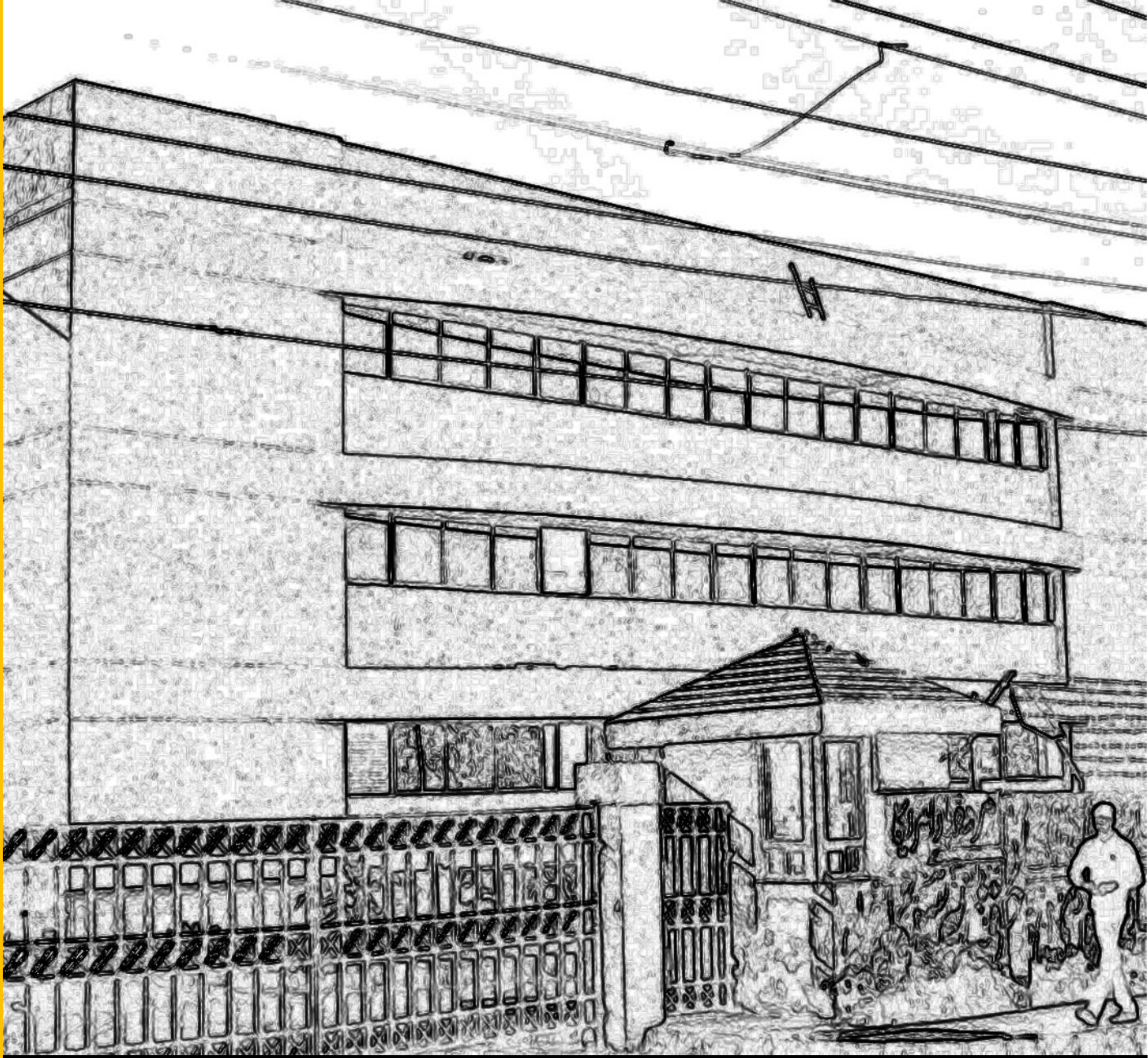

# Four Storey Academic Building in Karachi

## 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

HEC

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

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# Summary

The building is a reinforced concrete framed building with four storeys including the ground floor, which is located in Karachi. It is an academic building with classrooms, computer laboratories and administrative offices. 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 academic buildings: an eccentrically located reinforced concrete core, plan irregularities and heavy, stiff unreinforced masonry infill walls that were not considered during the structural design of the building.

