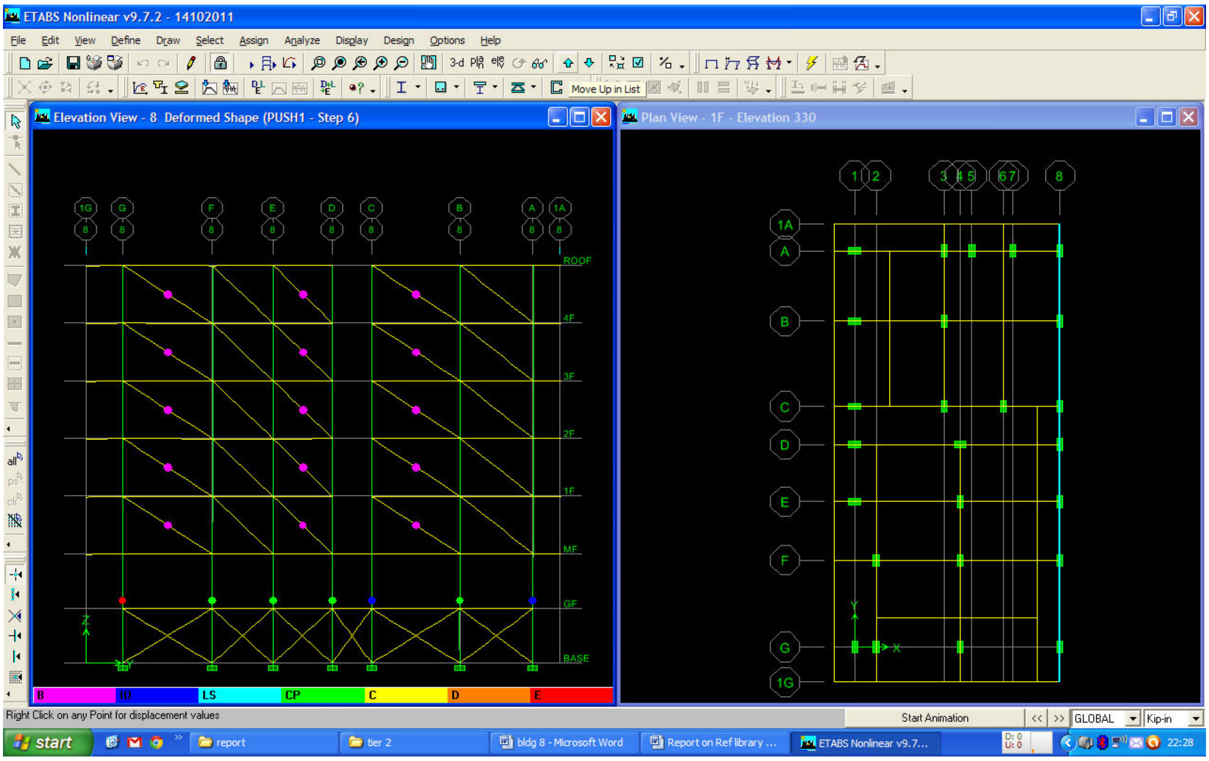


Figure 12. Hinge deformation vs. acceptance criteria

Retrofit Solution

Conceptual Solutions Considered

As some columns were failing under gravity loads and many columns had DCR>1, the team decided to jacket the columns first and replace the ordinary masonry infill walls with new RCC shear walls. However, due to architectural limitations, shear walls could only be provided at certain locations. The team investigated several possibilities before finalizing the locations shown in Figure 13 along with the columns to be jacketed. The highlighted column sizes are increased to 18"x30" and two different thicknesses of RCC walls are provided. Walls 9" thick go from base up to roof level and walls 12" thick between grid B and C only go from base to first floor.

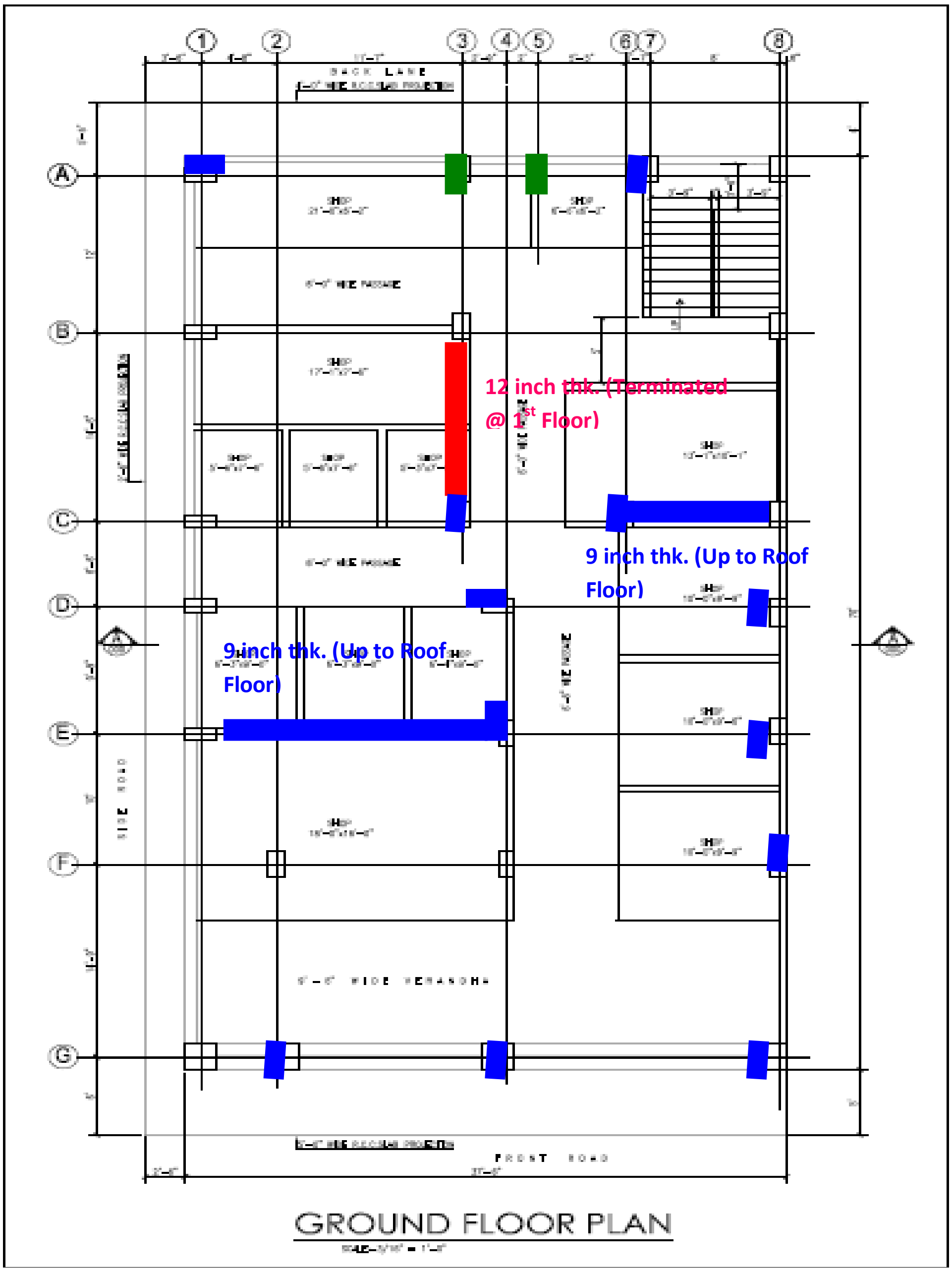
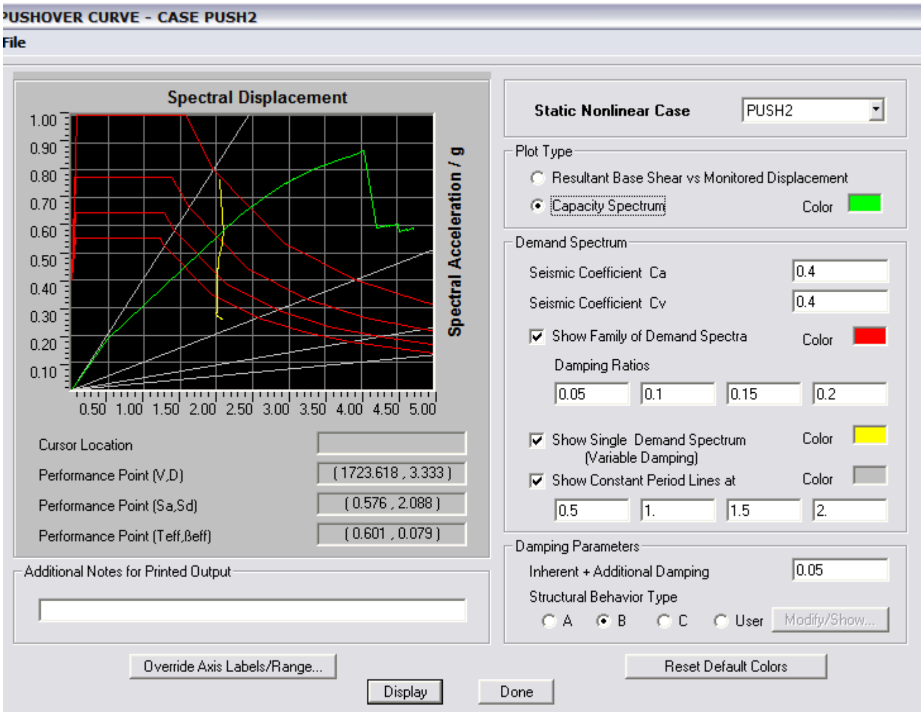


Figure 13. Proposed retrofit scheme

Comparison of Analysis Results

The retrofit scheme was modeled by taking the same 3-D nonlinear model used for the existing building, but increasing the column size and adding RCC infill walls modeled with single strut members. Figure 14 shows the pushover curve and capacity curve for the retrofitted building.



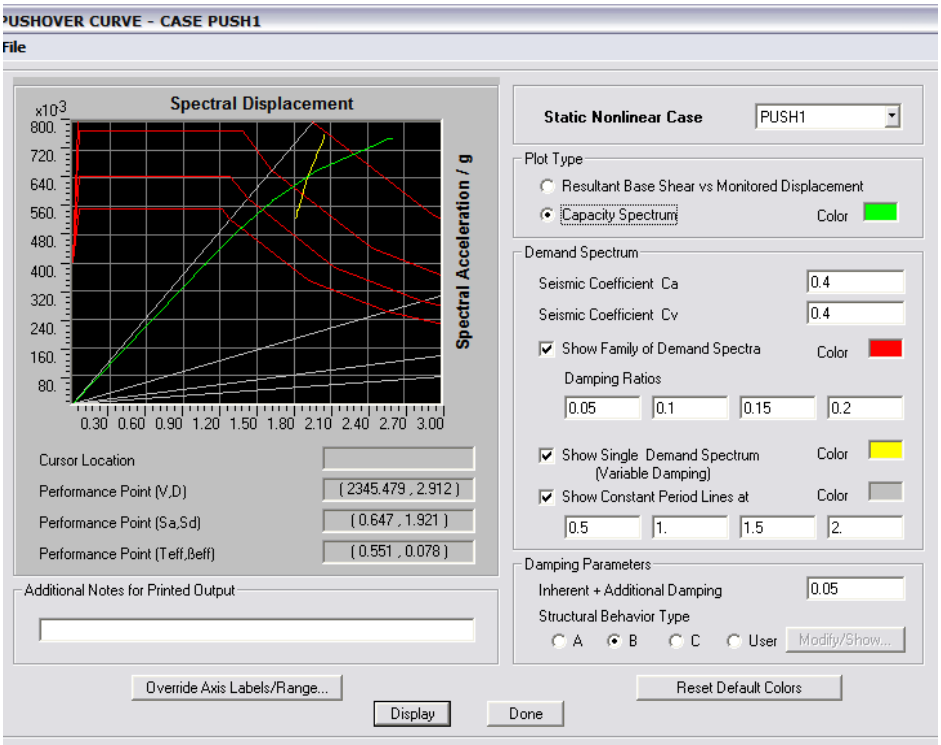


Figure 14. Performance level for the retrofitted building – Y-direction (top) X-direction (bottom)

Figure 15 shows a comparison of the state of the building's plastic hinges for one of the frames both before and after retrofit. It is clear that the retrofit greatly improves the performance and prevents the ground storey columns from hinging. Figures showing the deformed shapes of the remaining frames and the status of the plastic hinges for the retrofitted building are given in Appendix D of this report.

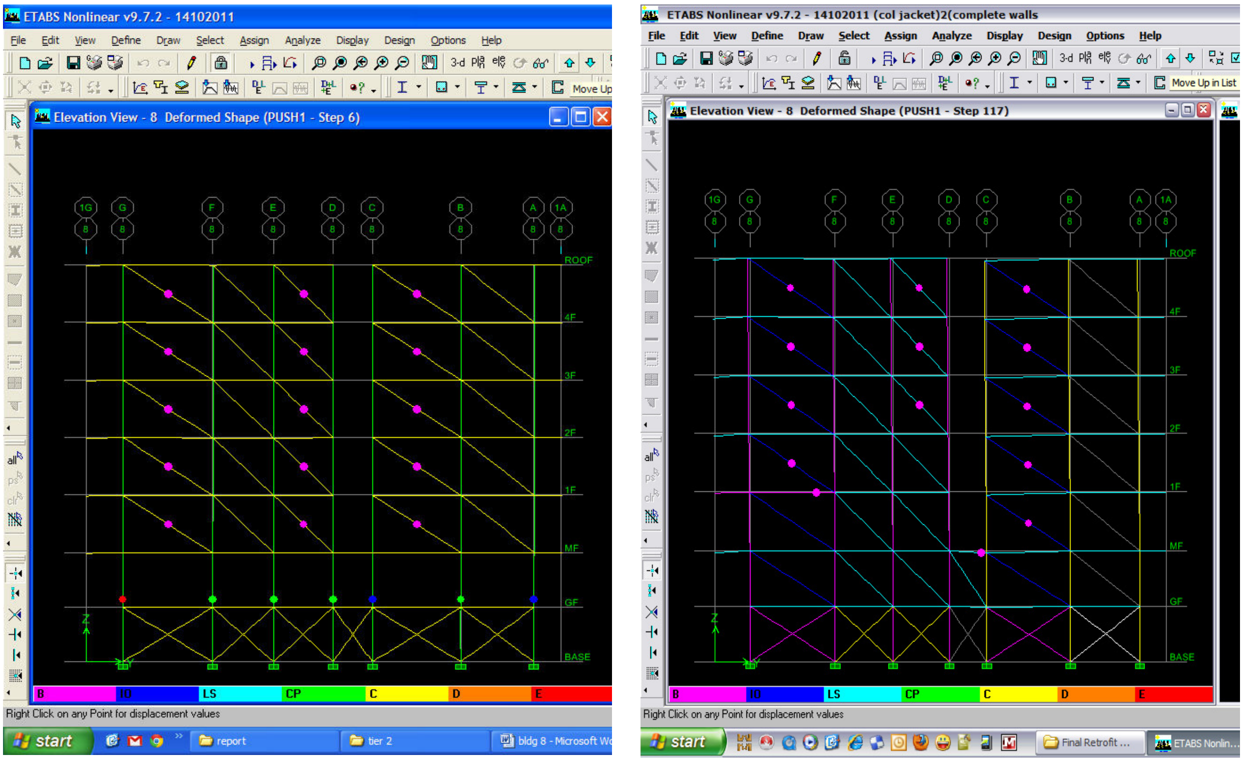


Figure 15. Hinge deformation vs. acceptance criteria (grid-8) before retrofit (left) after retrofit (right)

Design and Detailing

Details for the reinforced concrete spine and column jackets are shown in Figure 16 and Figure 17, respectively. Appendix E contains the full set of retrofit drawings.

