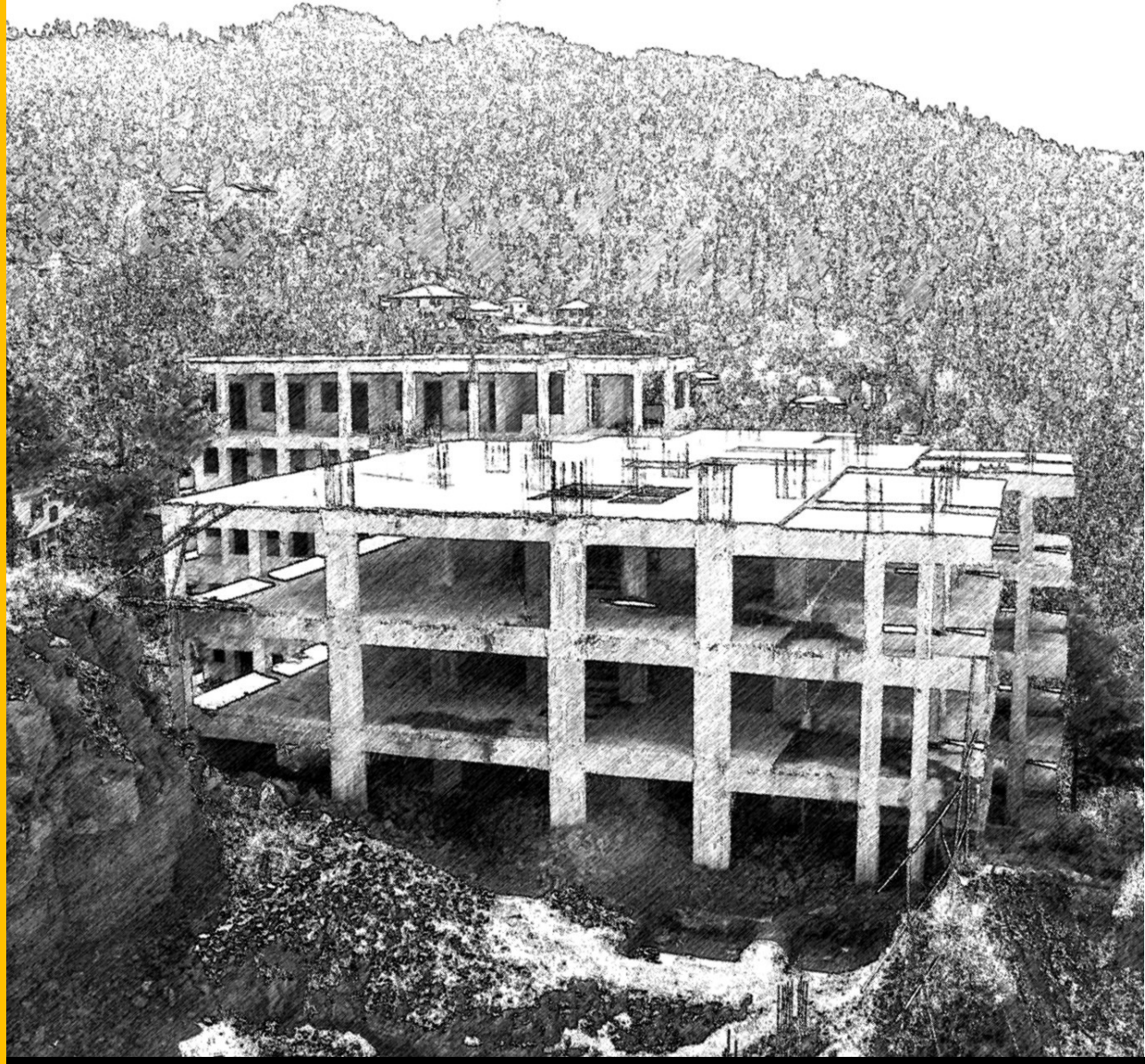

Five Storey Residential Apartment Building Near Murree

A Case Study of Seismic Assessment

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 case study building is located near Murree, a popular hill station and a summer resort for people, especially for the residents of Rawalpindi/Islamabad. The building is a reinforced concrete framed structure with five storeys including the ground floor. Car parking is located at the ground floor while the above floors have residential apartments. The building was constructed after the 2005 Kashmir Earthquake. This building was selected as a case study because it has several seismic vulnerabilities common to mixed-use residential buildings in northern Pakistan. The building was designed for a lower level of seismic forces than those prescribed in the newest edition of the building code – it was designed for Zone 2B, but with the approval of the Building Code of Pakistan (Seismic Provisions-2007), Murree is now in Zone 3. With the new zoning comes more stringent requirements for the structural detailing of the reinforced concrete frame, so the building must now be considered as an ordinary moment frame rather than an intermediate moment frame, meaning the design forces will be higher. The building also has a weak story created by open space at the ground floor, has an L-shaped plan, and has with stiff unreinforced masonry infill walls that were not considered during the structural design of the building.

The case study building was assessed for potential seismic vulnerabilities using the US Federal Emergency Management Agency (FEMA) Pre-standard 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. Structural analysis showed that the building is anticipated to protect the lives of its occupants in the design earthquake, and was therefore adequately designed to meet the performance expected of residential buildings.

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 Ahmed Khan, Associate Professor, Department of Civil Engineering, NED University of Engineering and Technology, and Dr. Janise Rodgers, Project Manager, GeoHazards International.

This case study building was investigated by Dr. Rashid Ahmed Khan, Associate Professor, Department of Civil Engineering, NED University of Engineering and Technology; Mr. Shaukat Quadeer, Chief Engineer, Structural Engineering Division, NESPAK; and Ms. Syeda Saria Bukhary, Assistant Professor, 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 building was used as an example building during a workshop that NED University of Engineering and Technology faculty conducted on building vulnerability assessment. The project team then developed it into a full case study. During the workshop, participants performed a Tier 1 vulnerability assessment exercise in which they completed checklist assessments for the building, which provided them 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, the case study team conducted a Tier 2 (linear static structural analysis) to assess the vulnerabilities in more detail, and analyzed the building using a 3-D model to better understand the effects of the plan irregularities. The detailed evaluation provided hands-on practice using structural analysis software ETABS and better understanding of the ASCE/SEI 31-03 and FEMA documents.

Building Information

Figure 1 shows the five storey apartment building under construction (ground plus four). Car parking is located at the ground floor while the above floors are residential apartments. The building's overall dimensions are 81'-0" by 102'-0", and it is approximately 50 feet tall. The building has a reinforced concrete moment frame structural system with unreinforced concrete block infill walls. The concrete block infill walls are 9" thick. The foundations are reinforced concrete spread footings. The building is relatively new built therefore; no condition assessment or repairs are needed.

Figure 1. The building during construction

The building's architectural and structural drawings are shown in Figure 2 through Figure 6. Original design calculations could not be acquired but the investigator was informed that the frame elements

were designed according to ACI-99 and earthquake analysis was carried out for Zone 2B using the 1997 Uniform Building Code (UBC-97).