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

## 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 Mr. Aslam Faqeer Mohammed and Ms. Najmus Sahar Zafar, Assistant Professors, Department of Civil Engineering, NED University of Engineering and Technology, and Mr. Shamsoon Fareed, Lecturer, 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 provided an opportunity for participants to study a real academic building with all the associated physical constraints. The Tier 1 vulnerability assessment exercise identified a number of potential vulnerabilities. A Tier 2 assessment consisting of a linear static structural analysis was carried out to assess the vulnerabilities and potential solutions in more detail. The detailed evaluation provided the students working on the case study with valuable hands-on practice using structural analysis software ETABS and with a better understanding of the ASCE/SEI 31-03 and FEMA documents.

## Building Information

This building, shown in Figure 1, is a four-storey (ground plus three) academic building containing a combination of classrooms, administrative offices, computer laboratories, faculty offices and common areas. The ground floor contains primarily offices, and the third floor contains a small library. The building's overall dimensions are 90'-0" wide by 100'-0" long, and it is approximately 50 feet tall. The building has a reinforced concrete moment frame structural system with few 6" thick unreinforced concrete block masonry infill walls. The foundations are reinforced concrete isolated footings.

**Figure 1. Front elevation view of the building**

The building's architectural and structural drawings are shown in Figure 2 through Figure 13. Typical story height is 10'-0". Circular columns have 12" diameter and square columns have sizes of 12"x12". Slab is 6" thick and RCC walls are 12" thick. The building has a lift core which is eccentrically placed. 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.