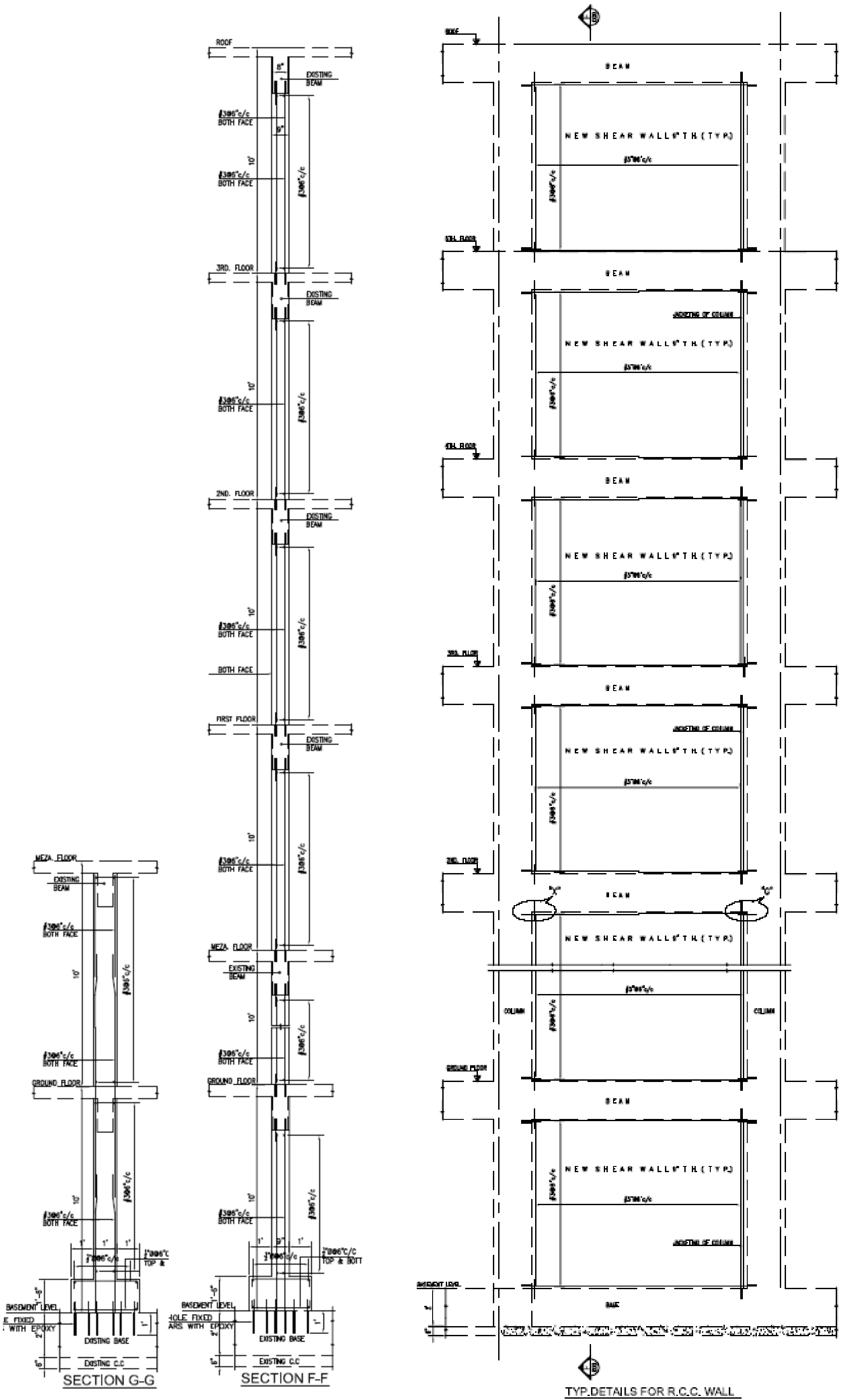


Figure 16. Proposed retrofitting details for new RCC shear walls

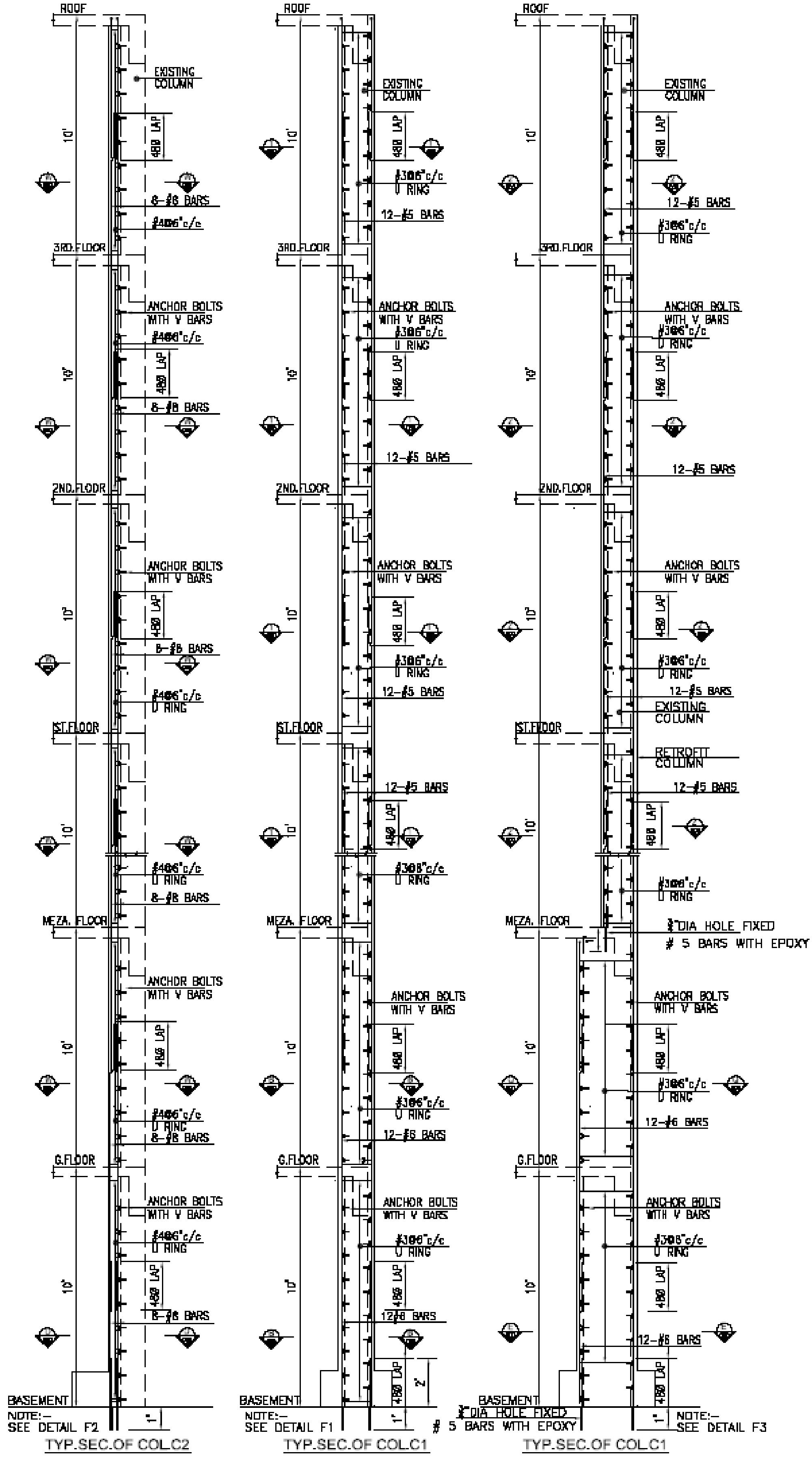


Figure 17. Proposed retrofitting details – column jacketing

Observations and Future Work

The retrofitted columns located on grid-8 will have eccentric section, as there is hardly any space on one side of the buildings due to an adjacent building as shown in Figure 1. However, in the retrofitted 3-D non linear analysis model, the sections have been taken as concentric. This may result in some additional forces in the columns, but the overall strength is quite high as can be seen in Figure 15, as hinges have yet to be formed in the retrofitted columns.

Similarly due to architectural limitations reinforced infill panels for spines and reinforced concrete shear walls could only be placed at certain locations in the building interior, which may not provide the most cost effective or least disruptive solution. Further work is needed to develop additional low-cost retrofit options that can be applied to buildings with difficult configurations.

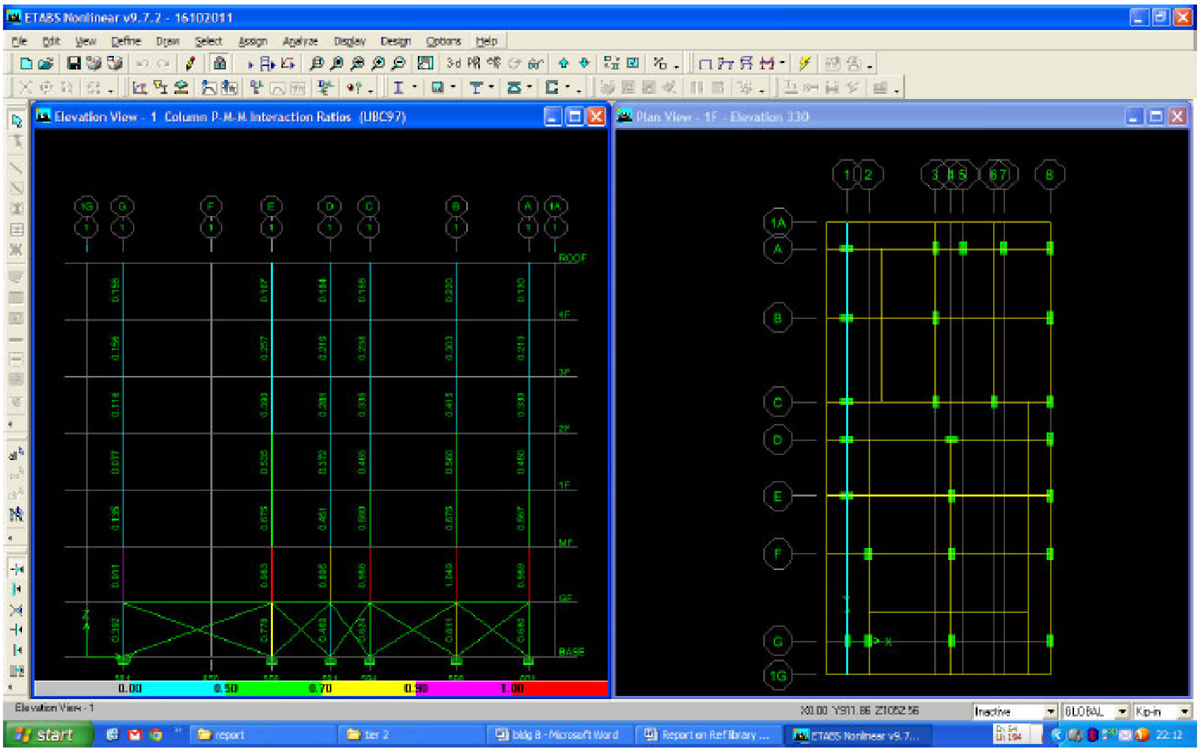
Appendix A: Tier 1 Checklists

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

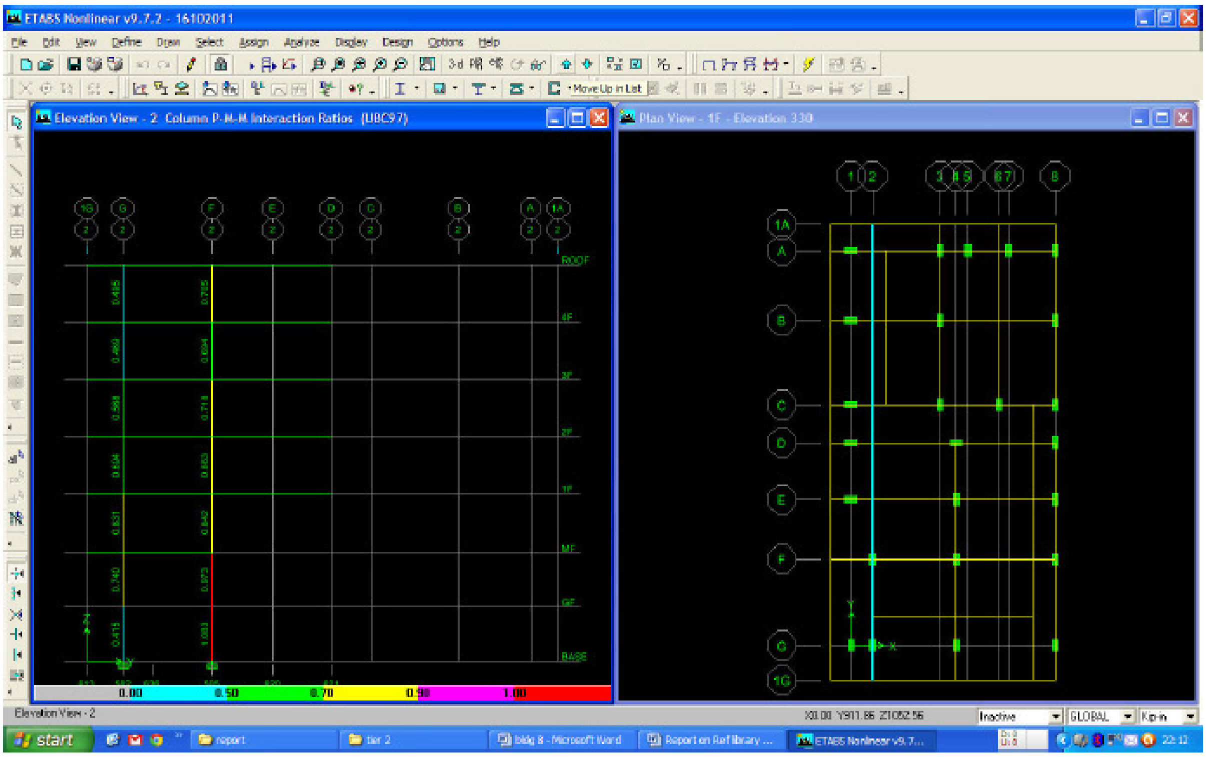
LATERAL-FORCE RESISTING SYSTEM	
Redundancy	C
Interfering Wall	NC
Shear Stress Check	NC
Axial Stress Check	NC
Proportion of Infill Walls	NC
Concrete Columns	C
Solid Wall	C
Over All Construction Quality	NC
Flat Slab Frames	NA
Pre-stressed Frames	NA
Captive Column	NC
Column Aspect Ratio	C