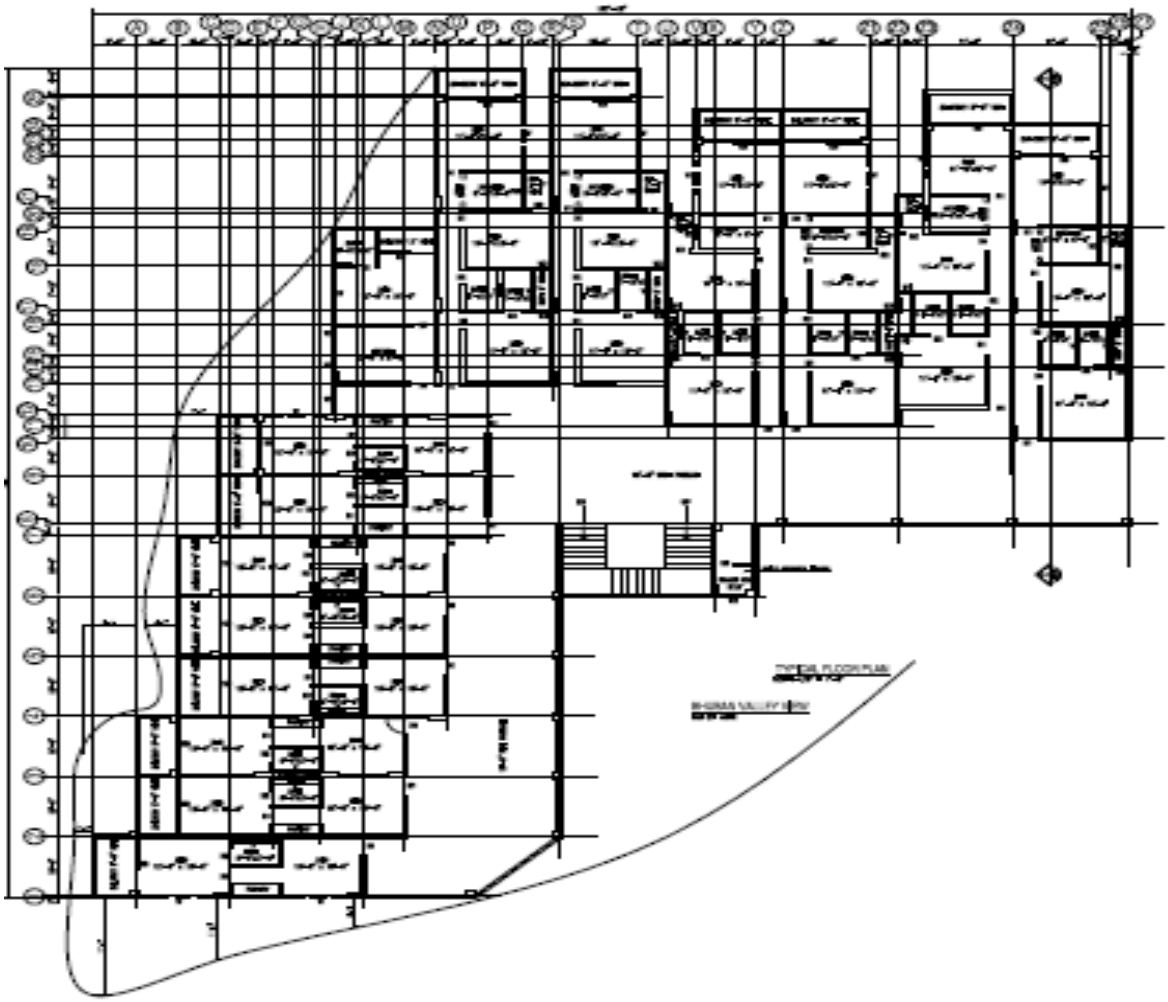


Figure 2. Typical architectural floor plan

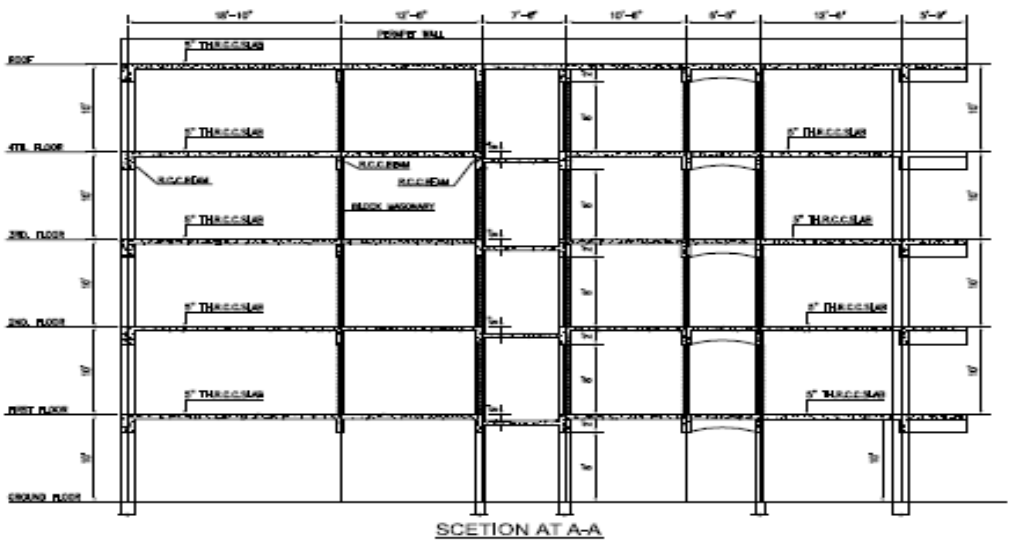


Figure 3. Architectural section view of the building

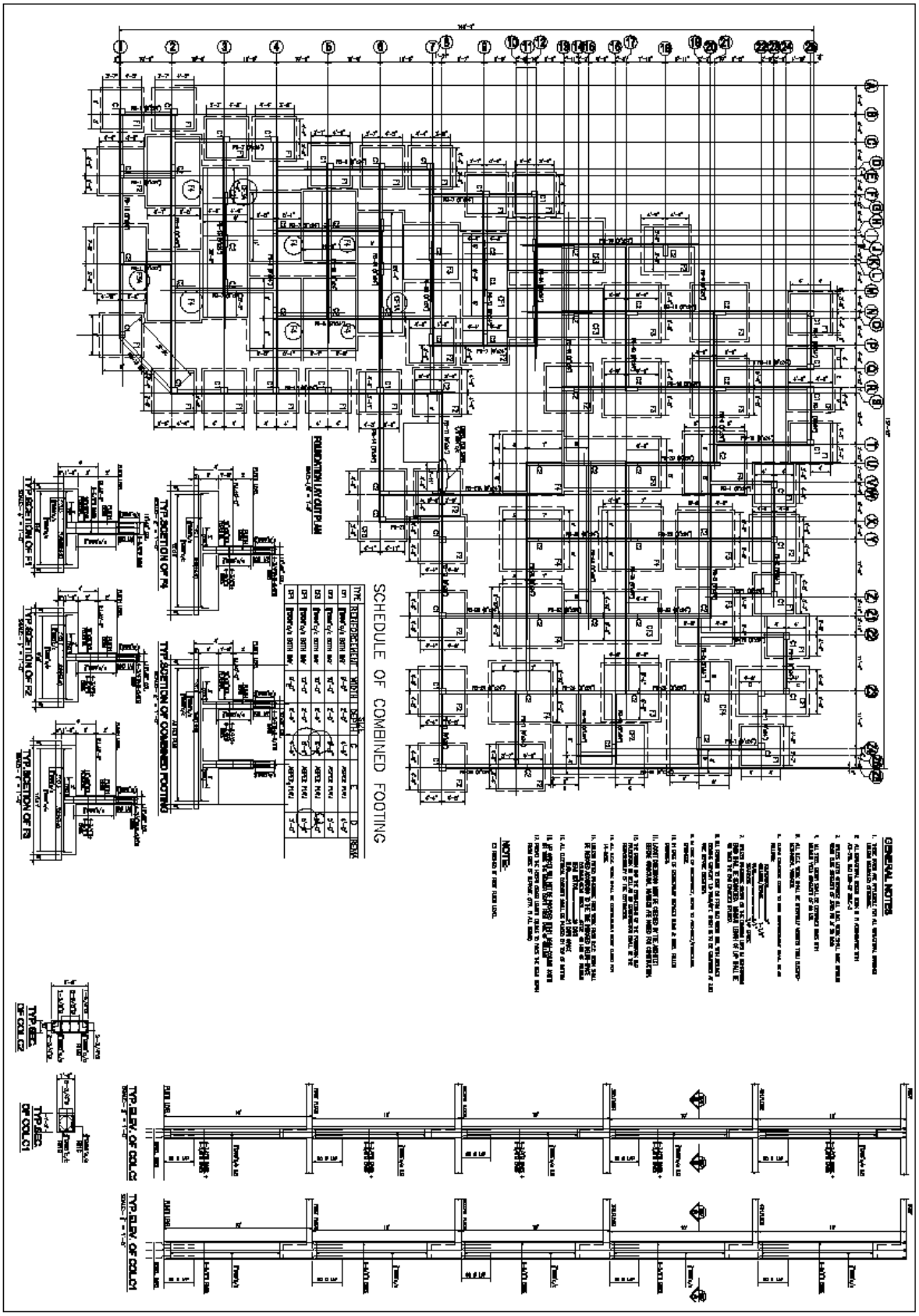


Figure 4. Structural drawings for foundation and plinth level

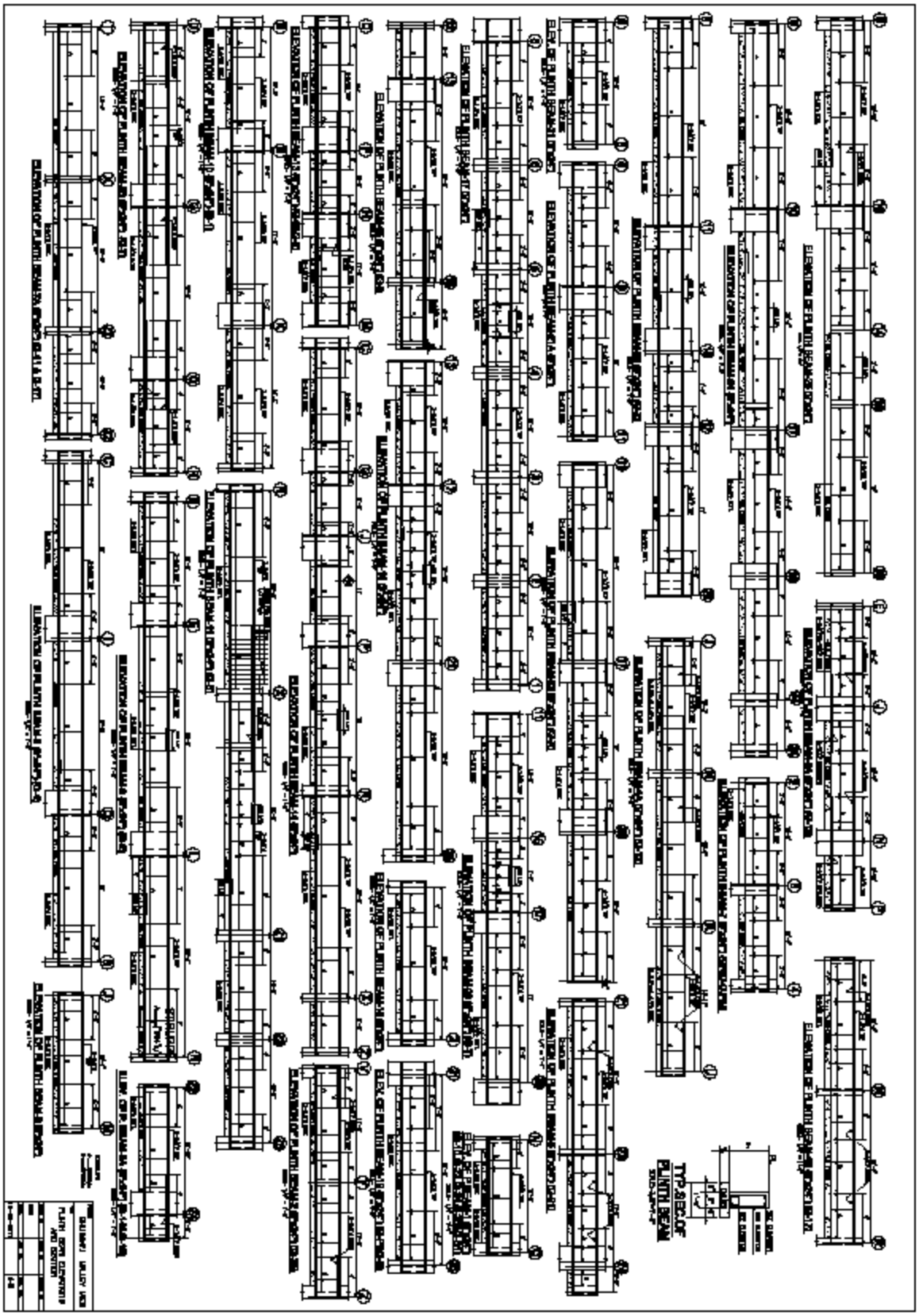


Figure 5. RCC beam elevations for plinth level

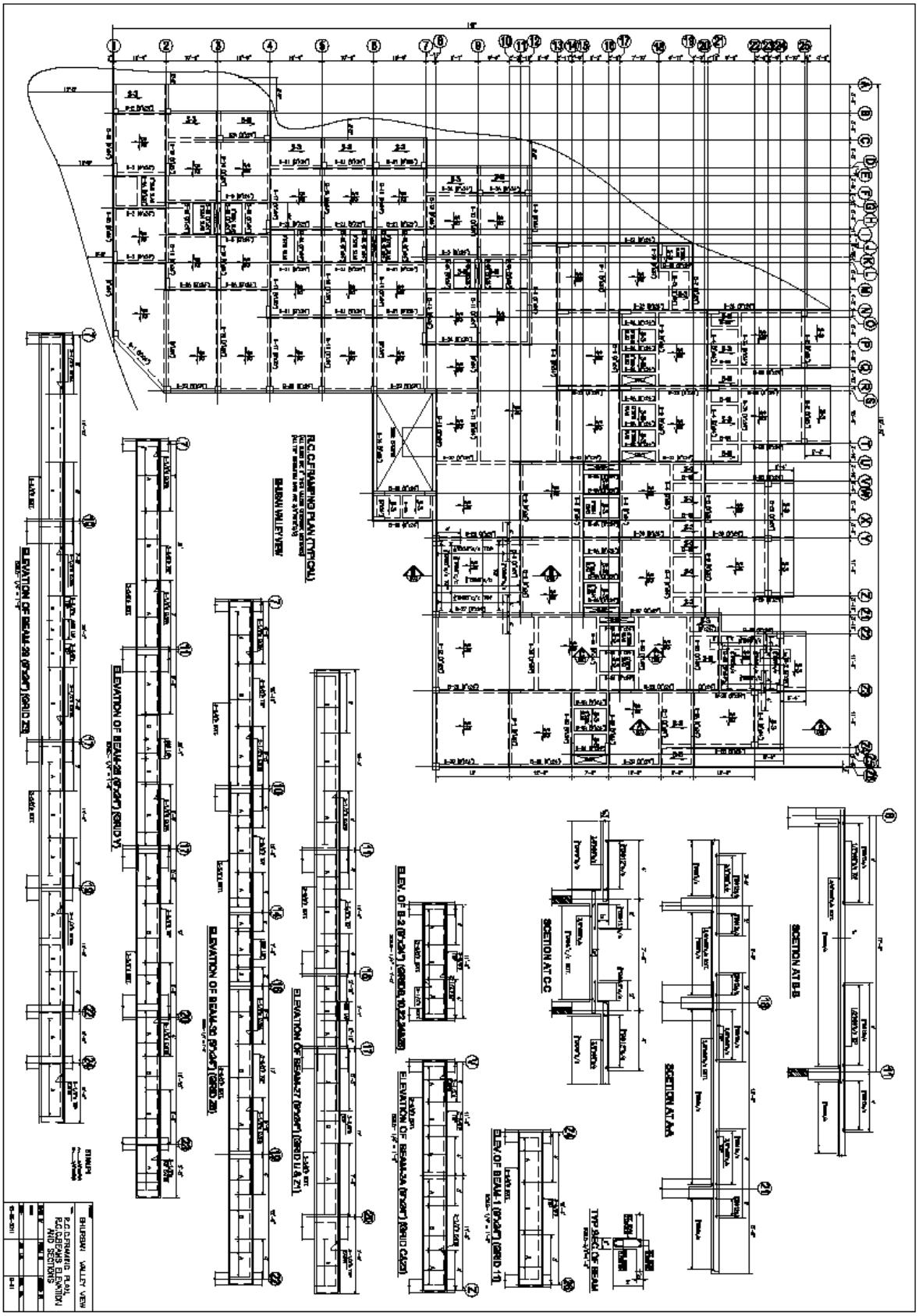


Figure 6. Structural framing for typical residential floor and RCC beam elevations

Site Information

Soil profile is taken as Rock (S_B), as the building is located in a hilly area having rocks and very dense 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 2.5 tons per square foot (tsf).

Hazard Information

The National Building Code of Pakistan places Murree in a Seismic Zone 2B (0.16g to 0.24g). However, there is currently uncertainty regarding the severity of the city's seismic hazard. For this reason, the building is being evaluated for Zone 3 of the 1997 Uniform Building Code with seismic coefficients Ca=0.3, Cv=0.3. The site is not located near any known active faults so near-source factors are not applicable.

Initial and Linear Evaluations of Existing Building

Checklist-based Evaluation

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

Checklist	Non-compliant Items
Building System	Soft storey Weak storey Mass irregularity Torsion irregularity
Lateral Force-resisting System	Beam Bar Splices Shear stress check Column Bar Splices
Geologic Hazards and Foundation	None

Linear Evaluation

For Tier-2 Analysis, a linear static analysis was performed for the building in ETABS Nonlinear version 9.7.0. Figure 7 shows the developed 3-D model of the building. In the 3-D model of the building, the beams and columns were modeled with linear beam-column elements, and the infill walls were modeled with single linear compression struts. The results of the linear analysis showed that there were no columns with demand/capacity ratios (DCRs) greater than one, but showed two beams had DCRs higher than one due to combined shear and torsion effects. However, these two local failures will not affect building stability and the nearby beams, which are not overstressed, are able to support the slab and prevent it from collapsing. Therefore the building was accepted as adequately designed and no nonlinear static analysis was needed. Please see Appendix B for linear analysis results.

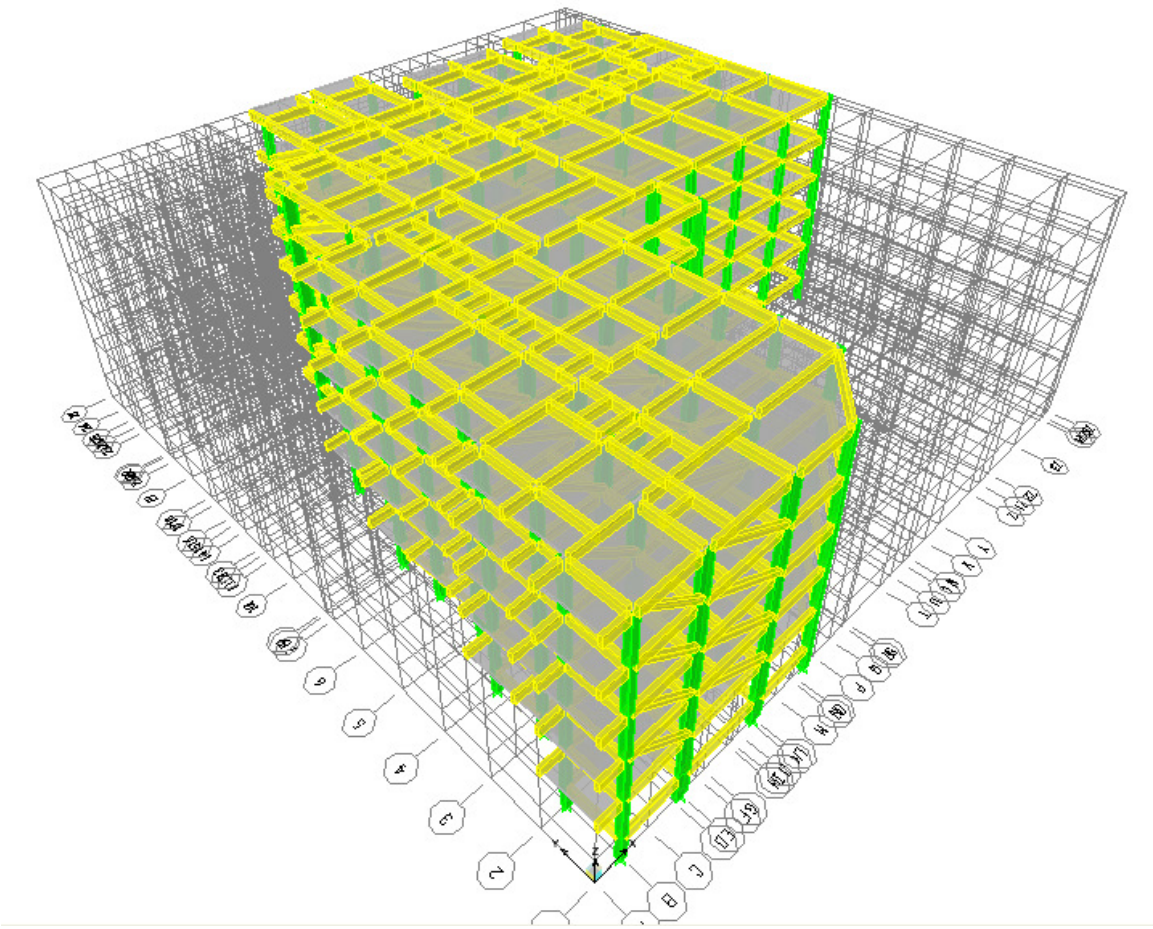


Figure 7. Rendering of linear ETABS model of the building

Other checks mandated in ASCE 31 for Tier 2 analysis based on the Tier 1 Checklist results were also carried out. 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

Shortest Direction in X-DIRECTION (ft) = 80.75
Shortest Direction in Y-DIRECTION (ft) = 102

Storey	Diaphragm	XCM (FT)	XCR (FT)	YCM (FT)	YCR (FT)	% diff in X	% diff in Y
RF	D1	66.772	72.578	77.759	64.829	**7.19**	**12.68**
FFL	D2	66.447	72.635	77.127	64.653	**7.66**	**12.23**
TF	D3	66.447	72.609	77.127	64.518	**7.63**	**12.36**
SF	D4	66.447	72.603	77.127	64.317	**7.62**	**12.56**
FF	D5	66.973	72.968	77.154	63.733	**7.42**	**13.16**
GL	D6	67.472	74.175	76.501	62.673	**8.30**	**13.56**

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

EQ X-Direction					% diff in K (< 30% allowed)	
					% Difference Compared to	
Story	Load	Storey Force (Kips)	Total Displacement (in)	Stiffness (K/in)	Above Storey	Below Storey
RF	EQX	386.2	0.1267	3048.1452	--------	35.81917972
FFL	EQX	484.79	0.1171	4139.9658	26.37269625	8.730434694
TF	EQX	380.12	0.1006	3778.5288	9.565548674	5.693735443
SF	EQX	275.45	0.0773	3563.3894	6.037494402	0.772289972
FF	EQX	169.85	0.0473	3590.9091	0.766371363	59.67380721
GL	EQX	15.06	0.0104	1448.0769	147.9777858	---------

EQ Y-Direction					% diff in K (< 30% allowed)	
					% Difference Compared to	
Story	Load	Storey Force (Kips)	Total Displacement (in)	Stiffness (K/in)	Above Storey	Below Storey
RF	EQY	386.2	0.1205	3204.9793	--------	35.2965202
FFL	EQY	484.79	0.1118	4336.2254	26.08826904	8.685943054
TF	EQY	380.12	0.096	3959.5833	9.512164222	5.610040163
SF	EQY	275.45	0.0737	3737.4491	5.943471289	0.989854985
FF	EQY	169.85	0.045	3774.4444	0.980152893	60.49513694
GL	EQY	15.06	0.0101	1491.0891	153.1333924	---------

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

Table 3. Storey drift check

EQ Forces without Eccentricities

Story	Etab Drift X	Code Modified Drift	Etab Drift Y	Code Modified Drift
	$\Delta_{S\,(FT)}$	Δ_M	$\Delta_{S\,(FT)}$	Δ_M
RF	0.00123	0.00302	0.00105	0.00257
FFL	0.00224	0.00548	0.00191	0.00468
TF	0.00316	0.00774	0.00269	0.00660
SF	0.00406	0.00994	0.00343	0.00841
FF	0.00491	0.01202	0.00414	0.01014
GL	0.00221	0.00542	0.00190	0.00466

$\Delta_{M\,(FT)}$ =	0.7XRXDrifts from ETABS	R =	3.5
$\Delta_{Sallowed\,(FT)}$ =	0.02		

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 the results of linear static analysis, as shown in Appendix B, the building response is not expected to go into the nonlinear range, furthermore the checks for building system (mass irregularities, torsion etc.) in Tier 1 analysis which were assumed non-compliant through visual inspection, were found to be compliant after Tier 2 analysis. The building has satisfactorily passed the Tier 2 analysis. Hence there is no need to perform nonlinear static analysis.