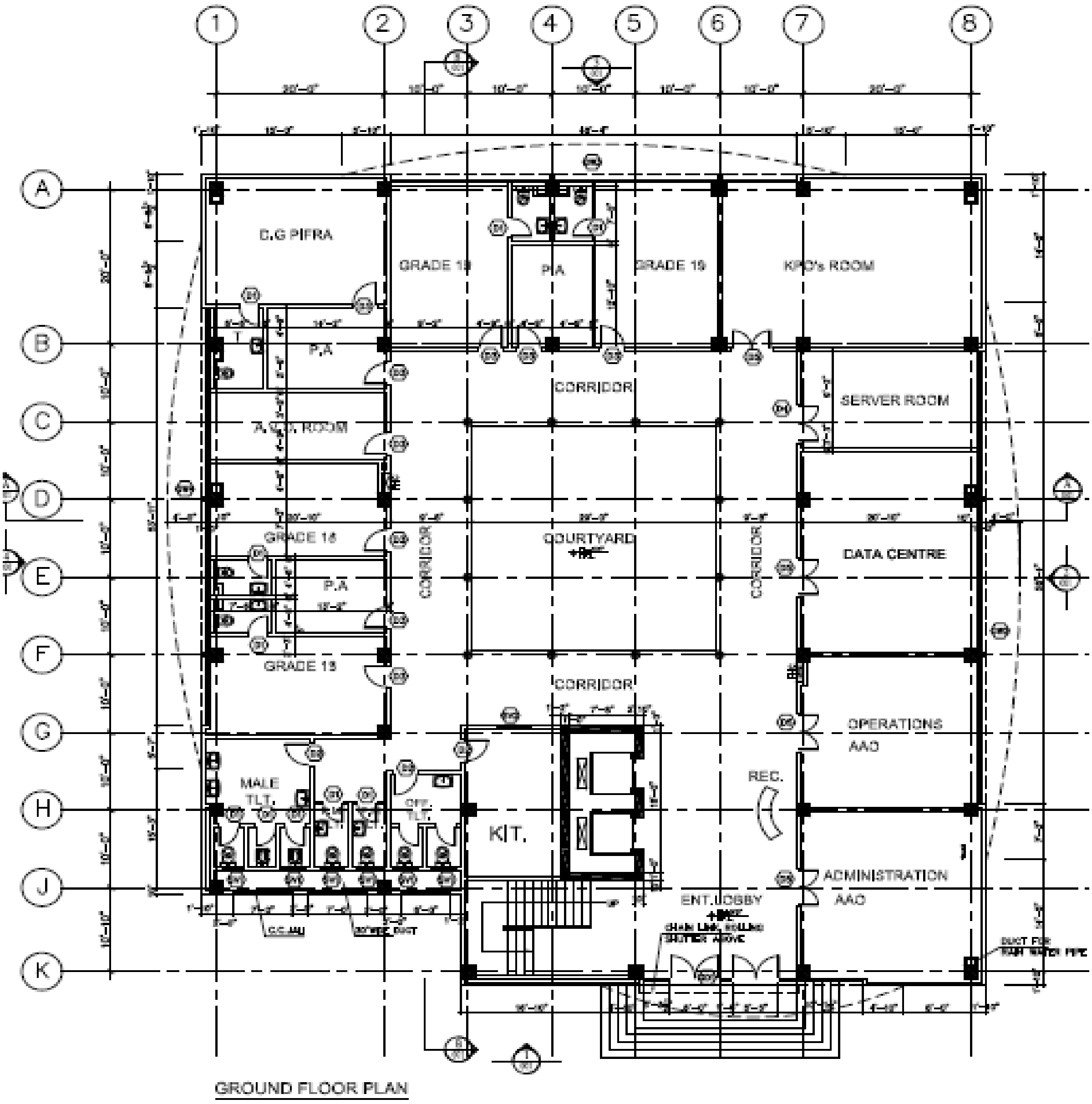



**Figure 2. Ground floor plan**

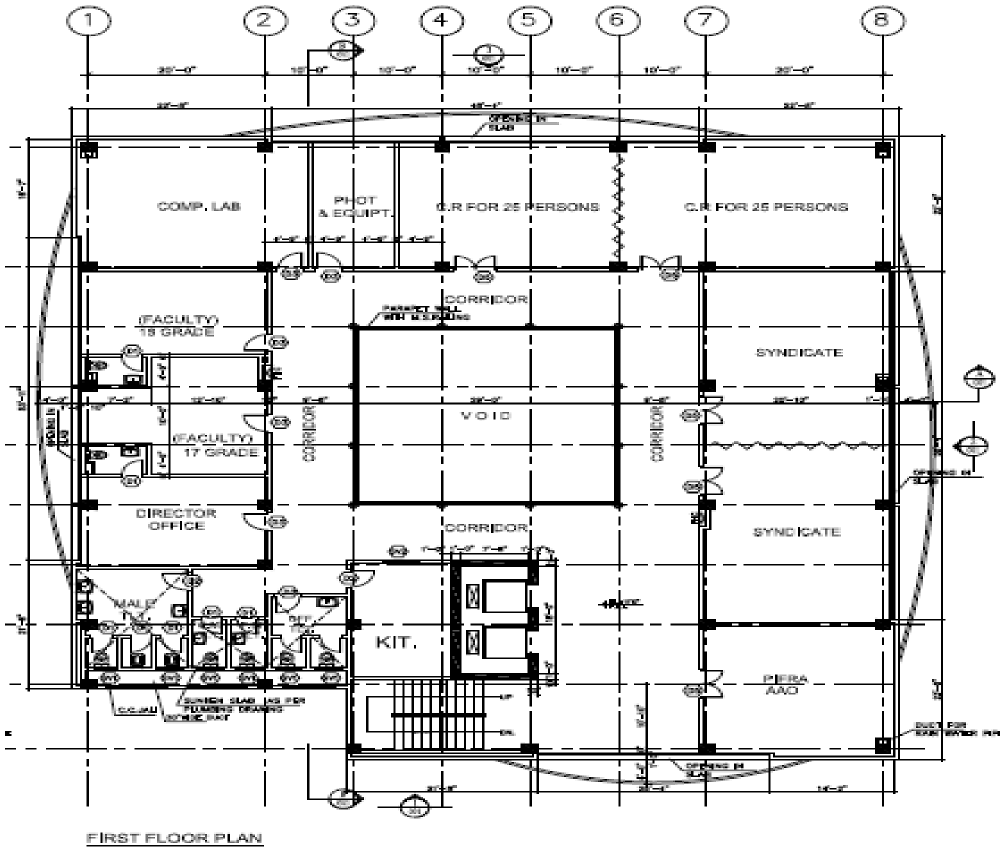