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

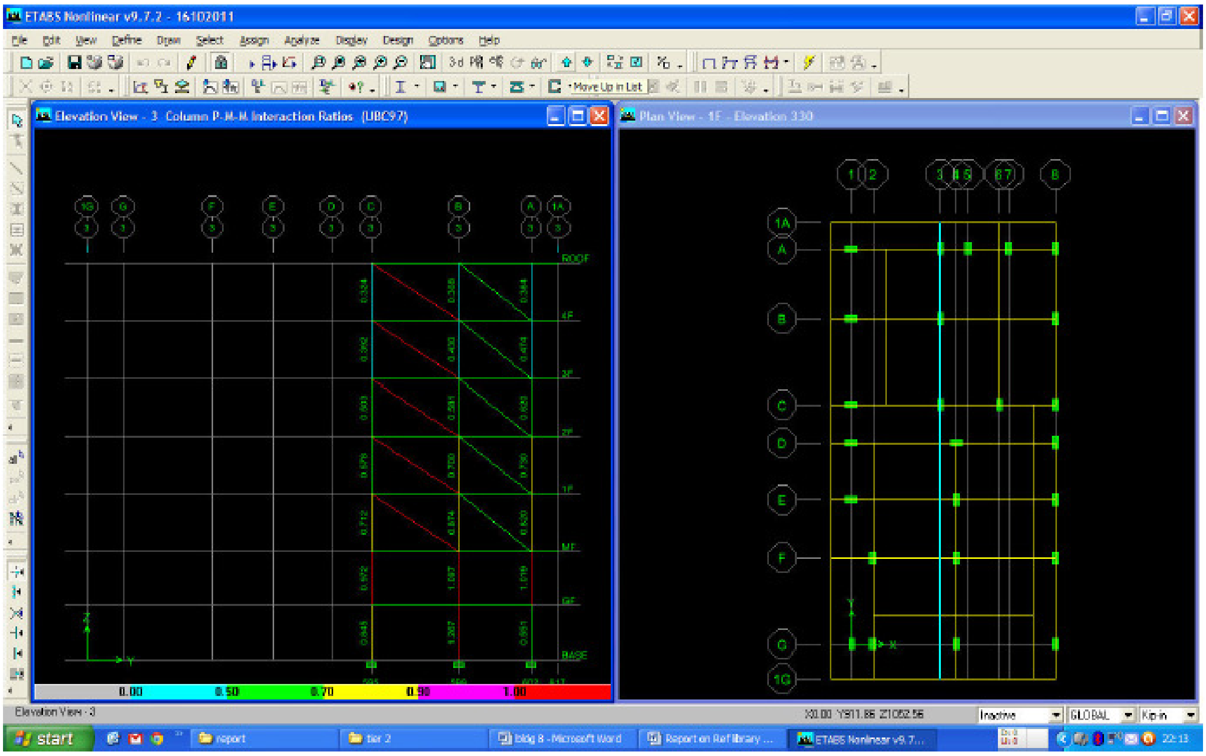
Appendix B: Linear Analysis (Tier 2) Results



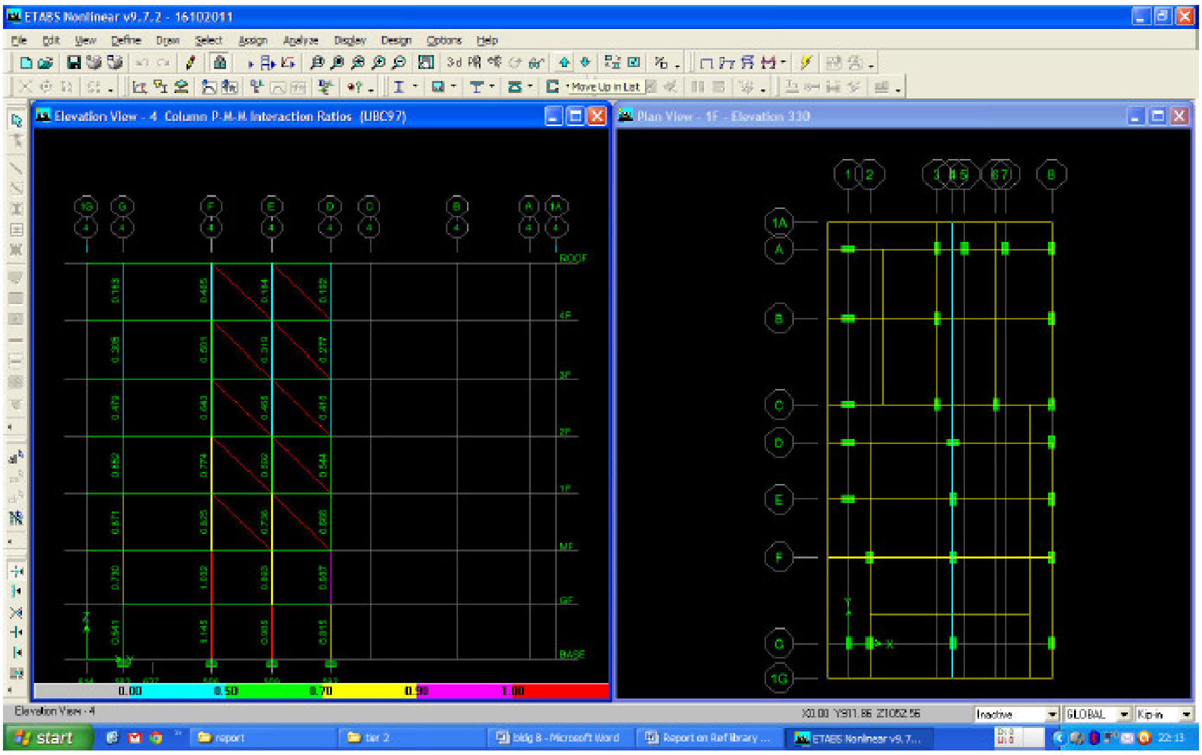
Demand/Capacity Ratios for Frame at Grid-1



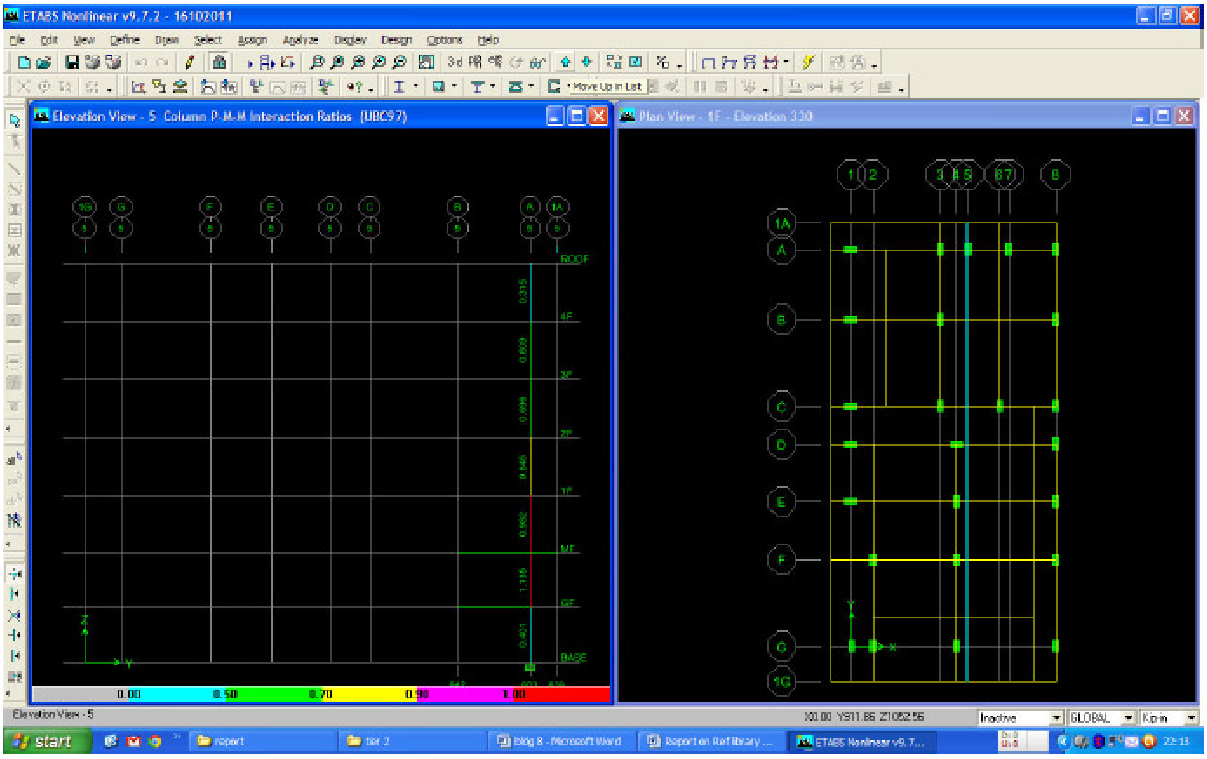
Demand/Capacity Ratios for Frame at Grid-2



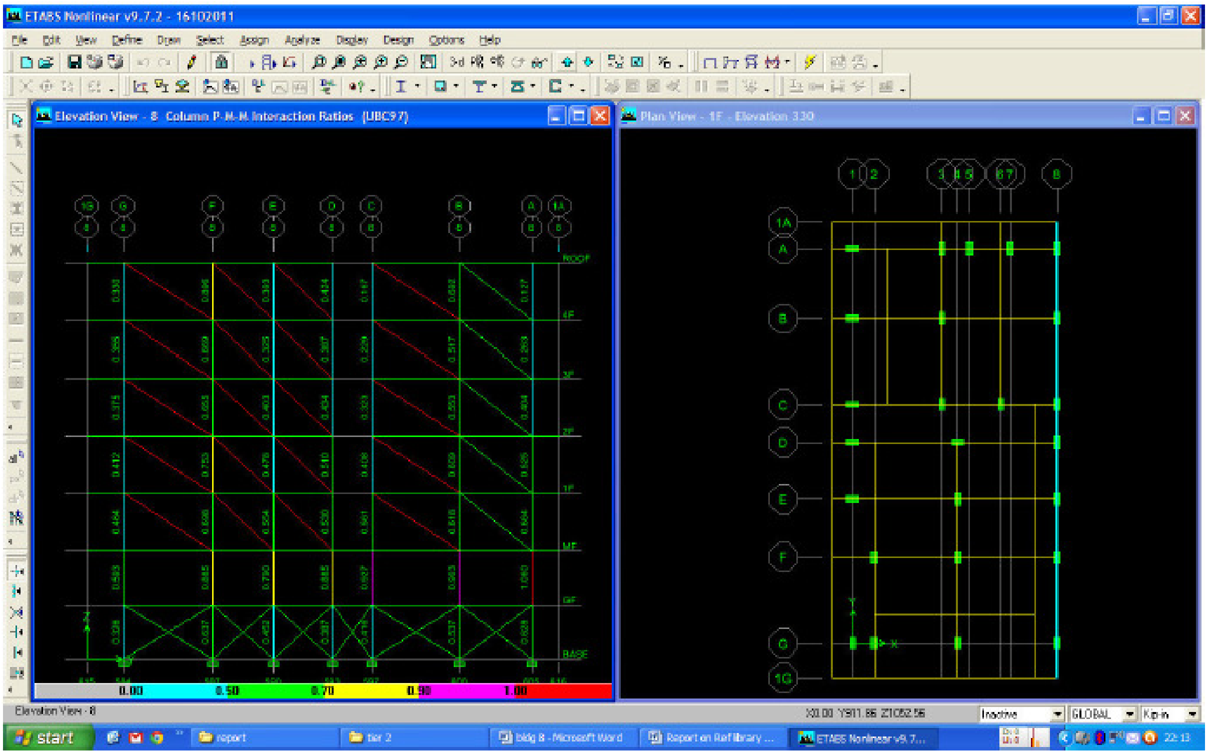
Demand/Capacity Ratios for Frame at Grid-3