Results Summary

- Tier 1 shows some vulnerabilities but linear elastic analysis shows the building to be stable and adequately designed.
- Tier 2 check shows that there is a possibility for soft story at ground and roof stories but the drifts are low. Differences in stiffness are due to differences in infill wall distribution.
- Tier 2 results for force demand capacity ratios (DCRs) for columns shows that all columns have DCRs less than one.
- Tier 2 results show that torsion irregularity check is less than 20% so there does not seem to be a problem even though the building is L-shaped.
- Two beams fail in combined shear and torsion check; however no retrofitting may be needed because the nearby beams are able to support and prevent collapse of the slab. Also, beam retrofits could be invasive and therefore costly, especially in a residential building like this.
- Joints have no reinforcing - column ties and beam ties are closely spaced at ends but do not continue through the joint. However, joint shear strength is adequate for the demand.
- Because the building was built after the 2005 earthquake, some seismic design requirements were followed. Perhaps this explains some of the better behavior of the building in Tier-2 Analysis.

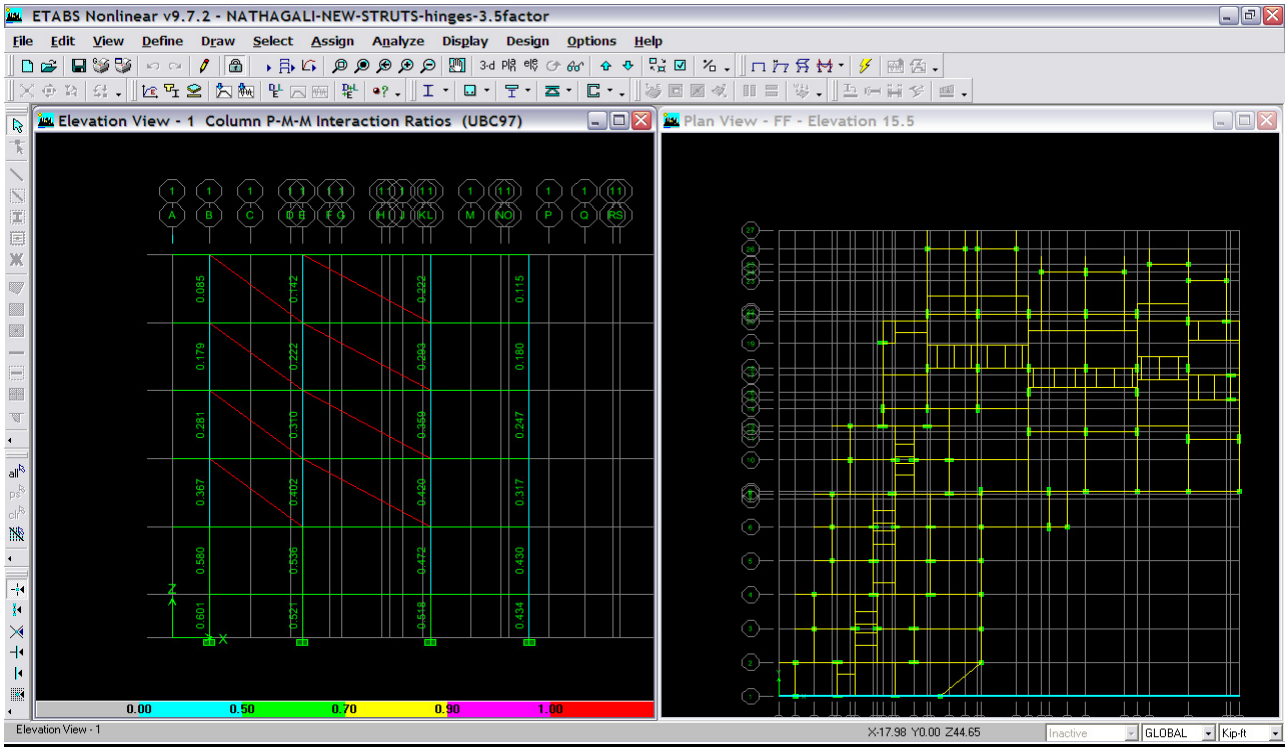
Appendix A: Tier 1 Checklists

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

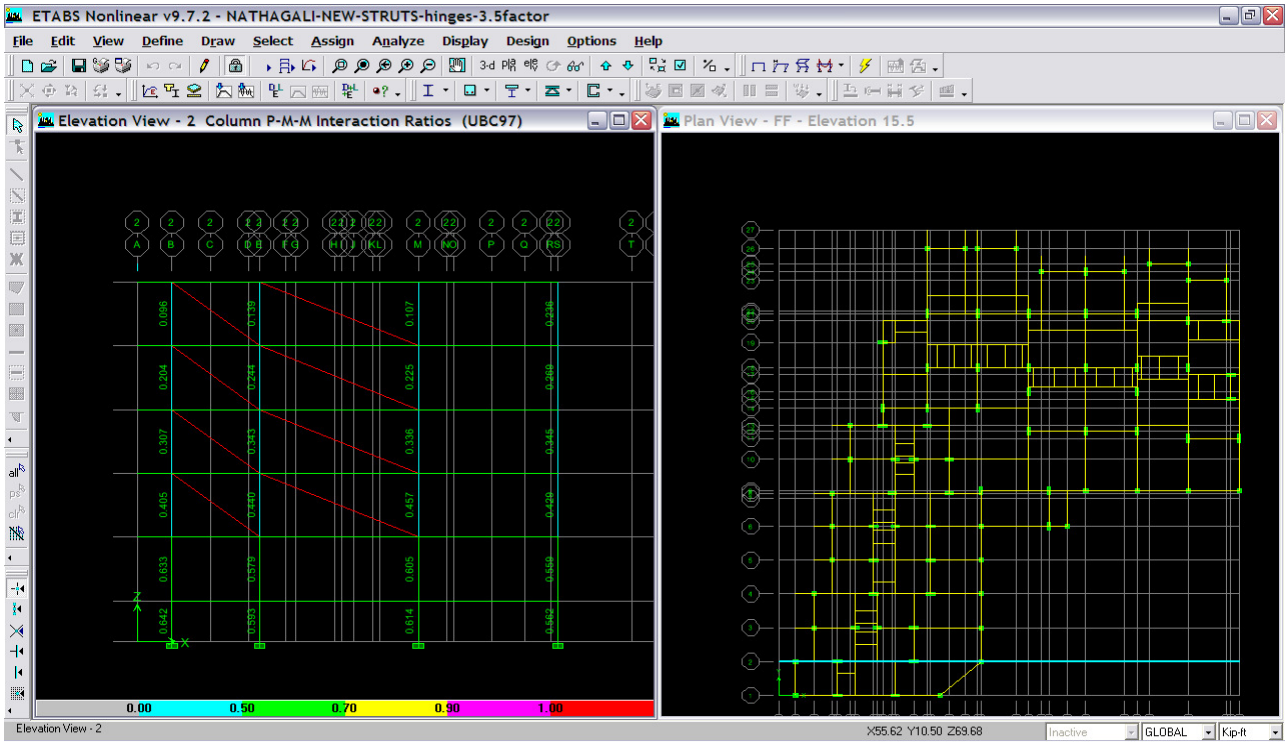
GEOLOGIC SITE HAZARDS AND FOUNDATION CHECKLIST	
Liquefaction	C
Slope Failure	C
Surface Fault rupture	C
Foundation Performance	C
Deterioration	C
Pole Foundation	NA
Over turning	C
Ties between Foundation element	NA
Deep foundation	NA
Sloping Sites	C

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

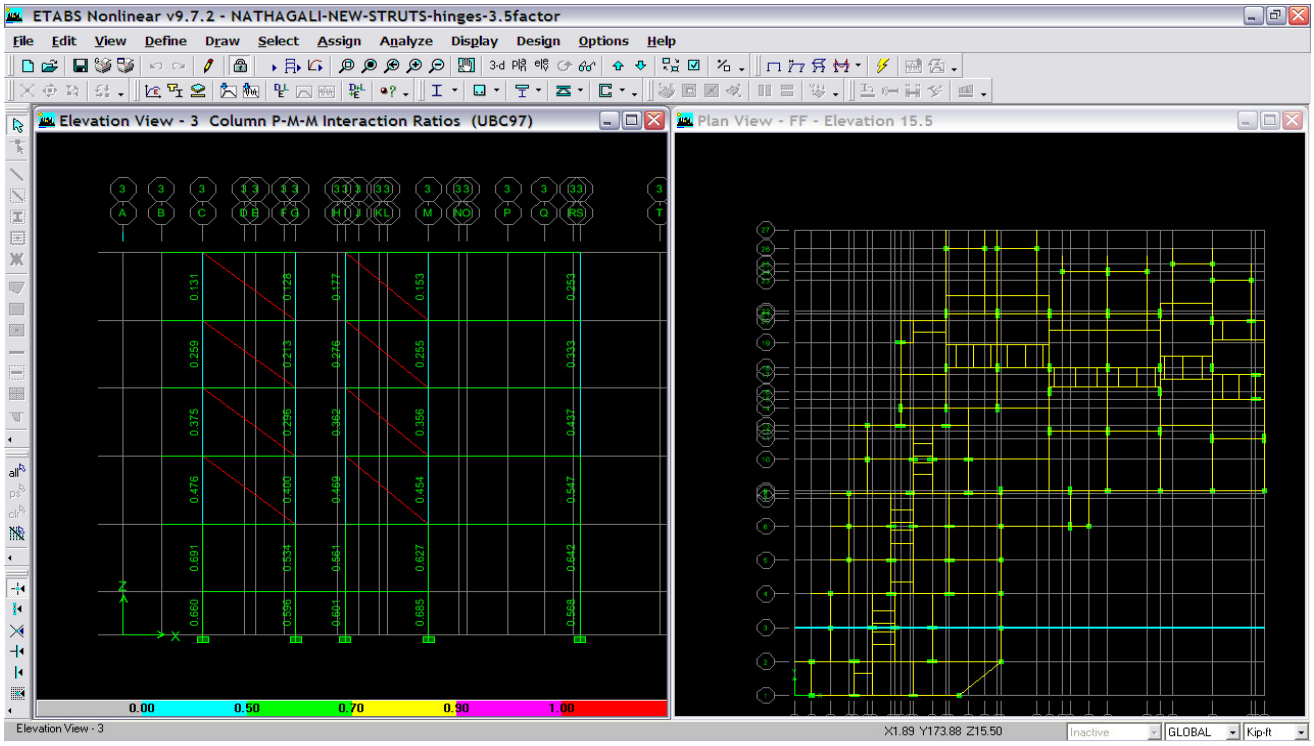
Appendix B: Linear Analysis (Tier 2) Results



Demand/Capacity Ratios for Frame at Grid-1



Demand/Capacity Ratios for Frame at Grid-2



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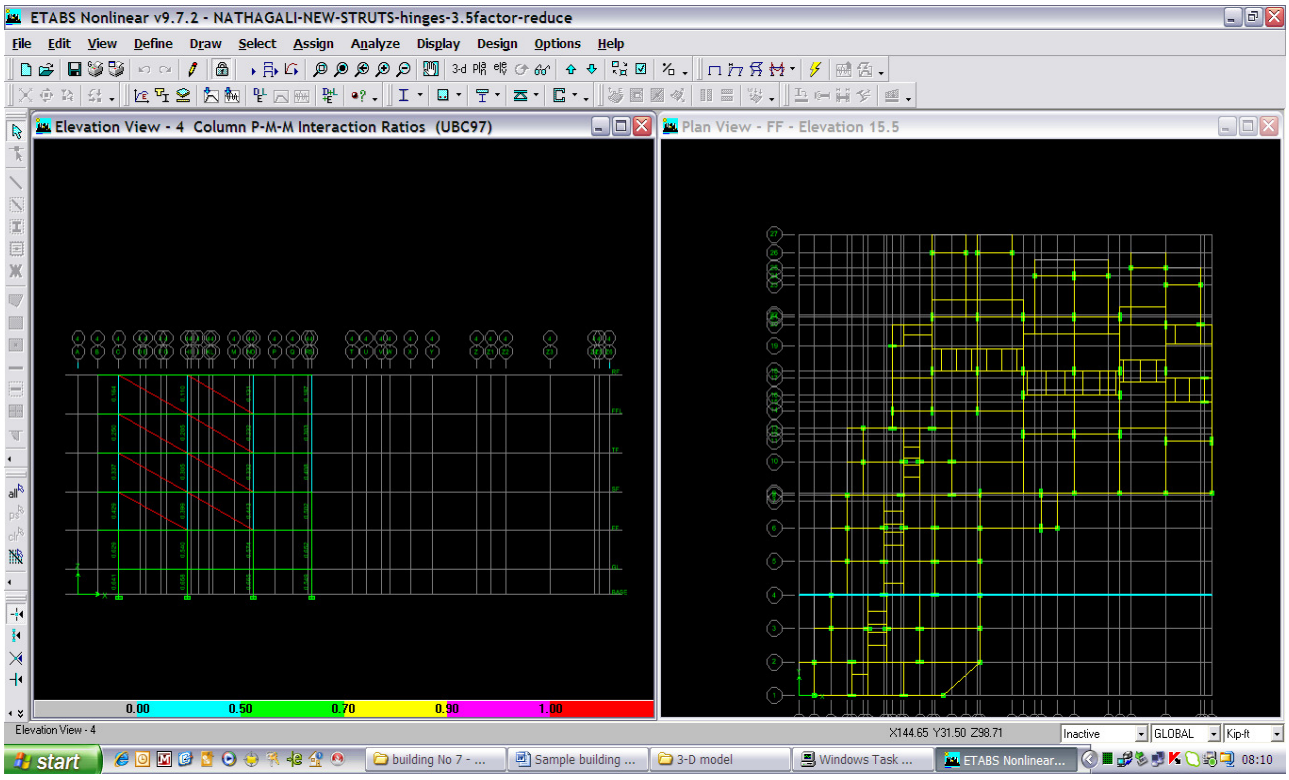