**Figure 3. First floor plan**

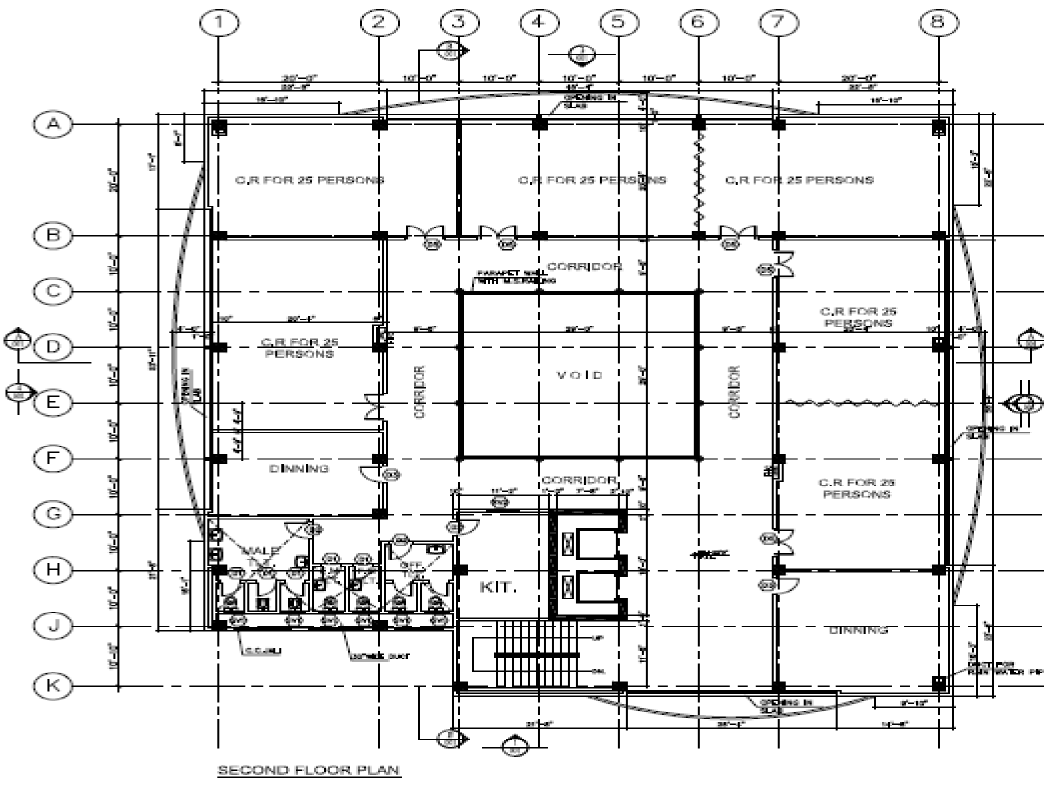



**Figure 4. Second floor plan**