Demand/Capacity Ratios for Frame at Grid-4

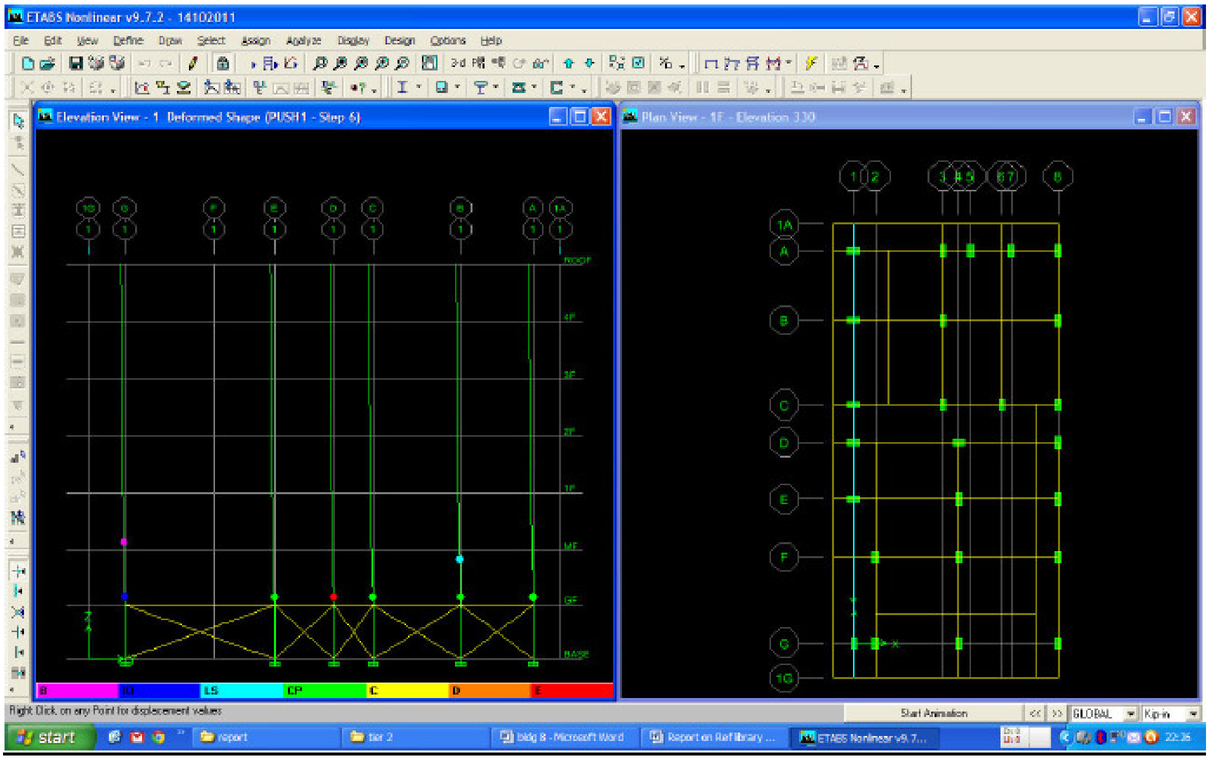


Demand/Capacity Ratios for Frame at Grid-5

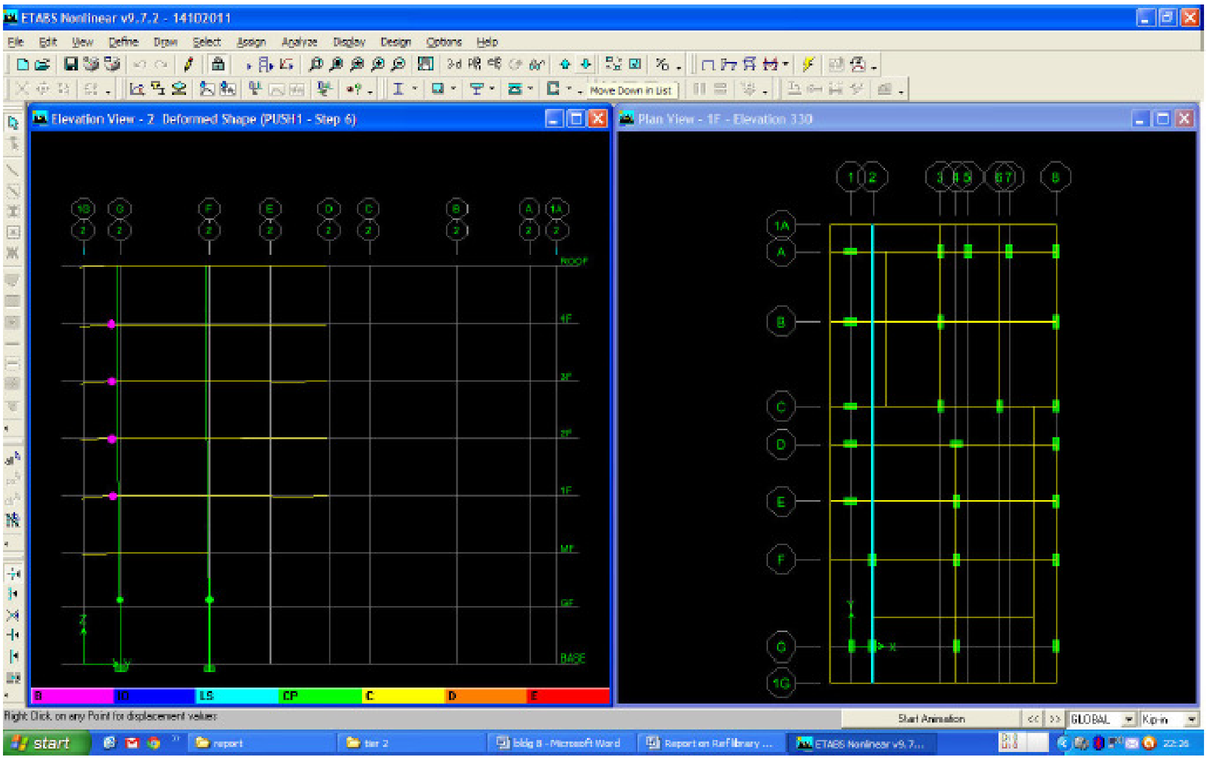


Demand/Capacity Ratios for Frame at Grid-8

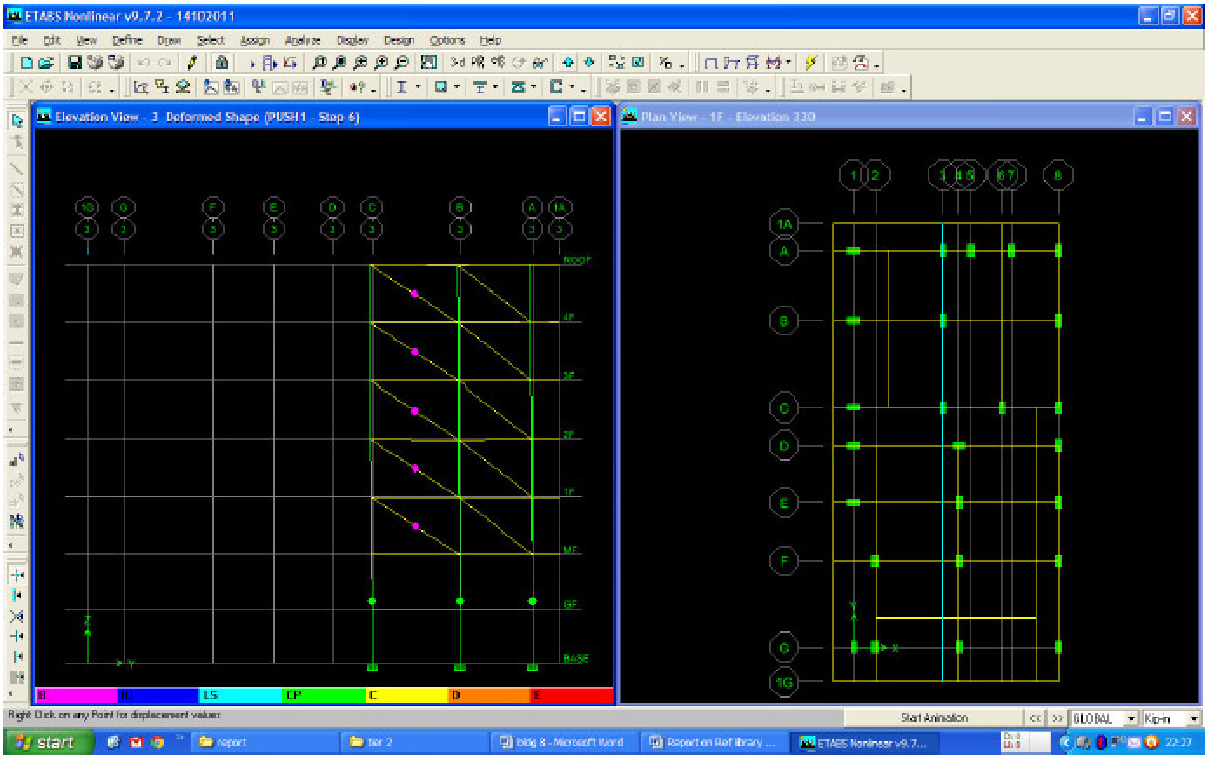
Appendix C: Non Linear Analysis (Tier 3) Results Before Retrofit



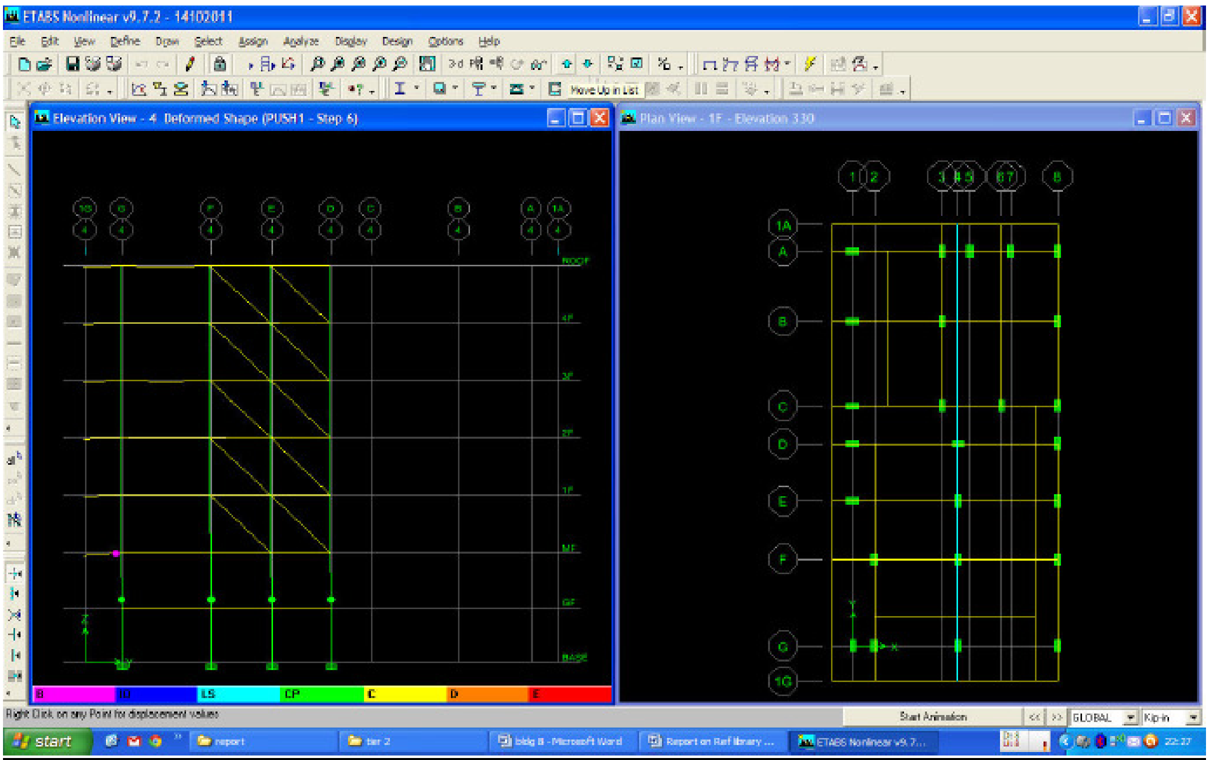
Mechanism or Deformed shapes at Performance Point - grid-1



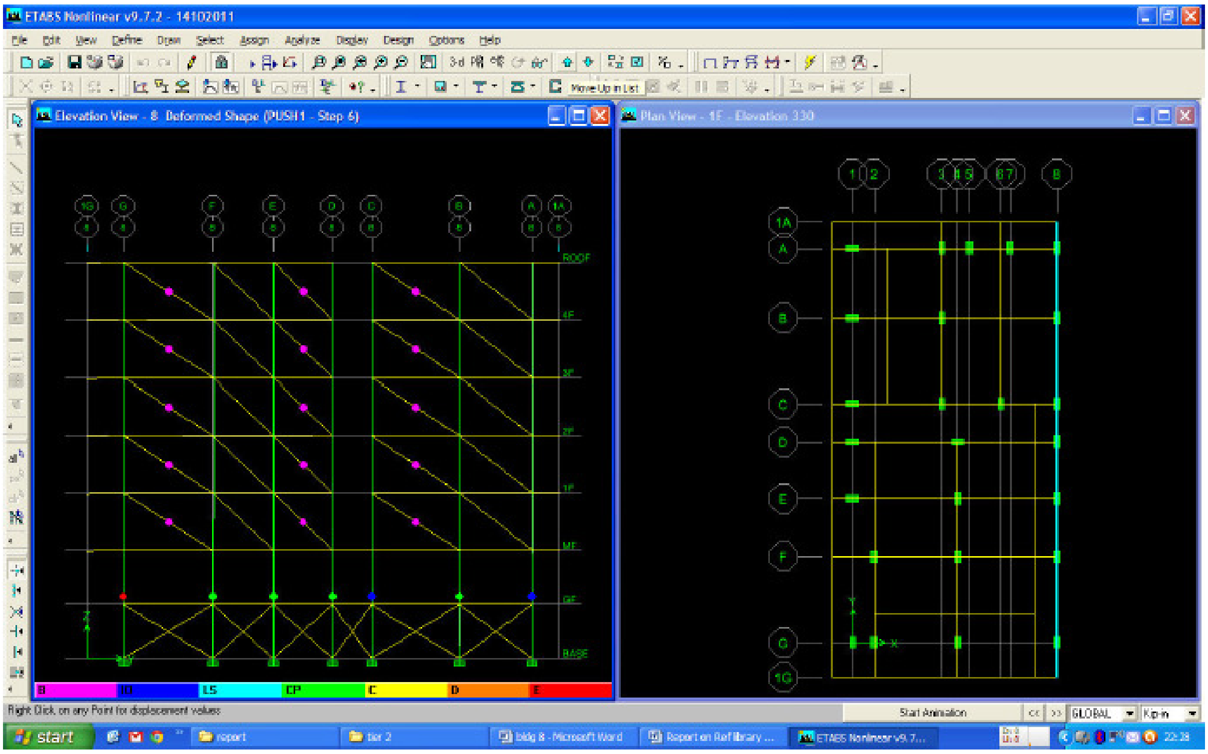
Mechanism or Deformed shapes at Performance Point - grid-2



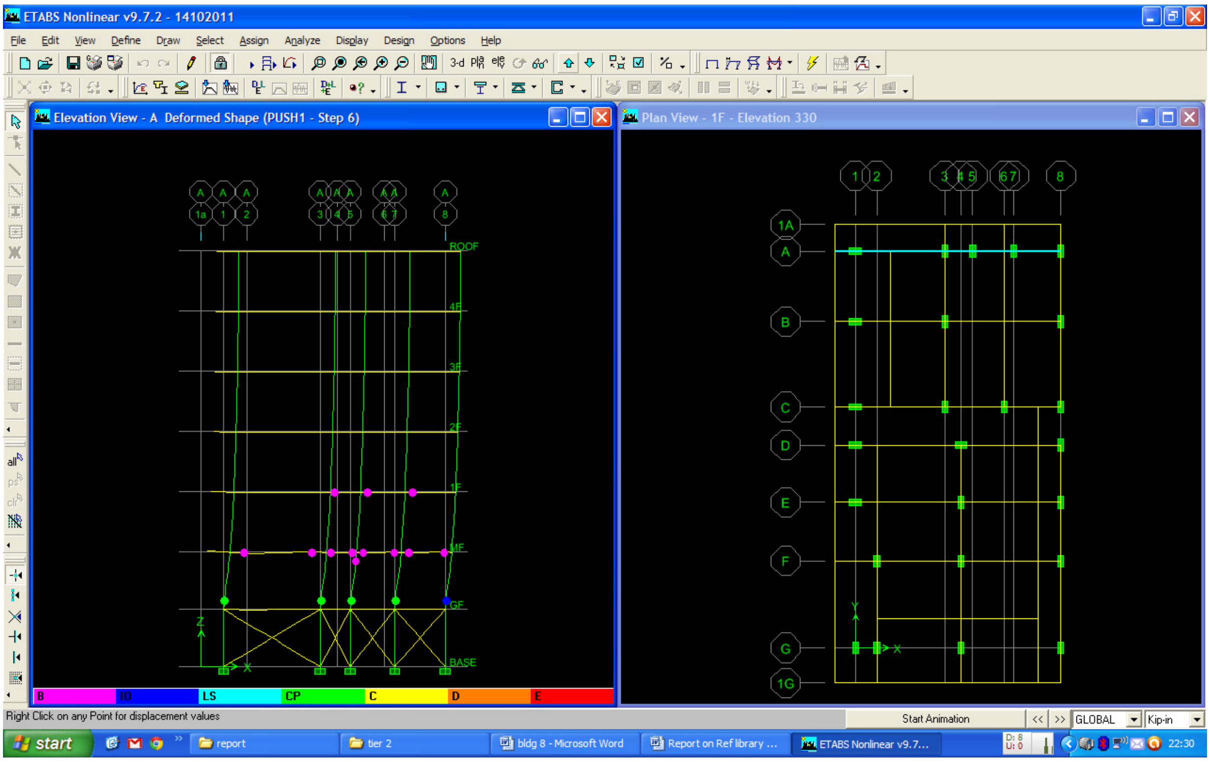
Mechanism or Deformed shapes at Performance Point - grid-3