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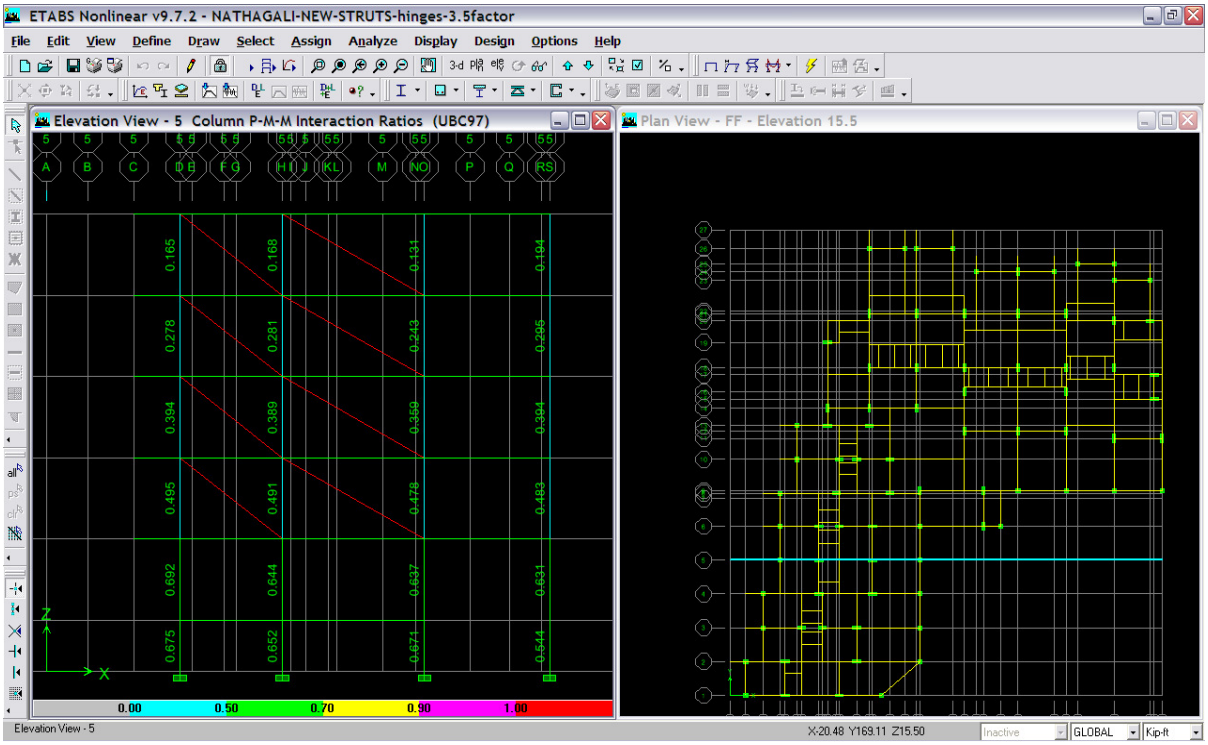
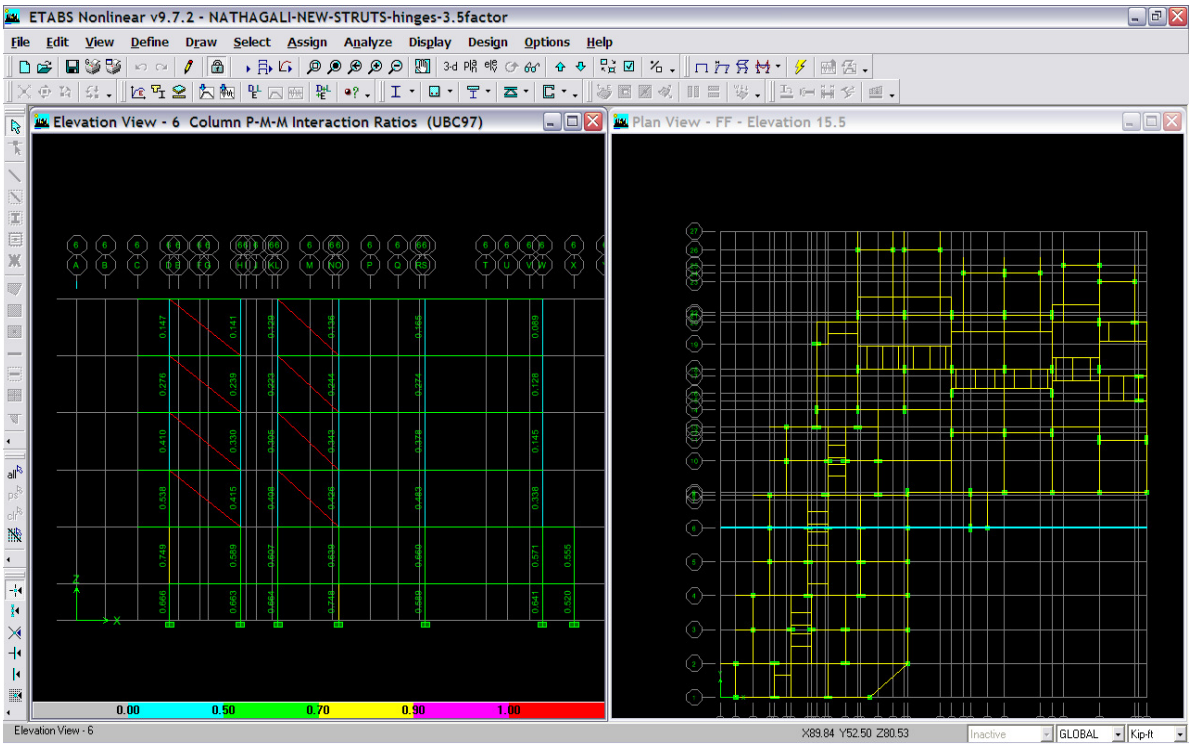


Demand/Capacity Ratios for Frame at Grid-5



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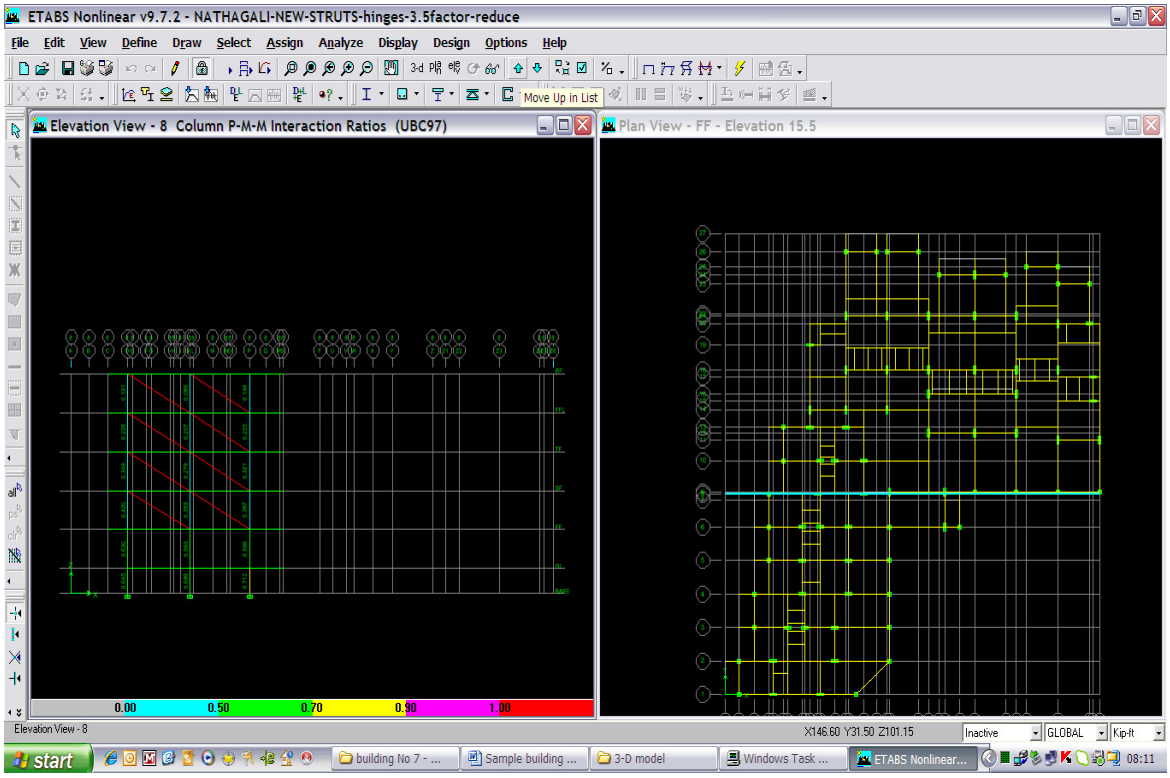

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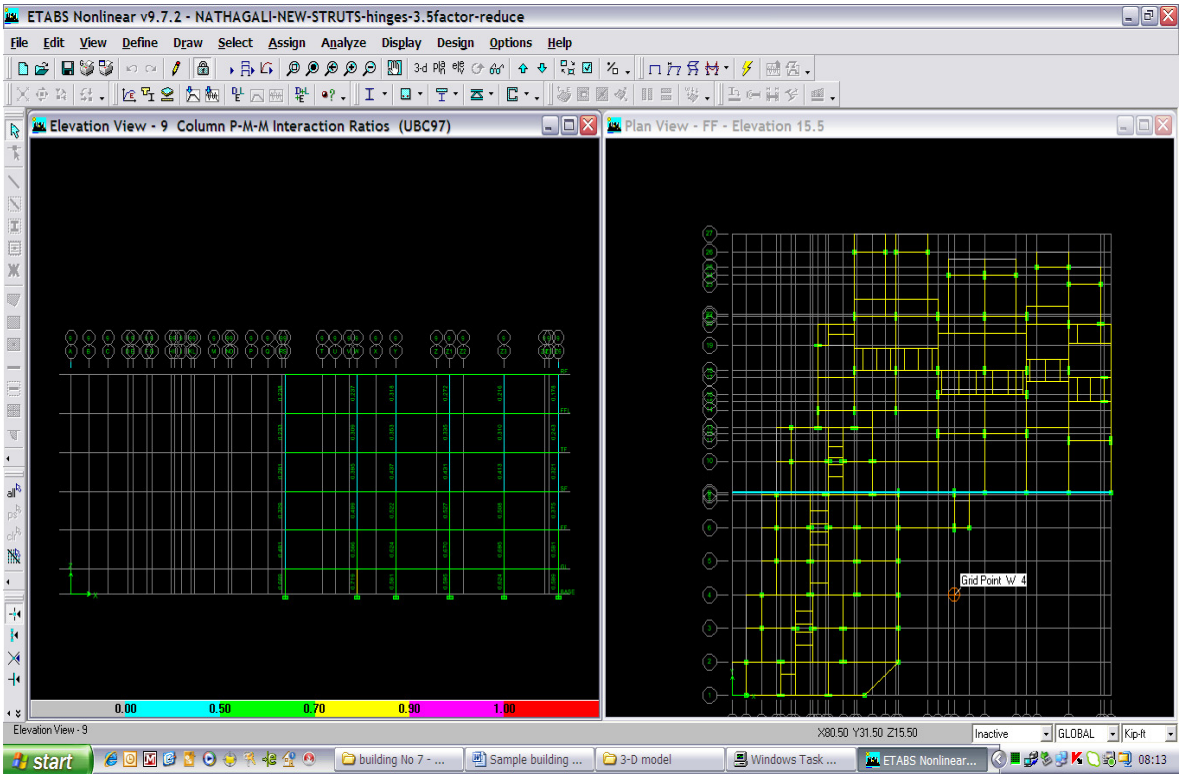

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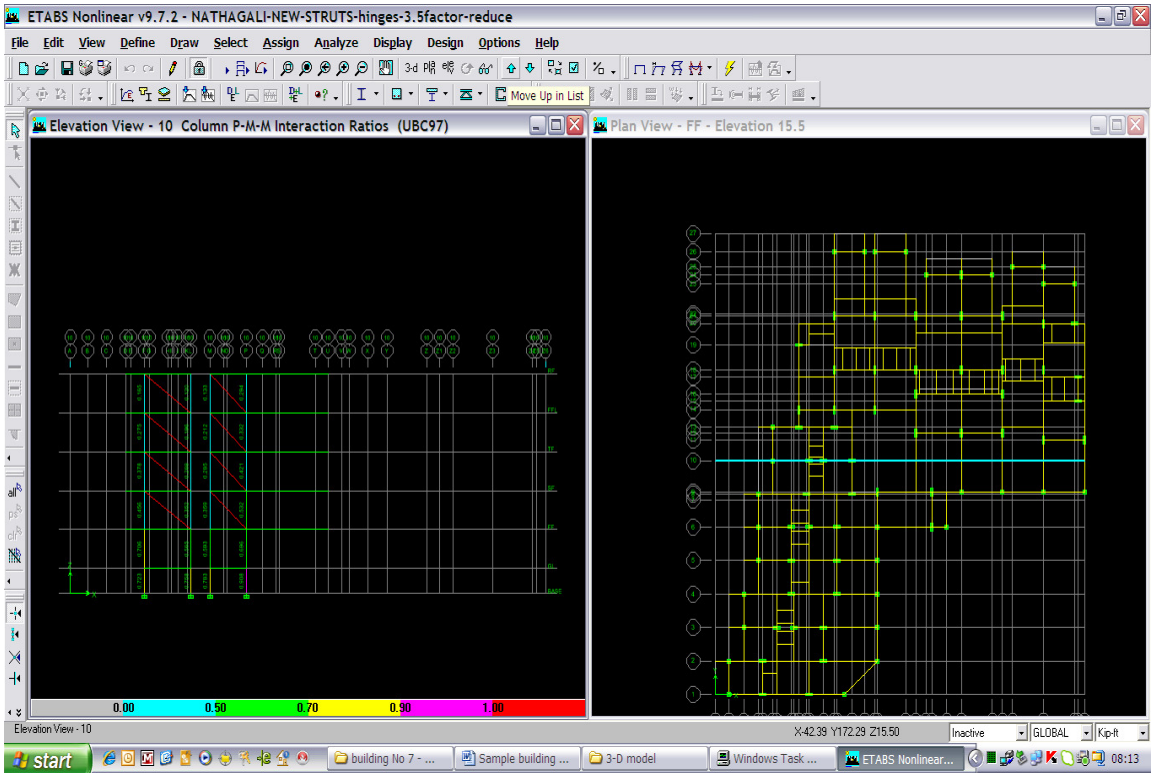

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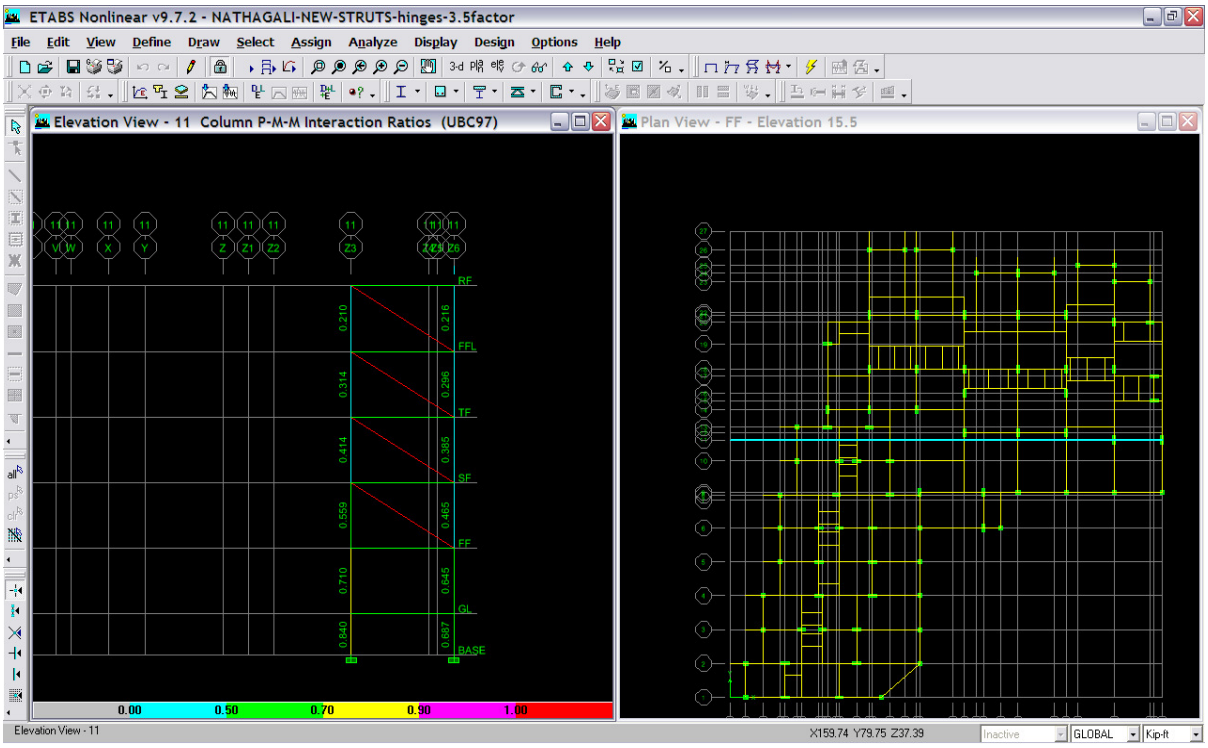

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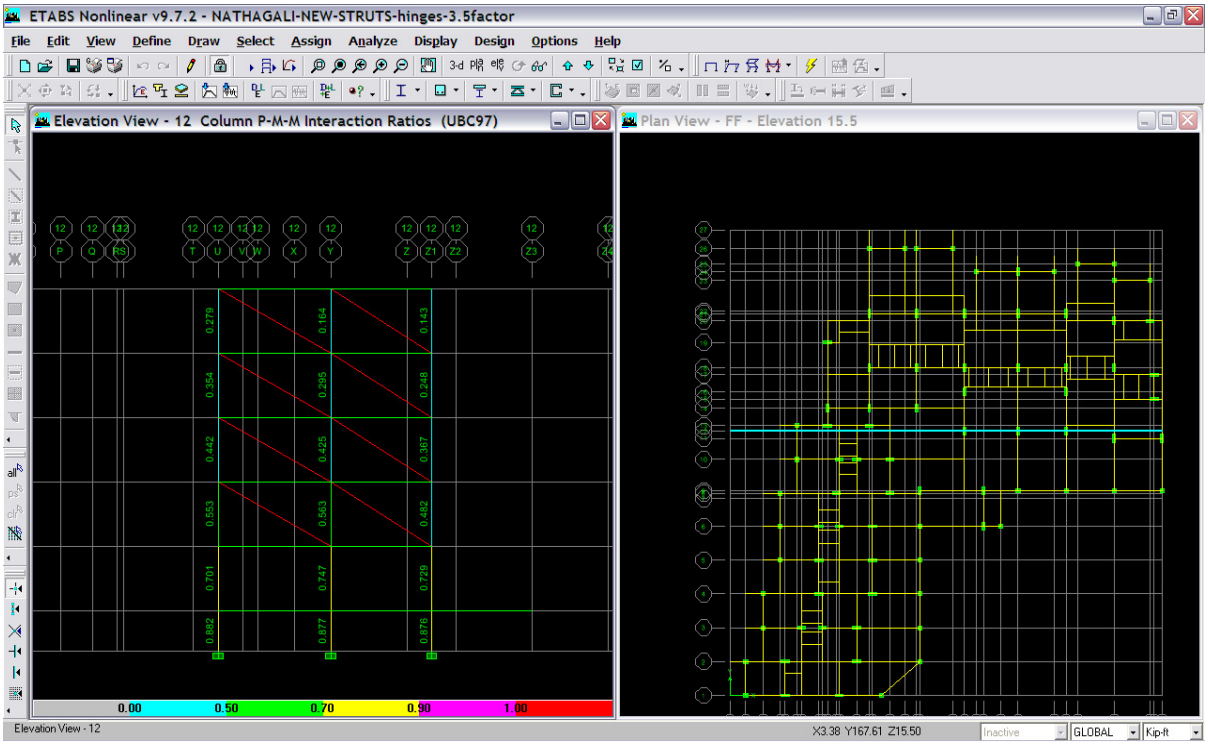

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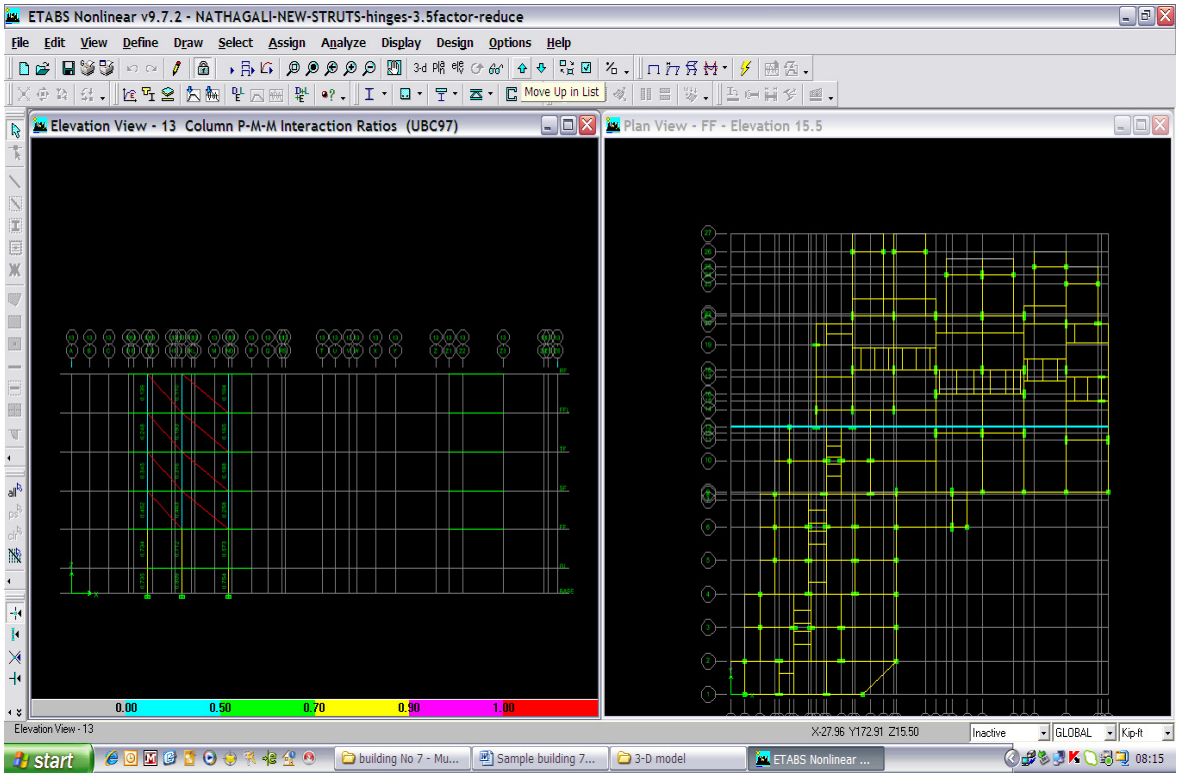

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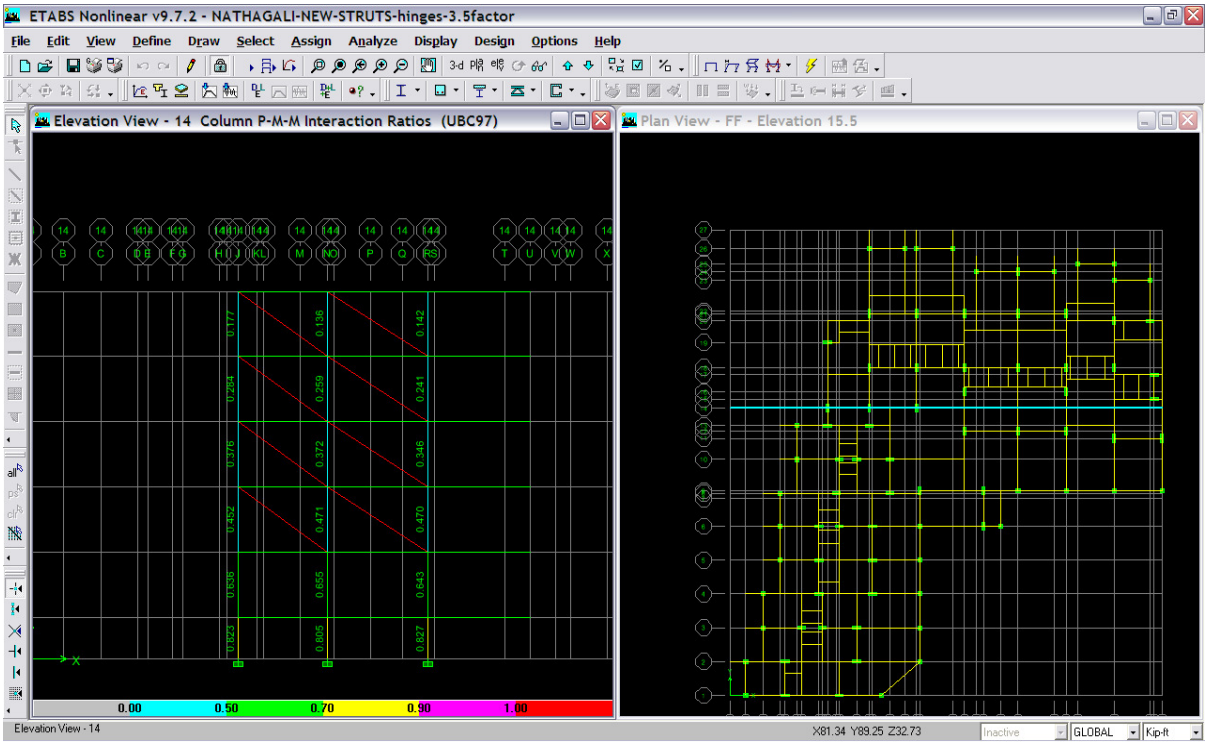

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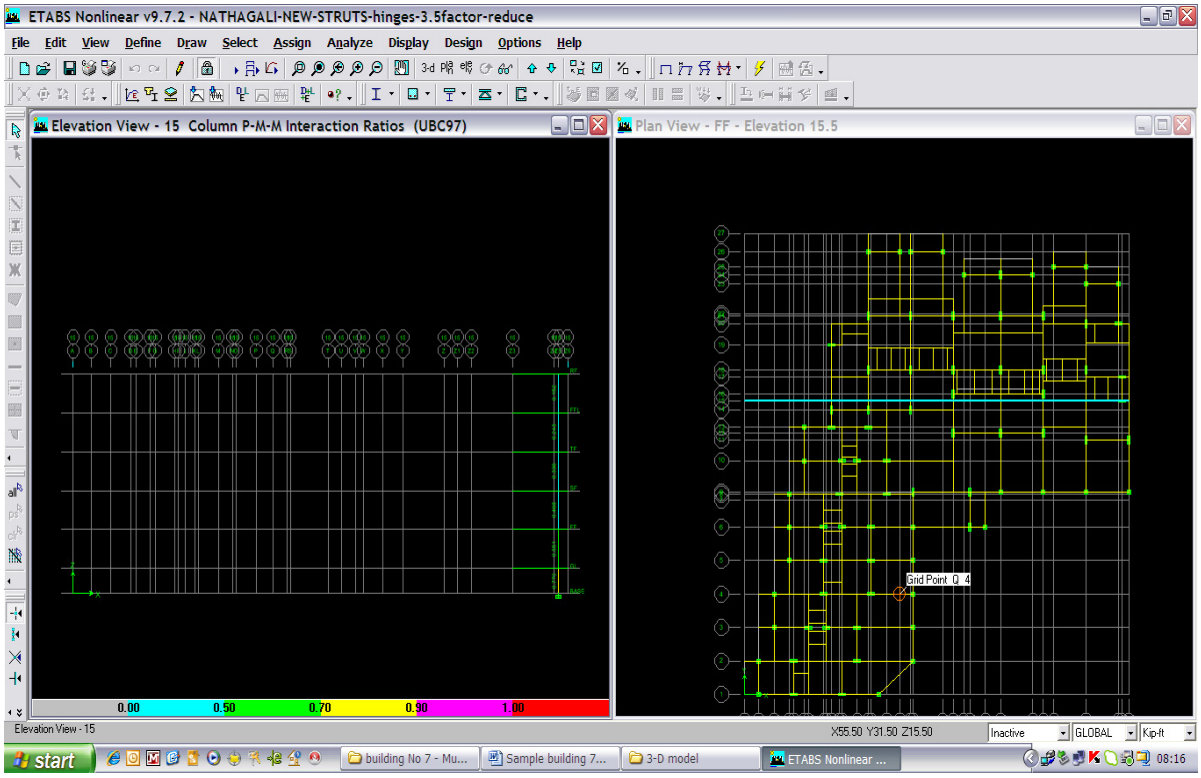