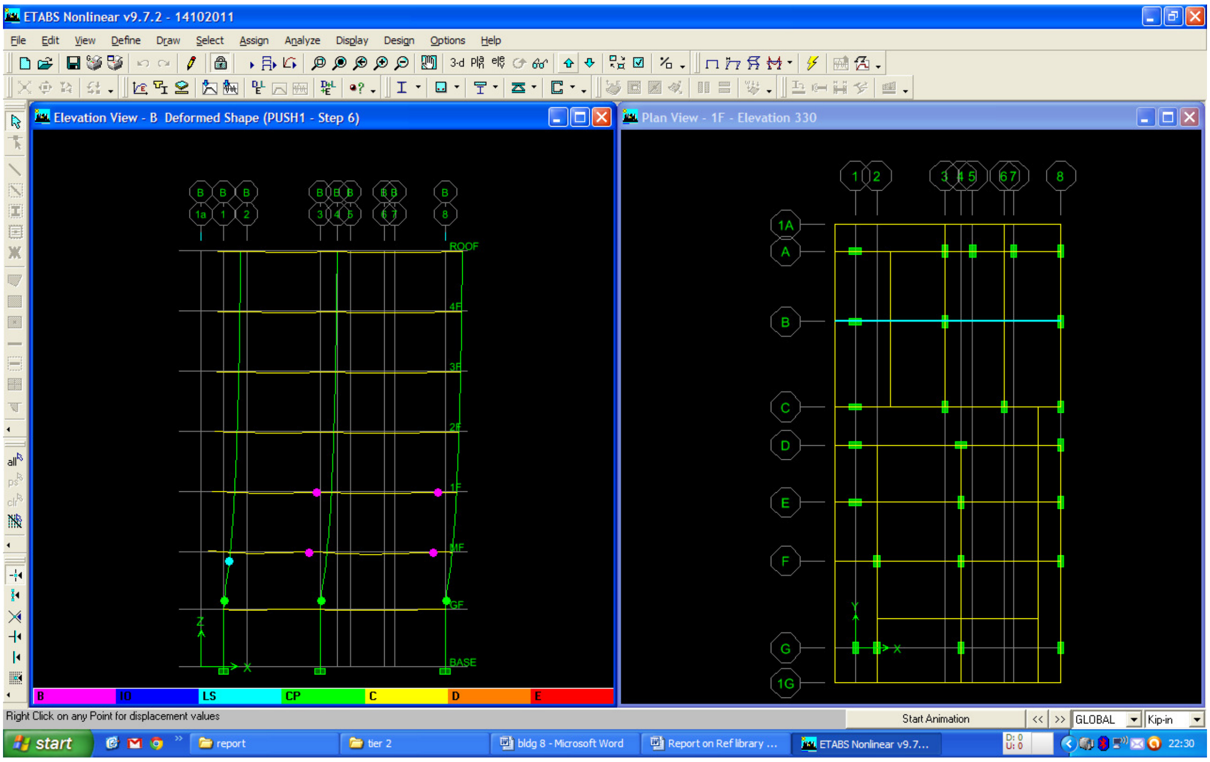
Mechanism or Deformed shapes at Performance Point - grid-4



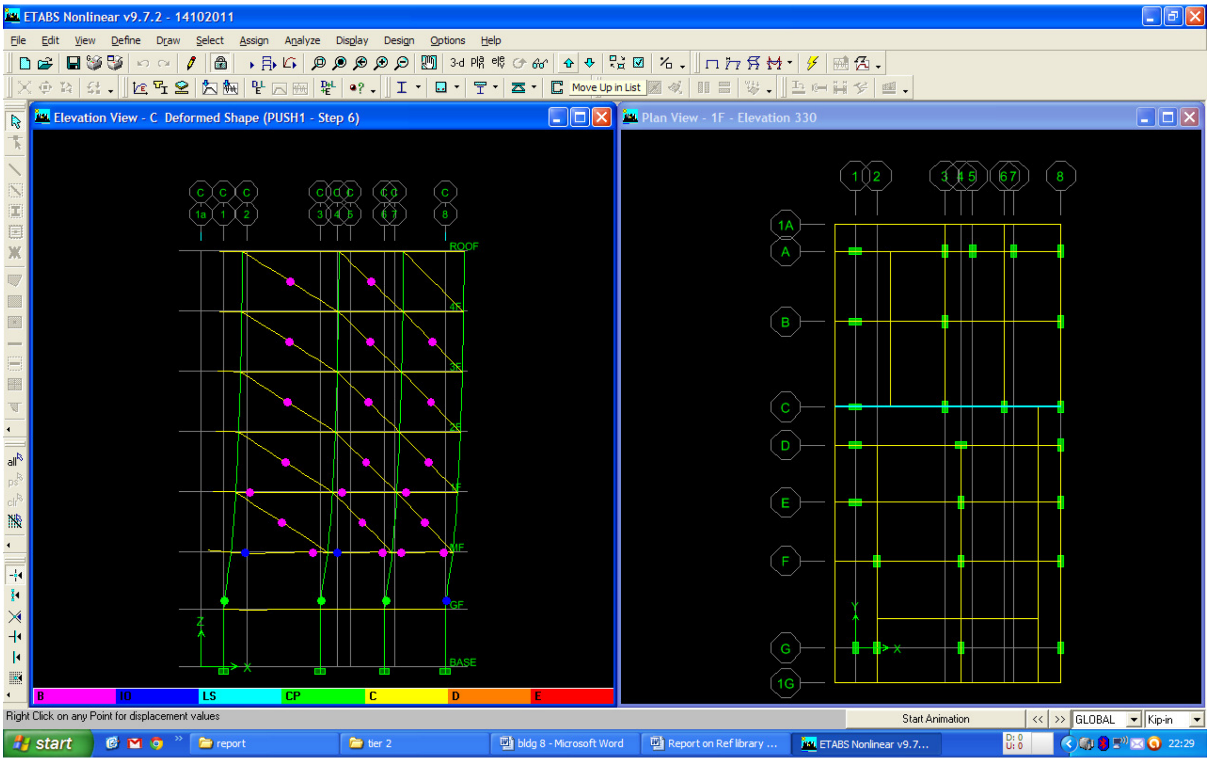
Mechanism or Deformed shapes at Performance Point - grid-8



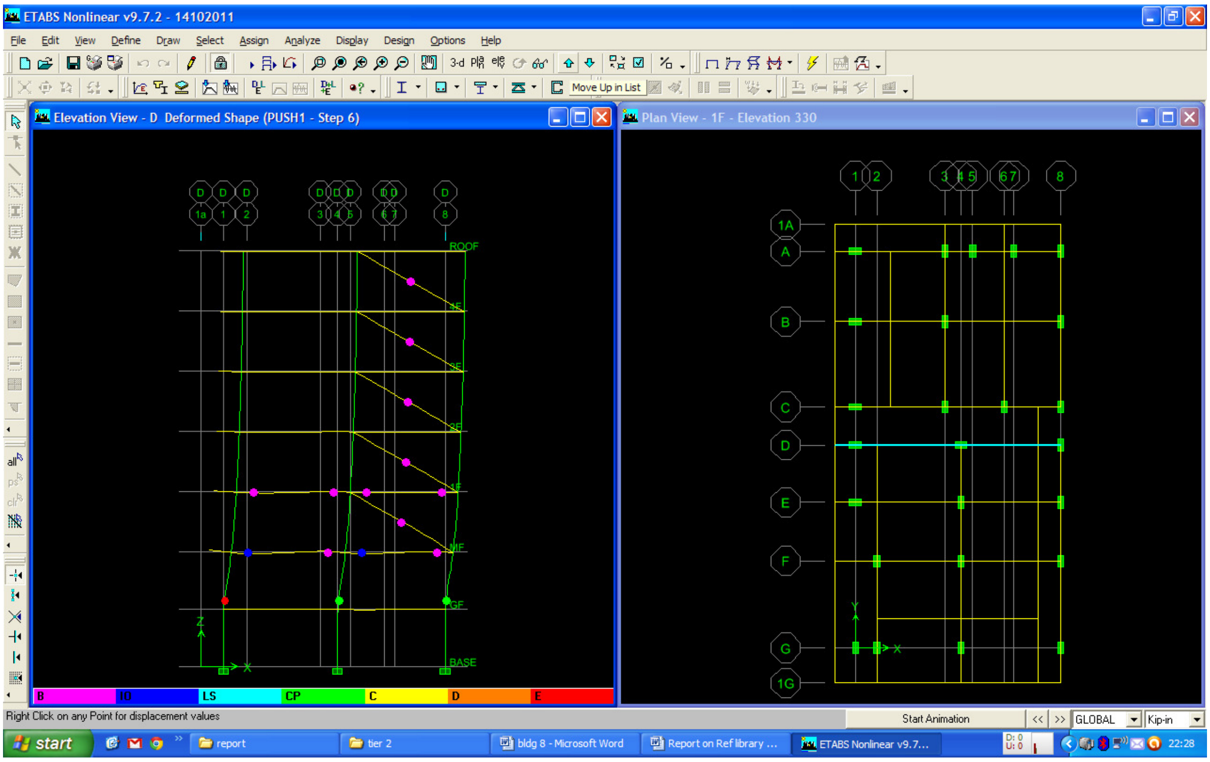
Mechanism or Deformed shapes at Performance Point - grid-A



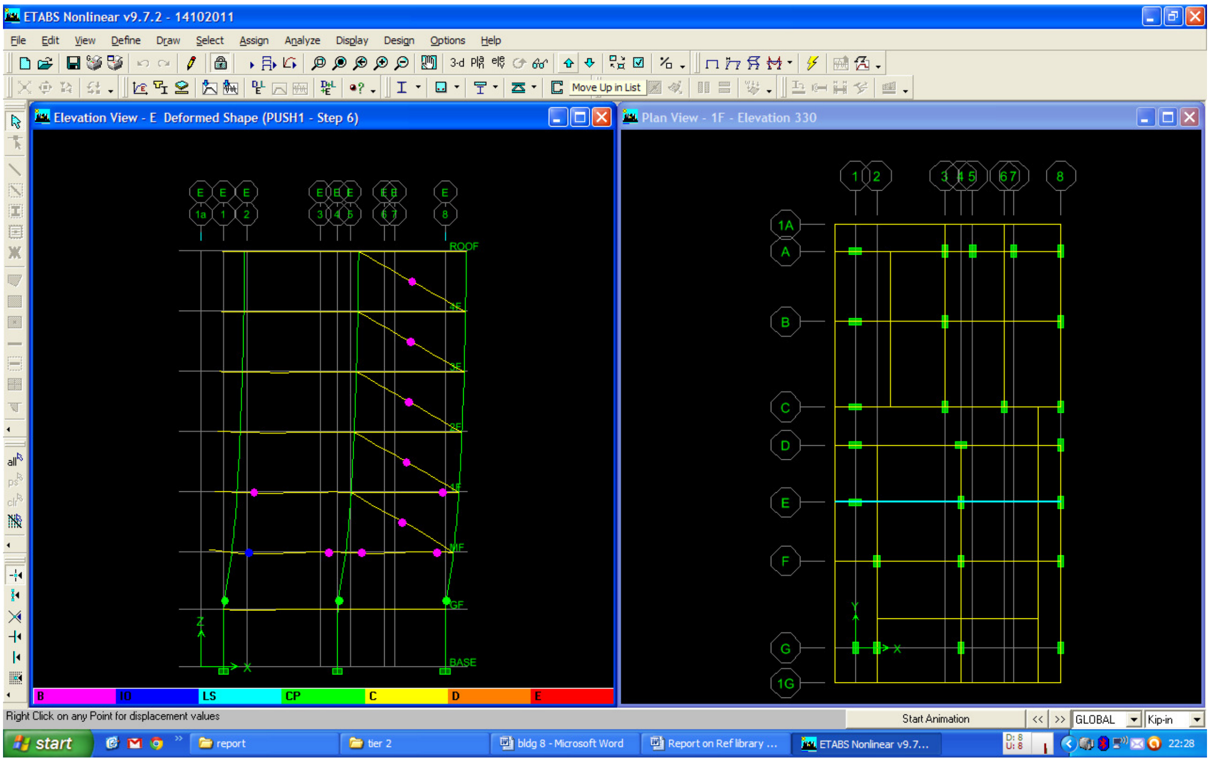
Mechanism or Deformed shapes at Performance Point - grid-B