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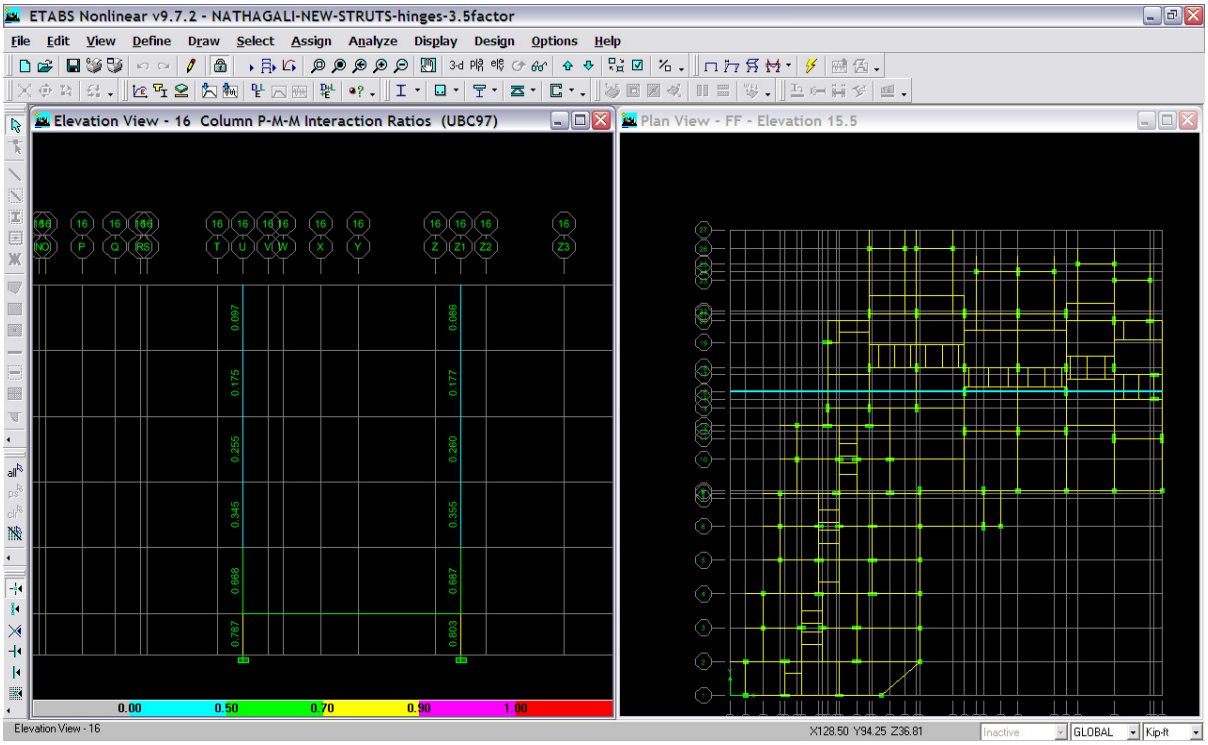

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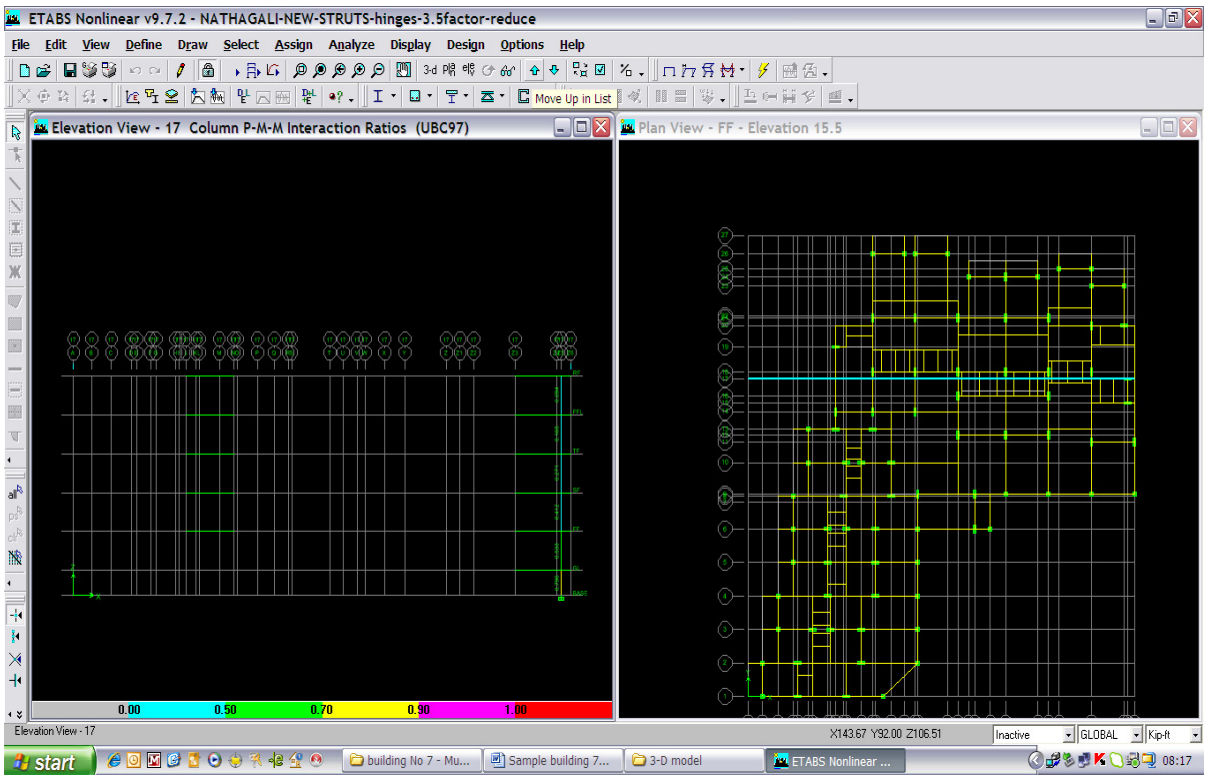

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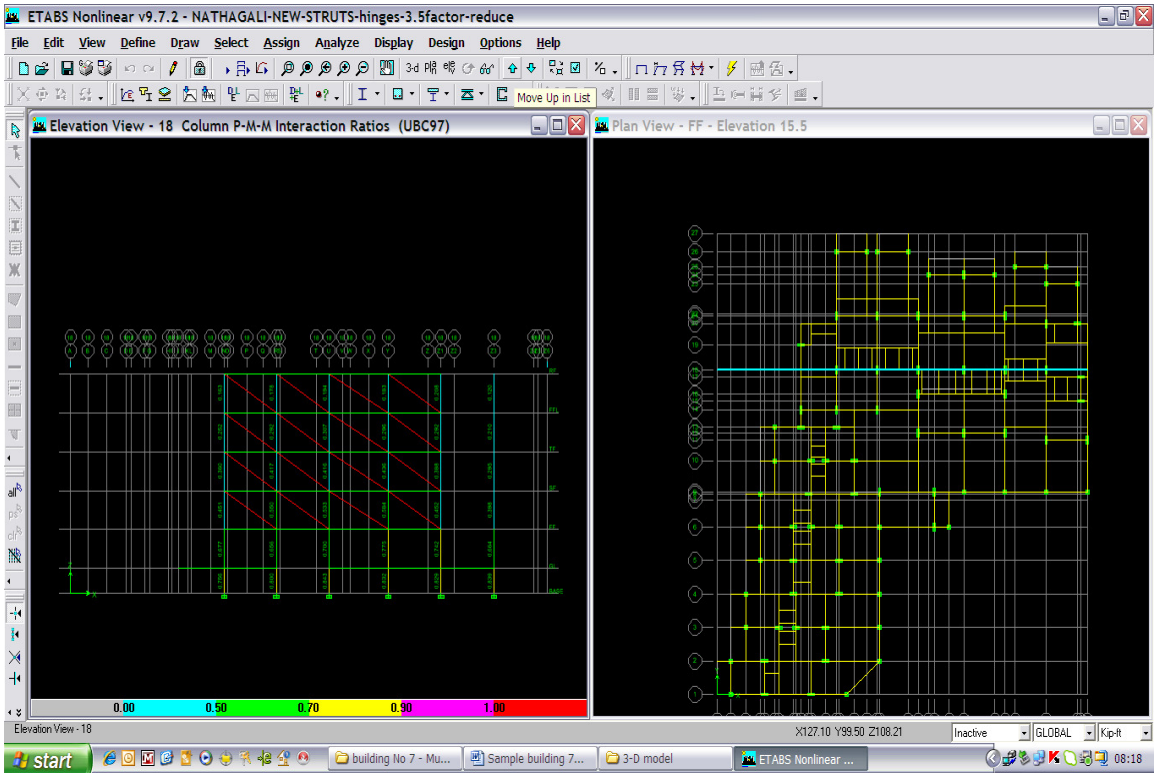

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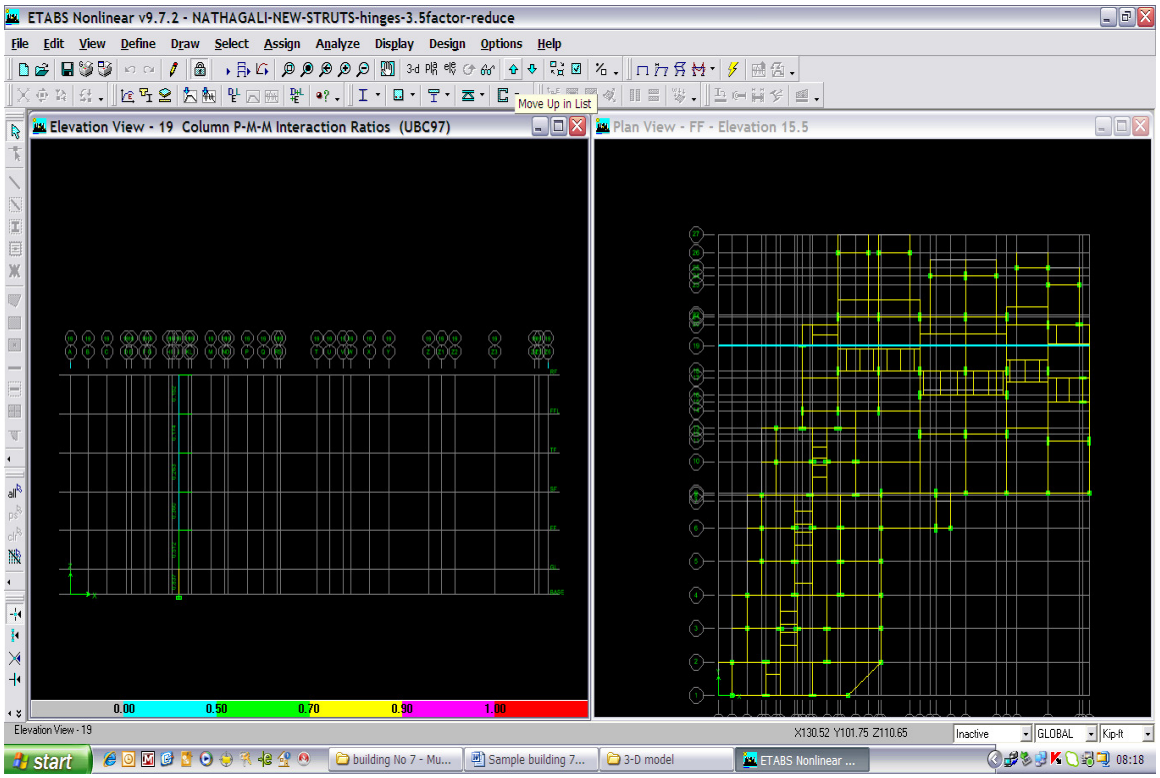

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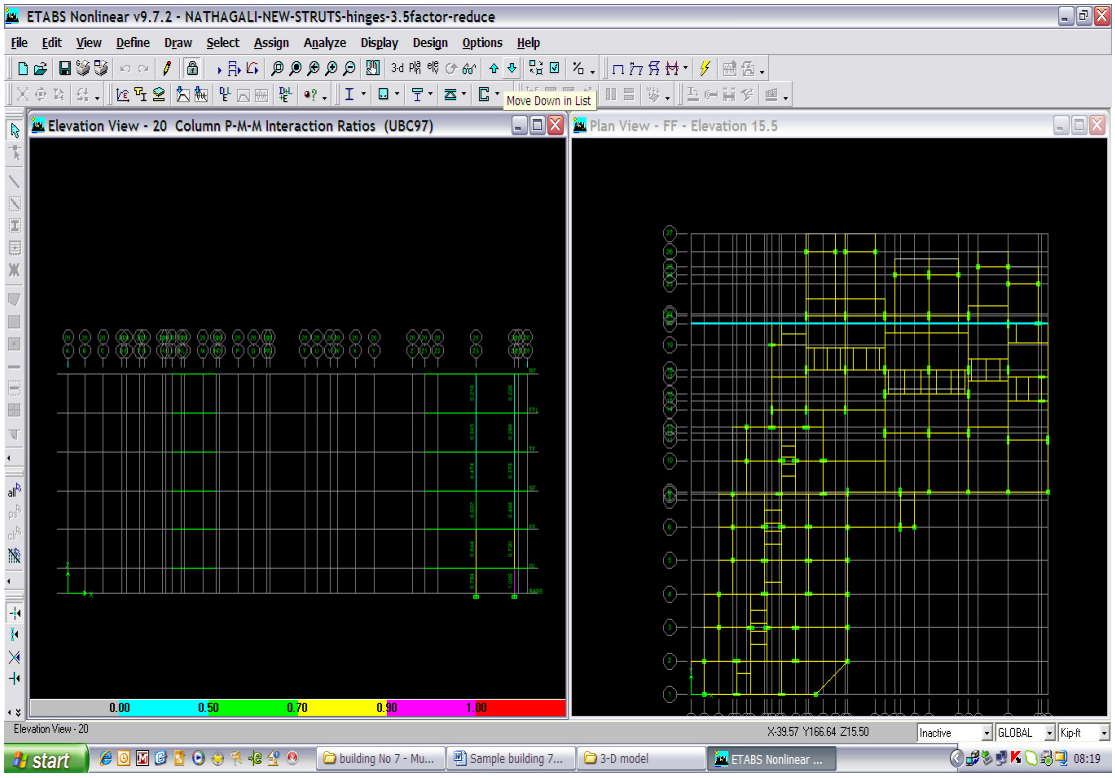

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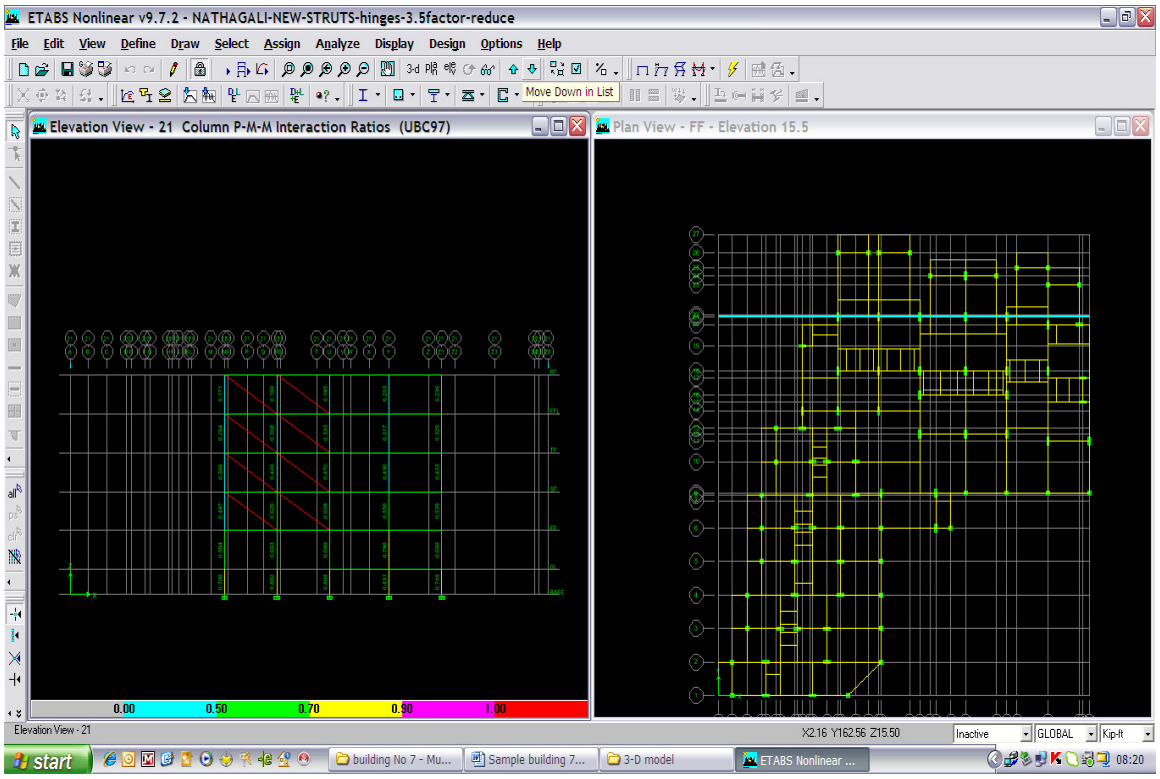

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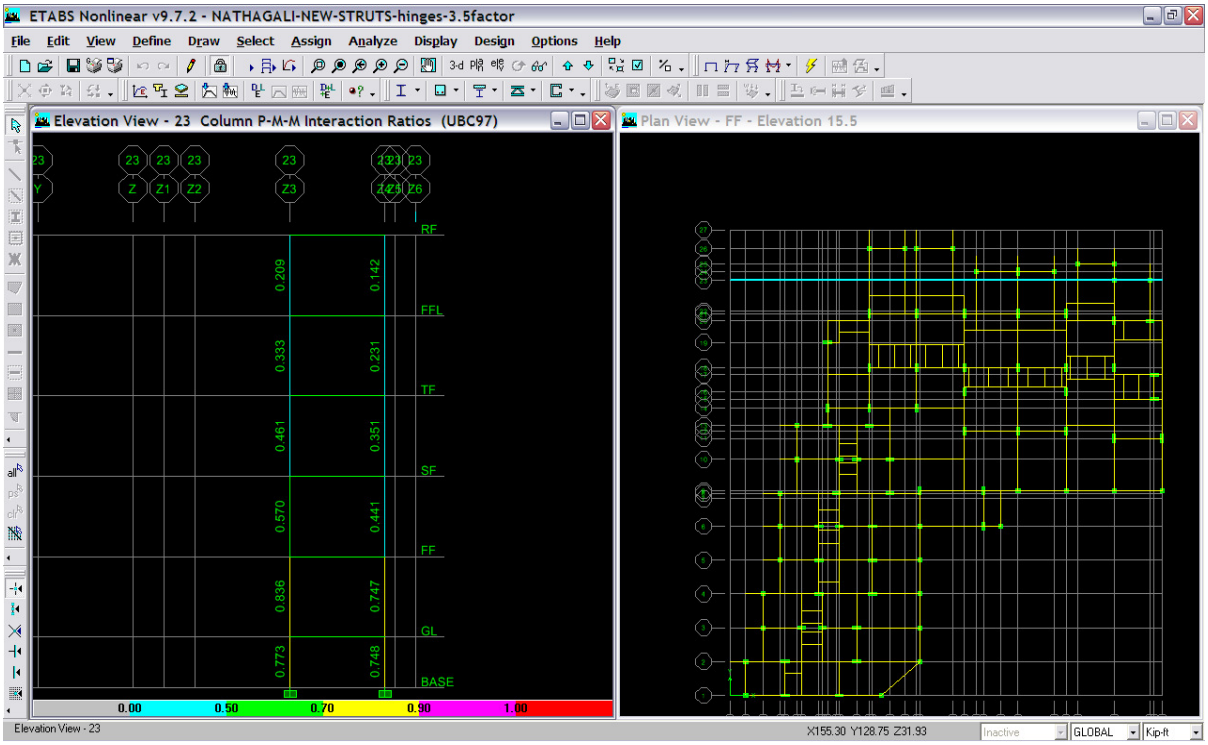

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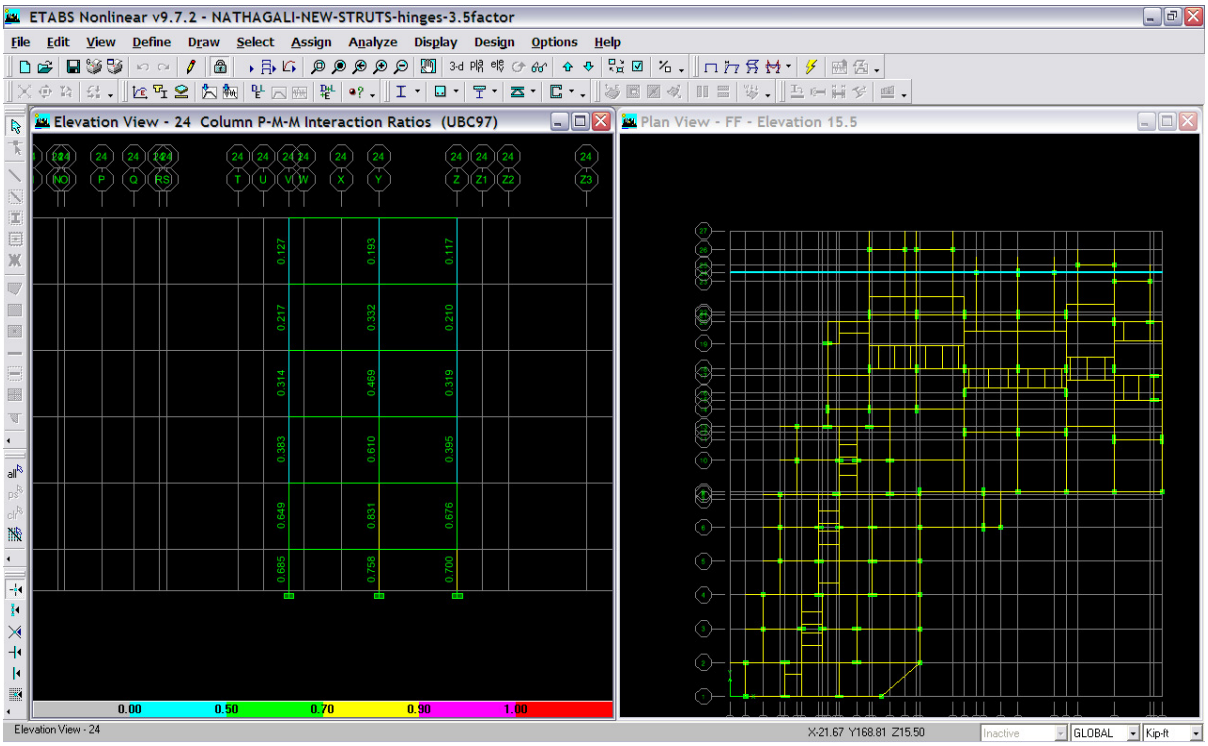
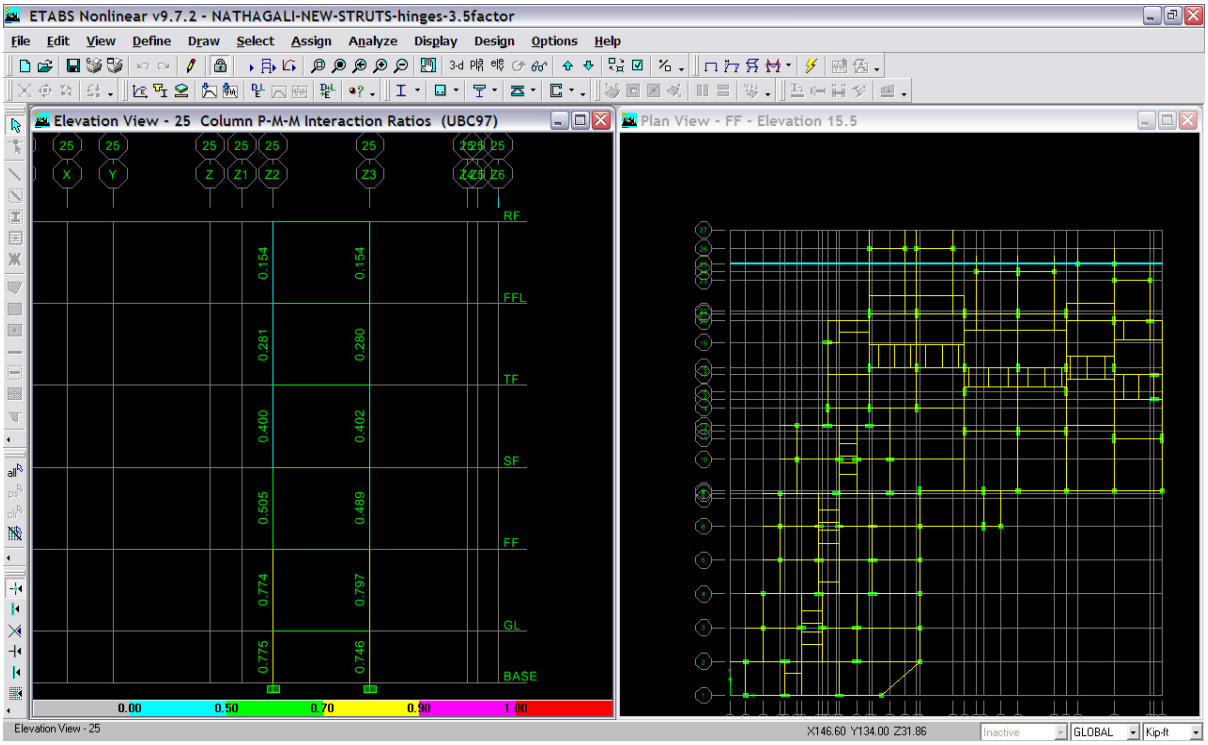
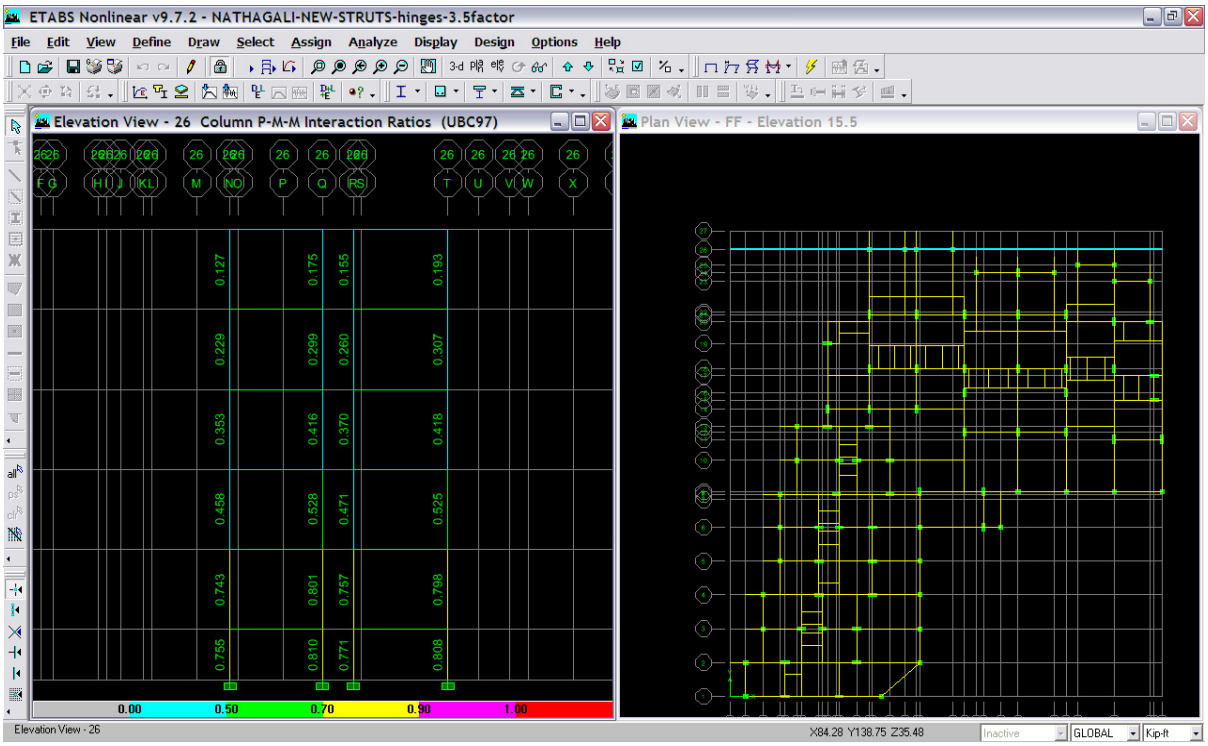


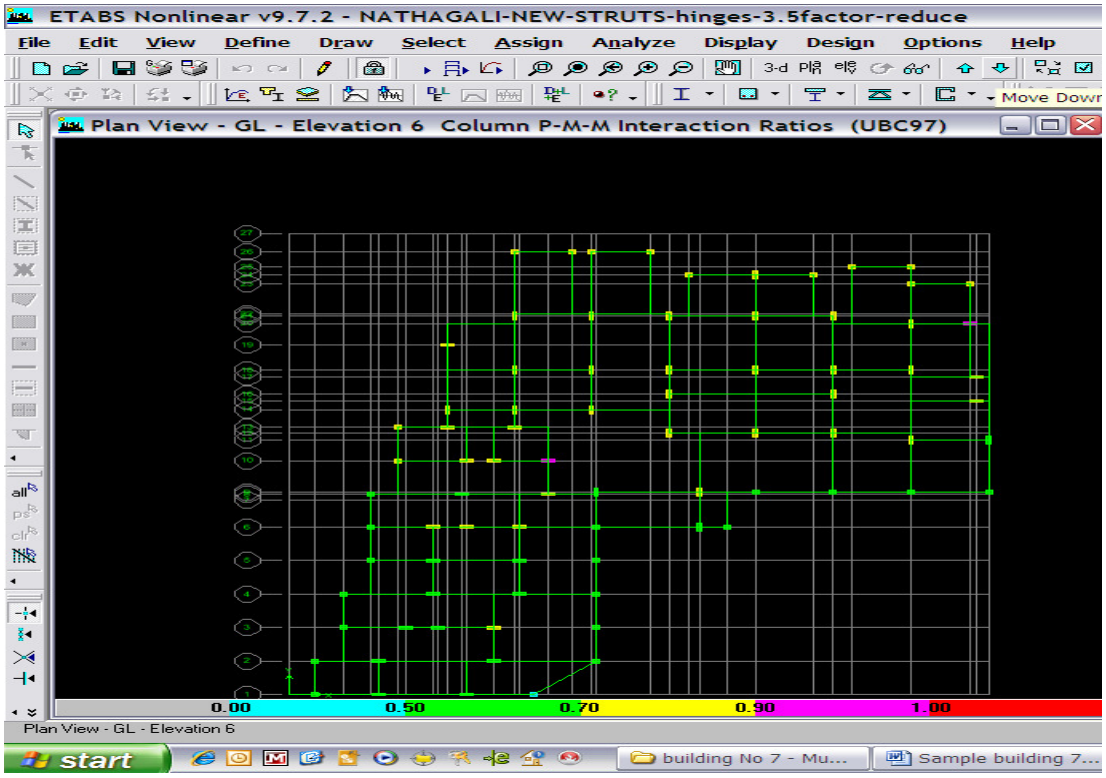
Demand/Capacity Ratios for Frame at Grid-24