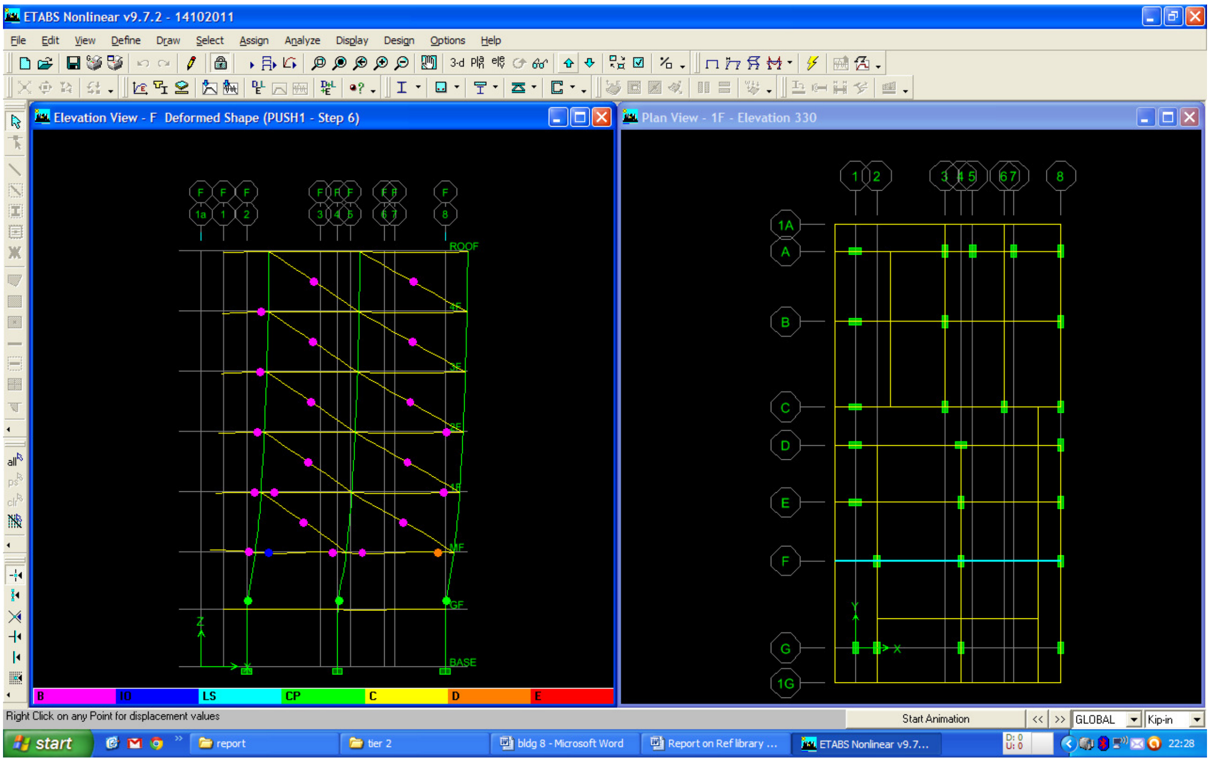
Mechanism or Deformed shapes at Performance Point - grid-C



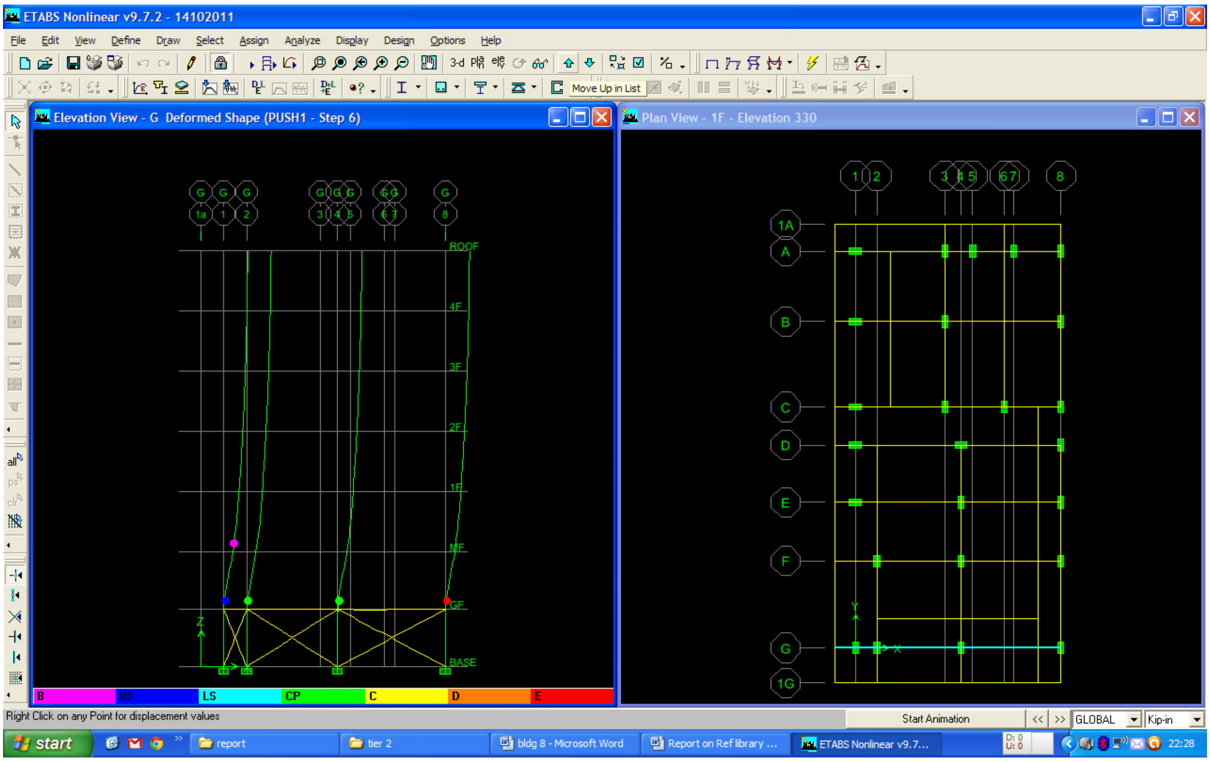
Mechanism or Deformed shapes at Performance Point - grid-D



Mechanism or Deformed shapes at Performance Point - grid-E

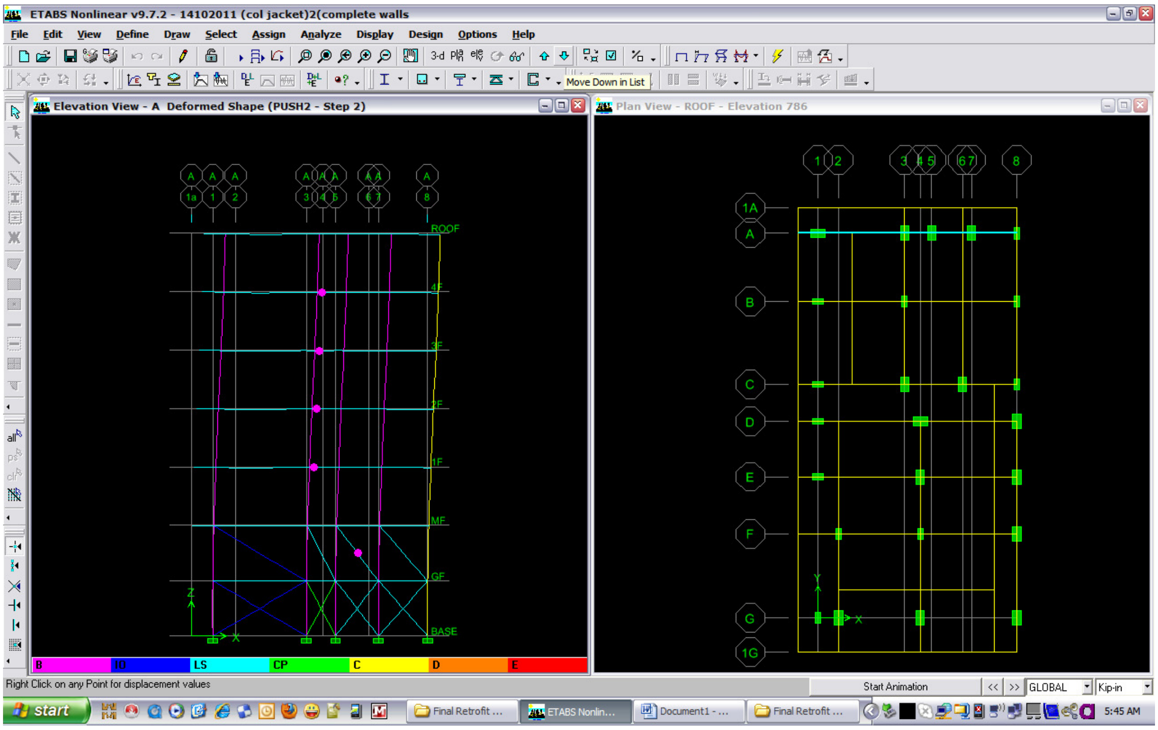


Mechanism or Deformed shapes at Performance Point - grid-F

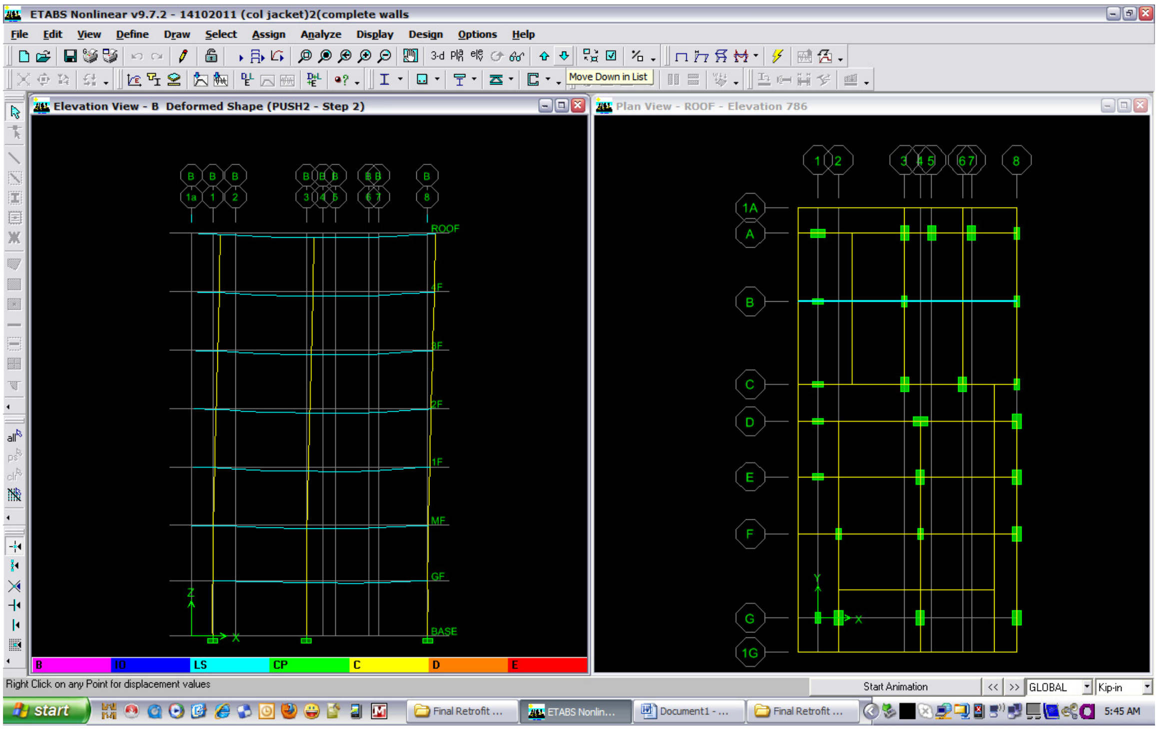


Mechanism or Deformed shapes at Performance Point - grid-G

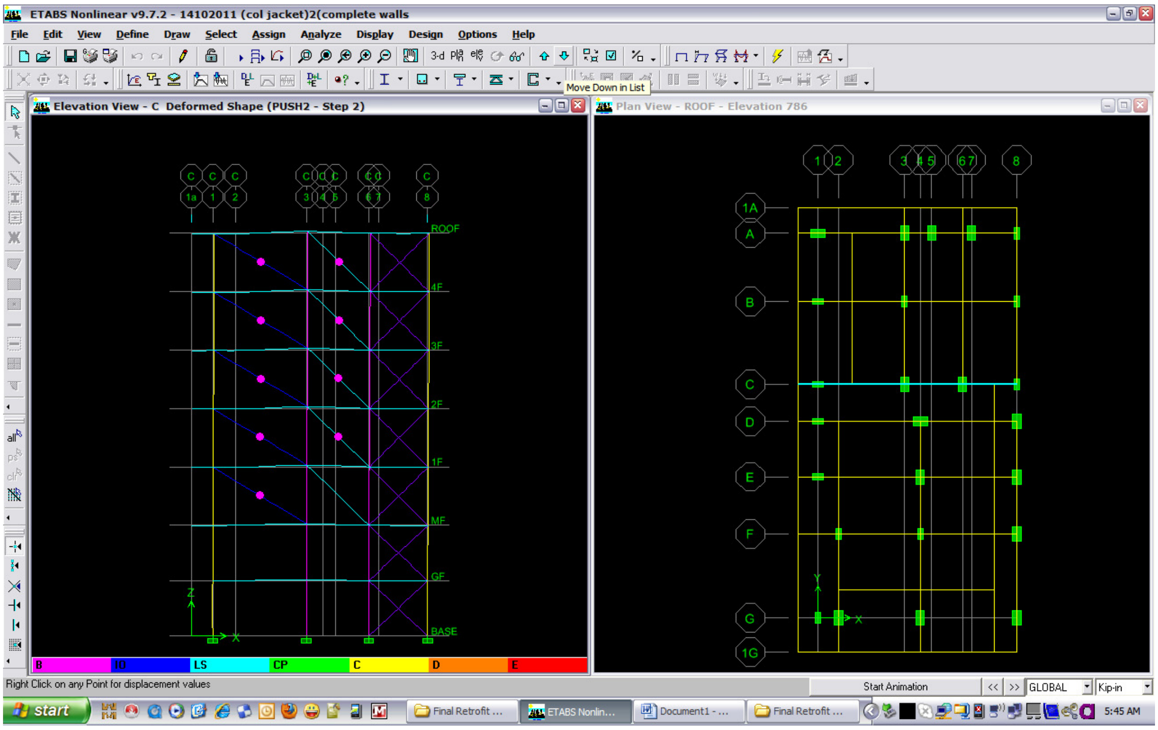
Appendix D: Non Linear Analysis (Tier 3) Results After Retrofit



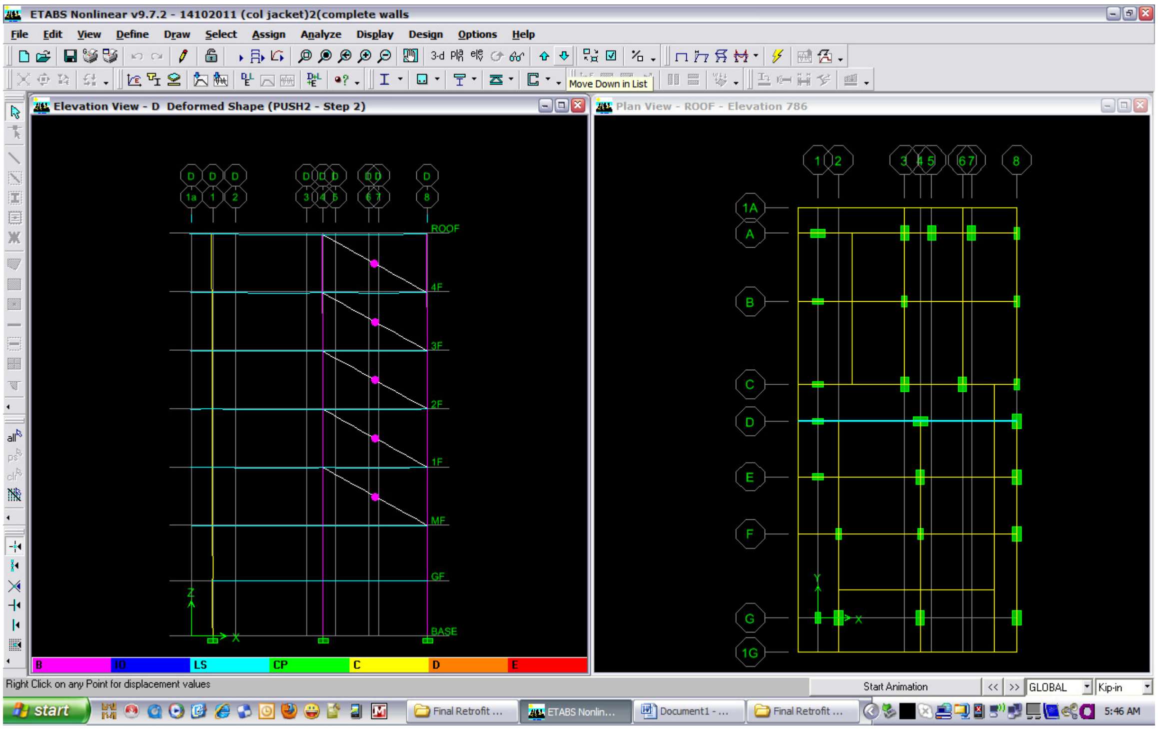
Mechanism or Deformed shapes at Performance Point - grid-A