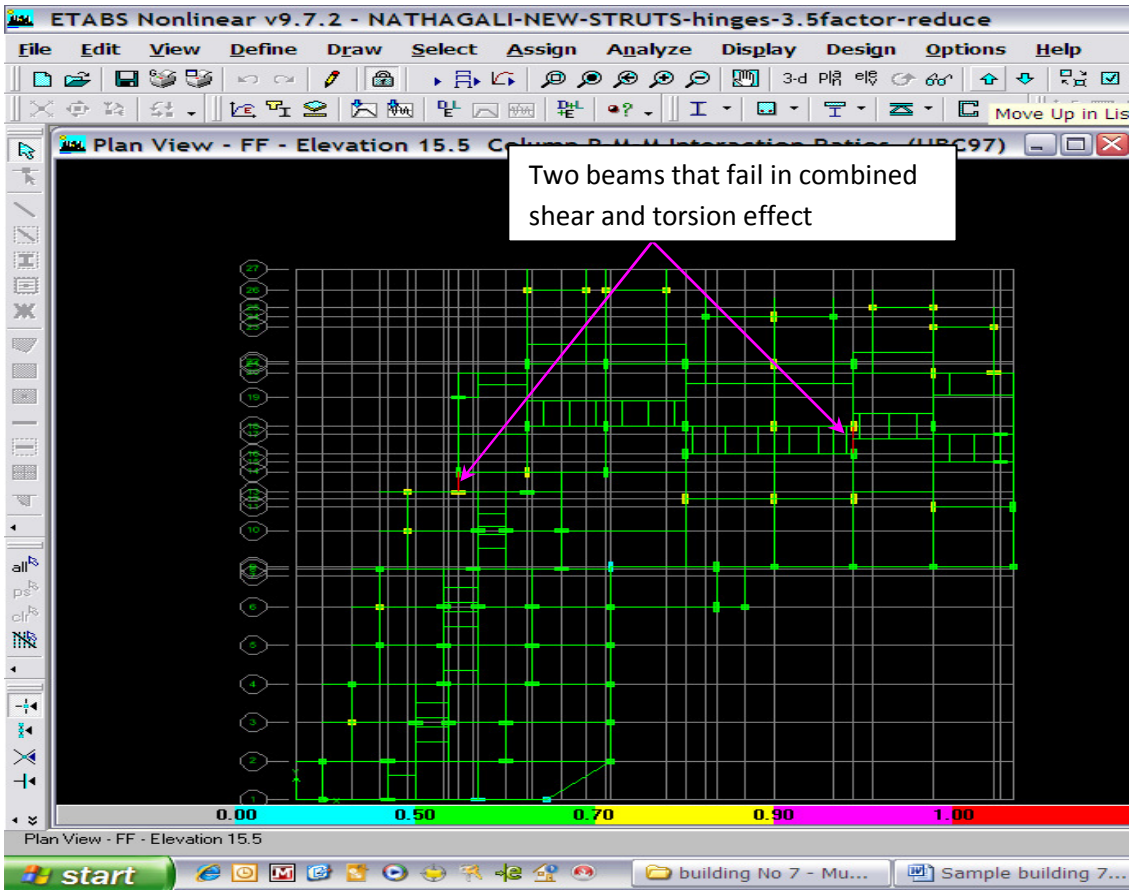
Demand/Capacity Ratios for Frame at Grid-25



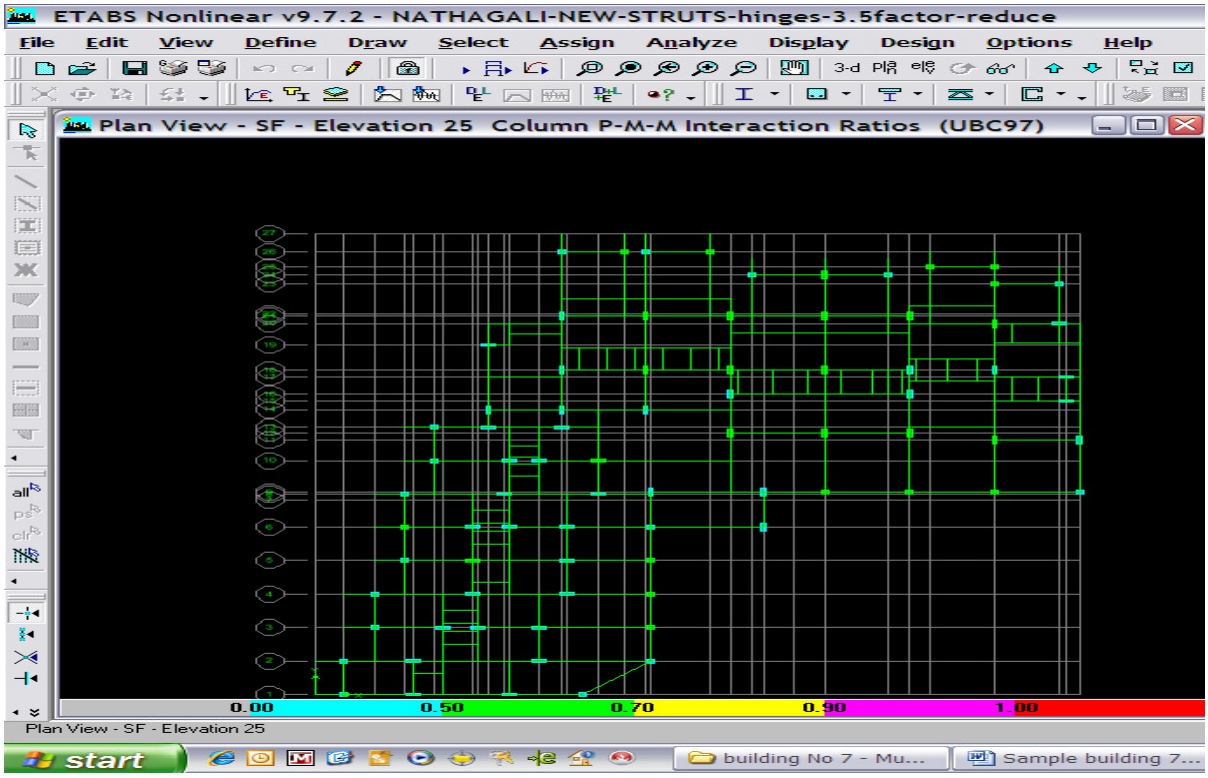
Demand/Capacity Ratios for Frame at Grid-26



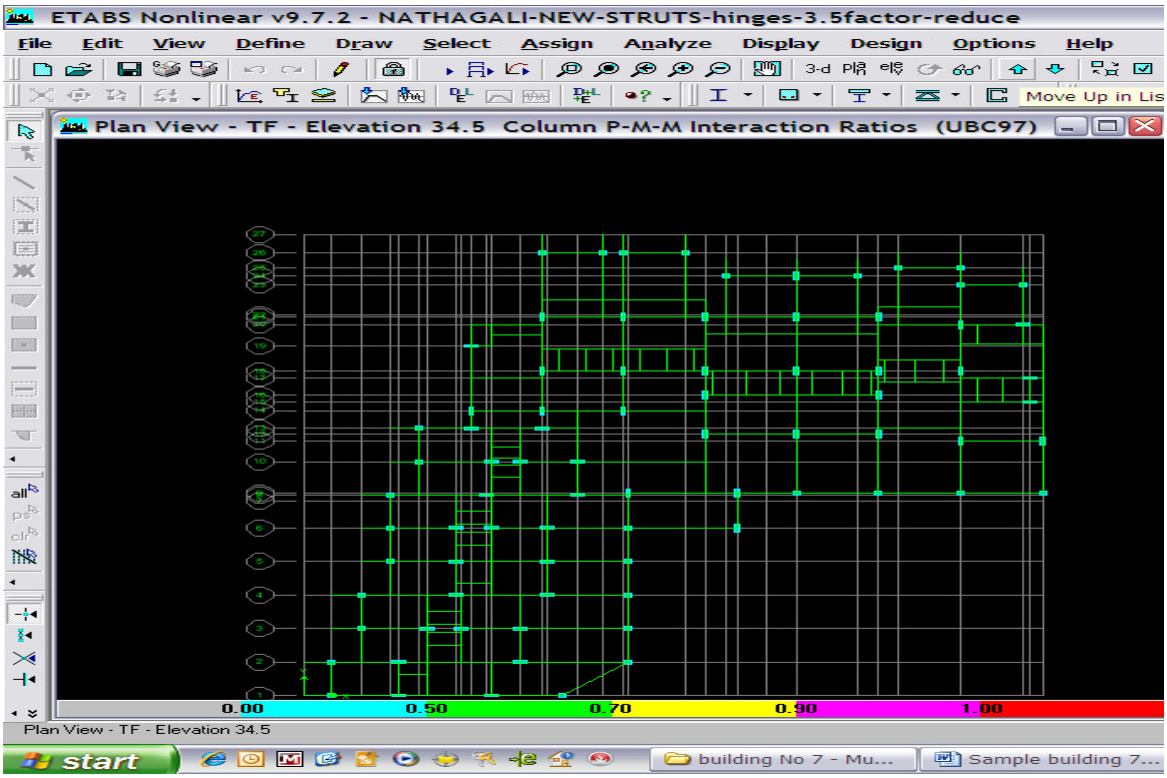
Demand/Capacity Ratios in plan at Ground Level



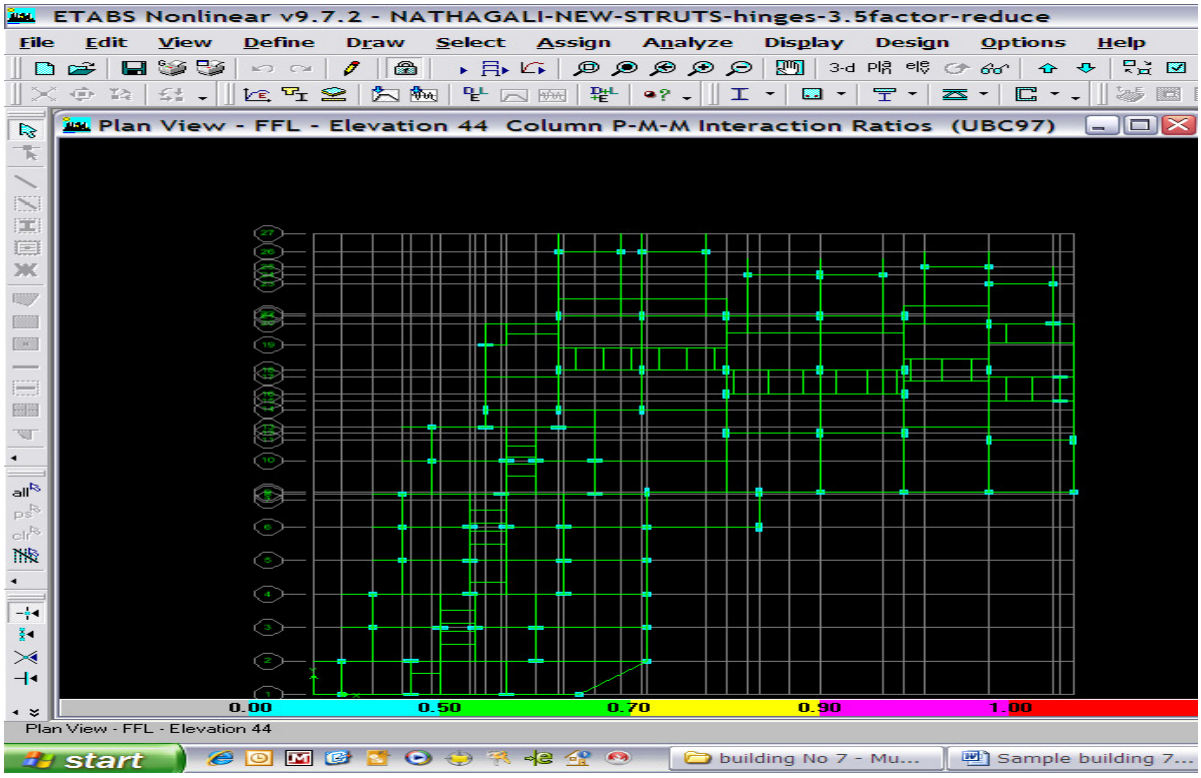
Demand/Capacity Ratios in plan at First Floor Level



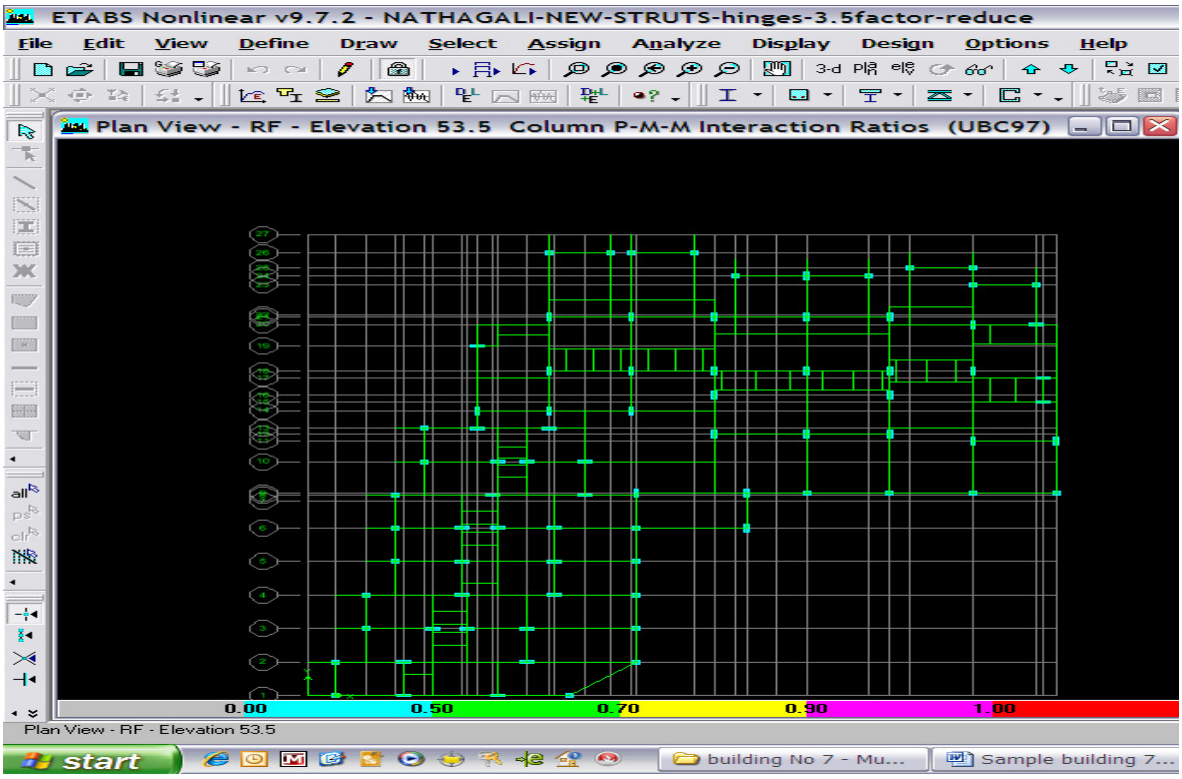
Demand/Capacity Ratios in plan at Second Floor Level



Demand/Capacity Ratios in plan at Third Floor Level



Demand/Capacity Ratios in plan at Fourth Floor Level



Demand/Capacity Ratios in plan at Roof Level