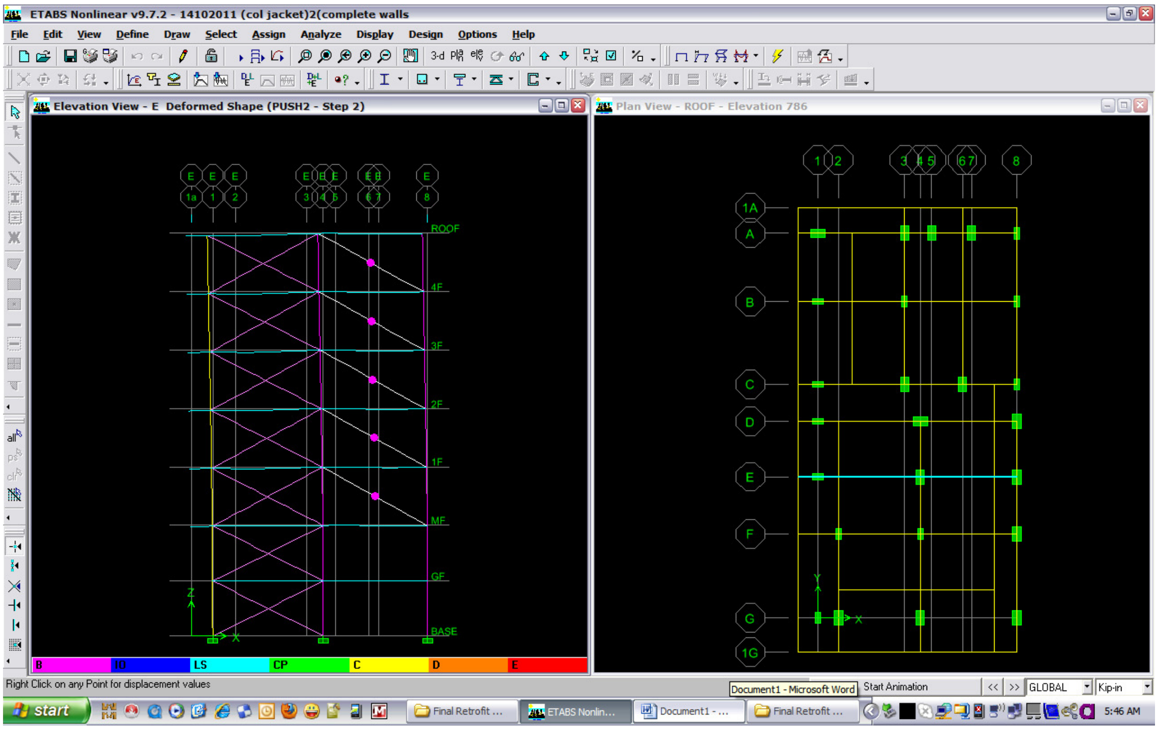
Mechanism or Deformed shapes at Performance Point - grid-B



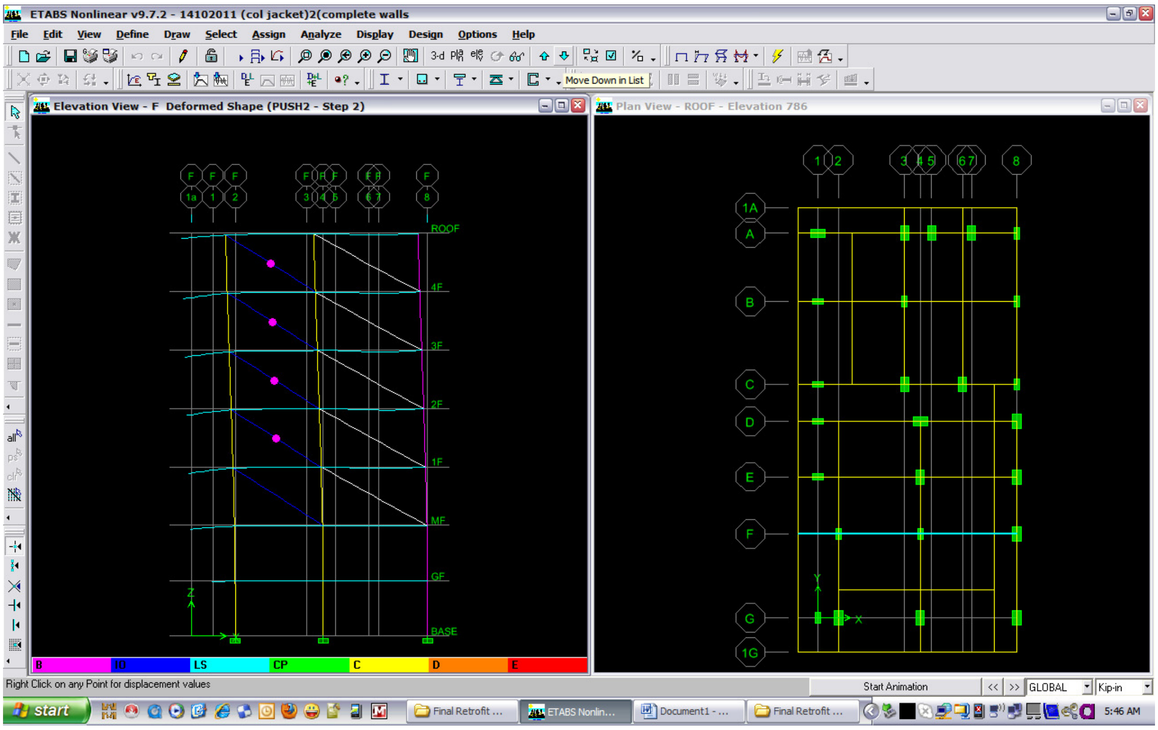
Mechanism or Deformed shapes at Performance Point - grid-C



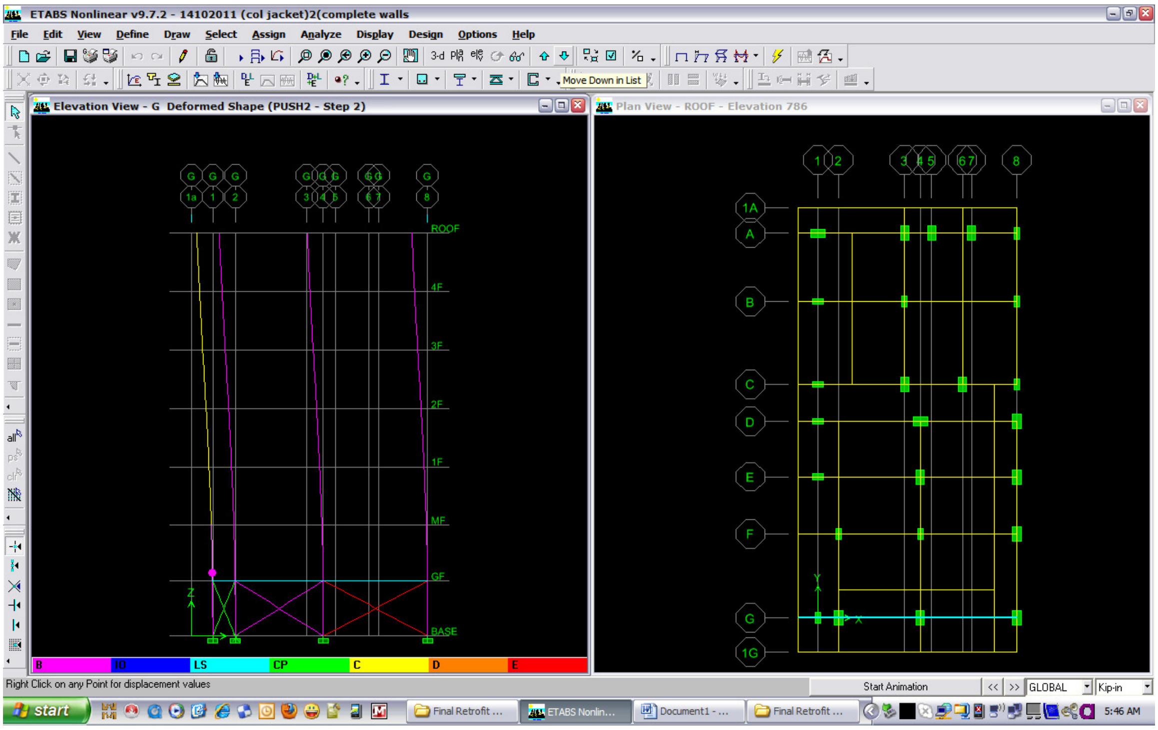
Mechanism or Deformed shapes at Performance Point - grid-D



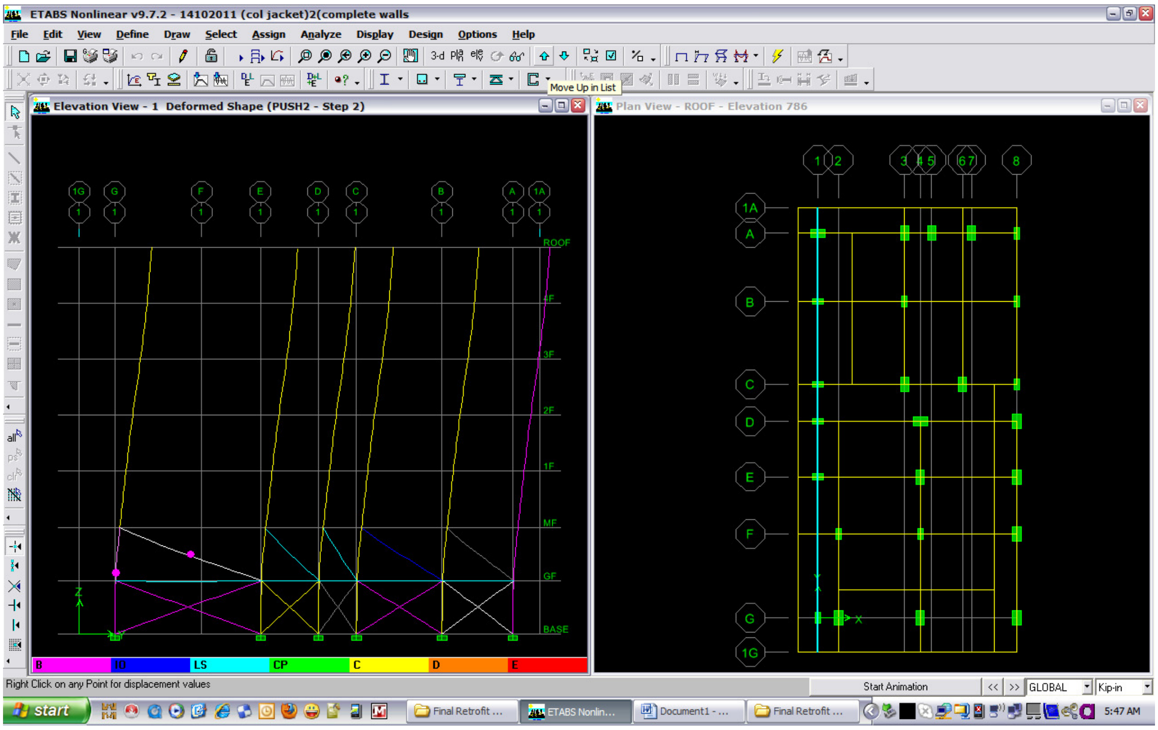
Mechanism or Deformed shapes at Performance Point - grid-E



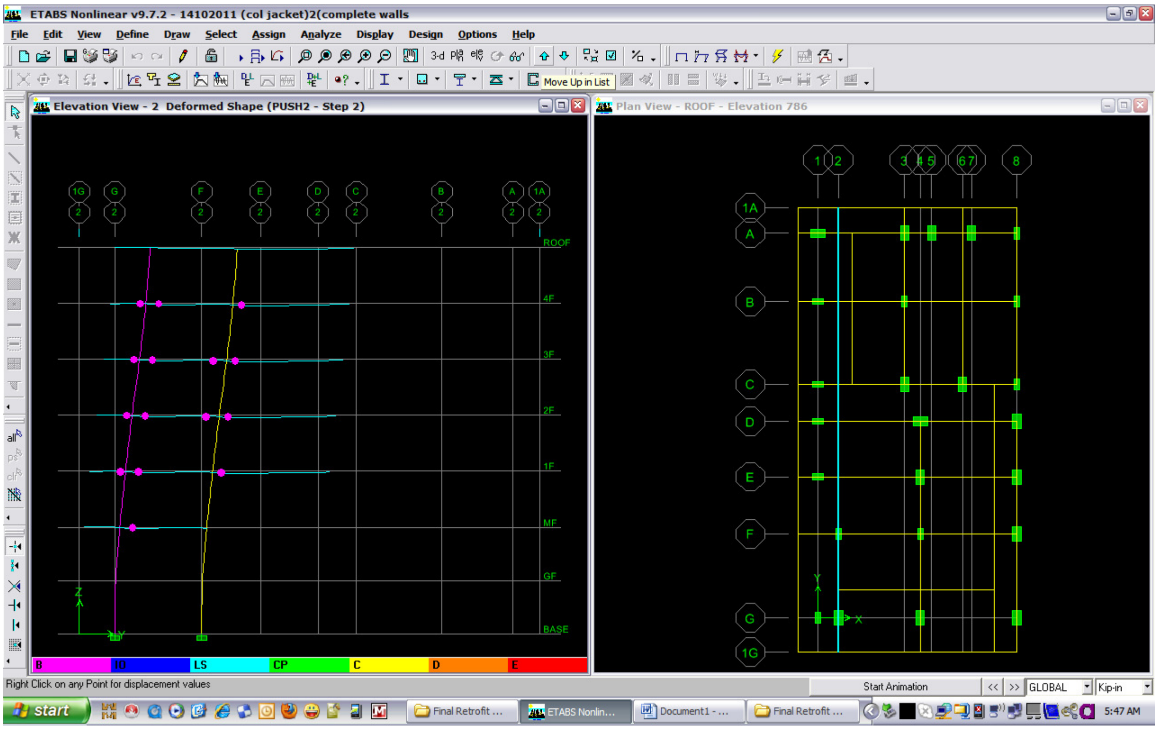
Mechanism or Deformed shapes at Performance Point - grid-F



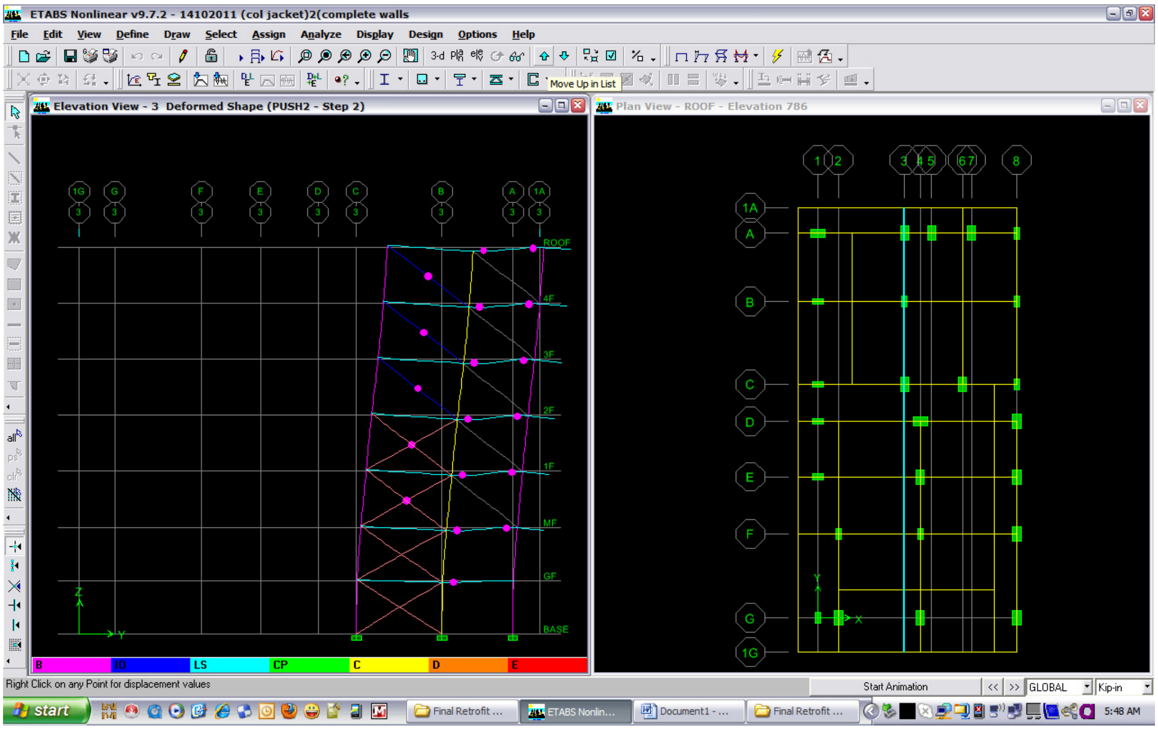
Mechanism or Deformed shapes at Performance Point - grid-G



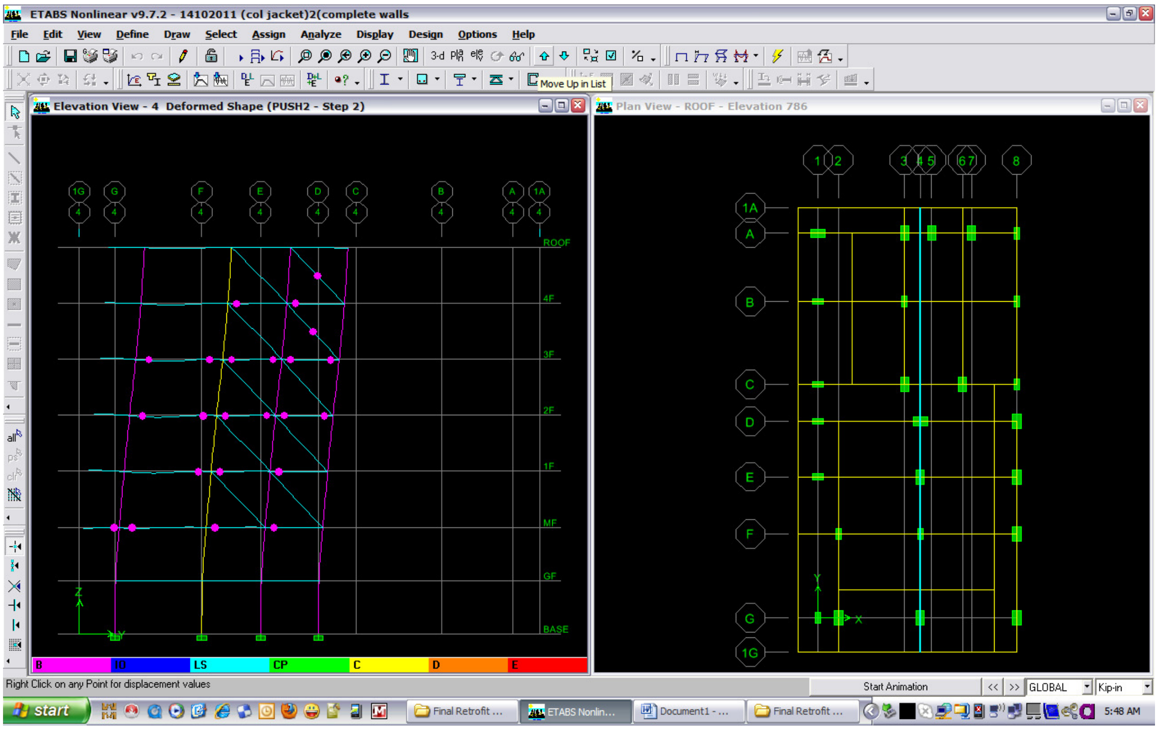
Mechanism or Deformed shapes at Performance Point - grid-1



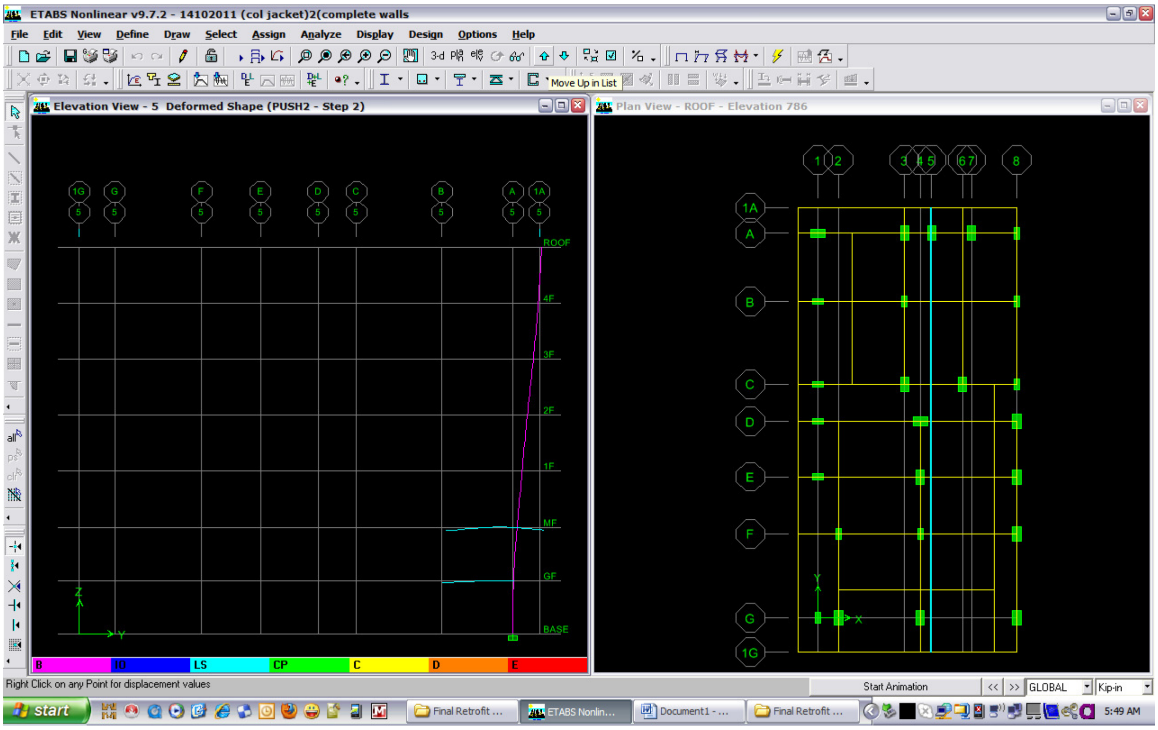
Mechanism or Deformed shapes at Performance Point - grid-2



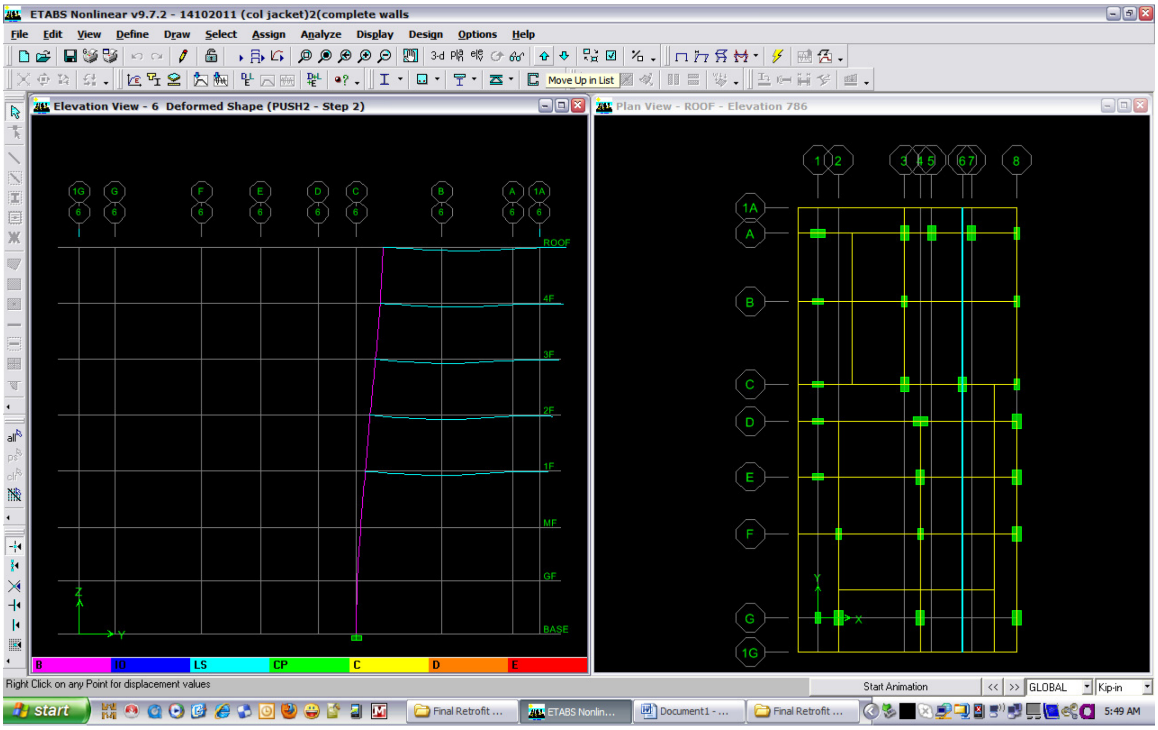
Mechanism or Deformed shapes at Performance Point - grid-3



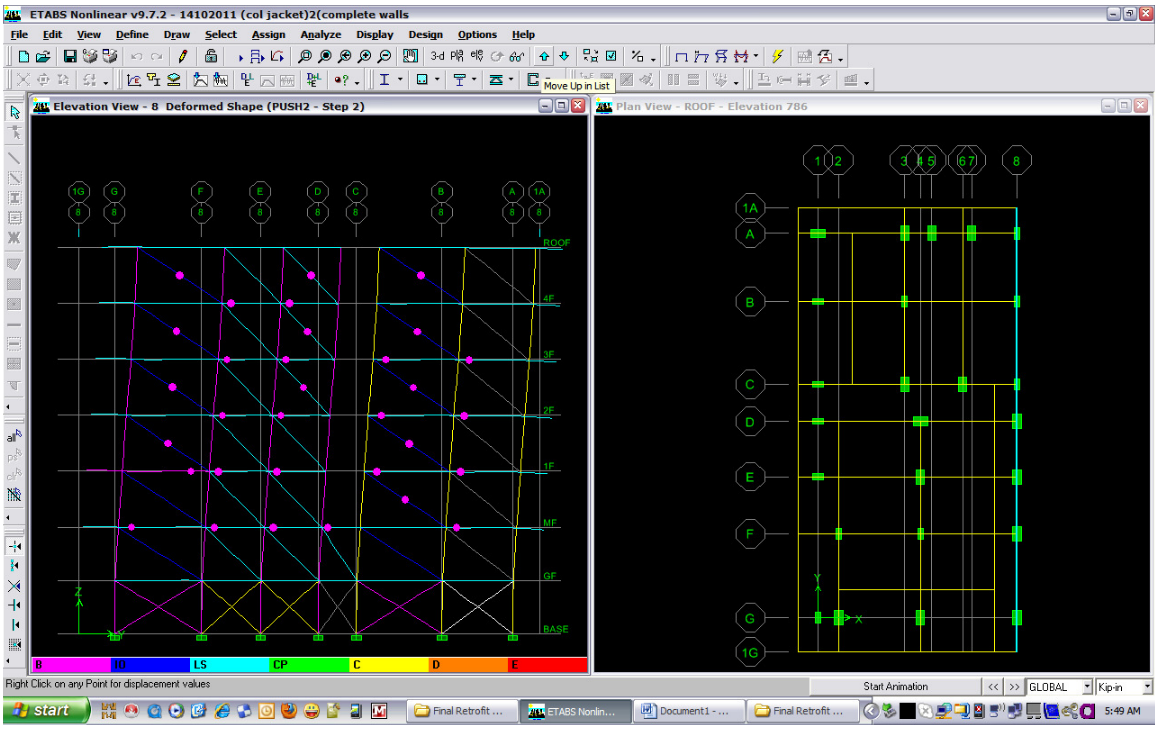
Mechanism or Deformed shapes at Performance Point - grid-4



Mechanism or Deformed shapes at Performance Point - grid-5



Mechanism or Deformed shapes at Performance Point - grid-6



Mechanism or Deformed shapes at Performance Point - grid-8

Appendix E: Retrofit Drawings