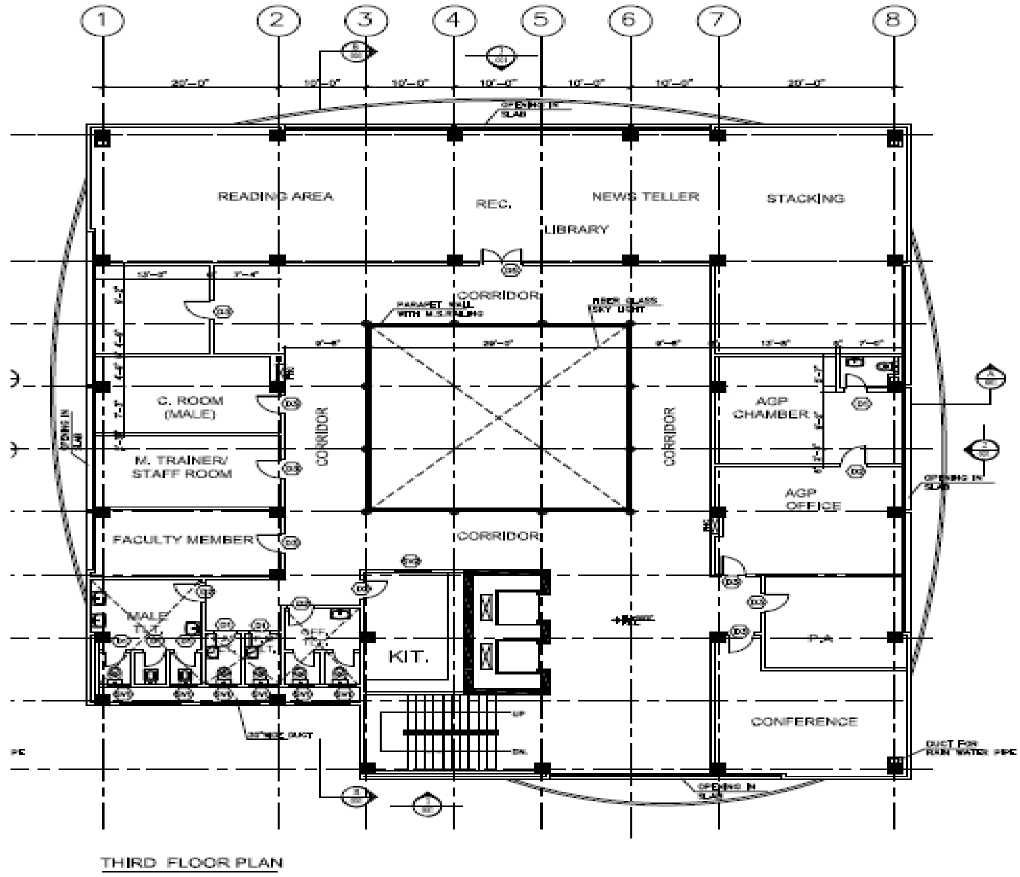



**Figure 5. Third floor plan**

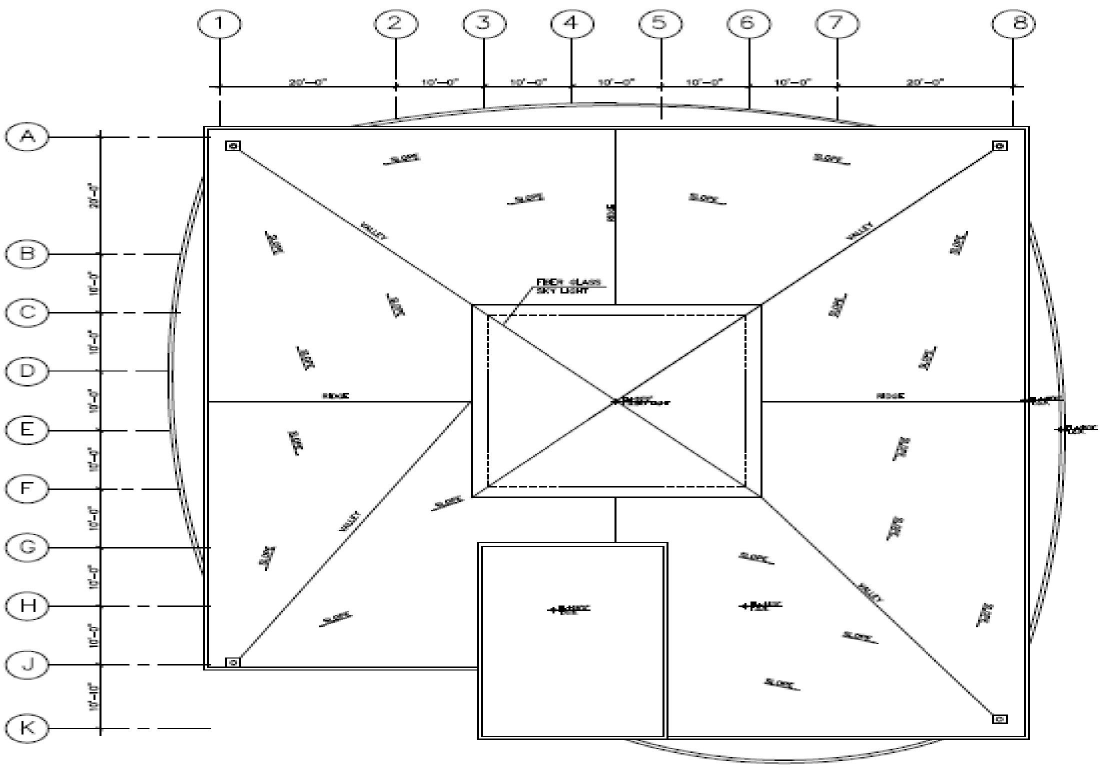


**Figure 6. Roof plan**

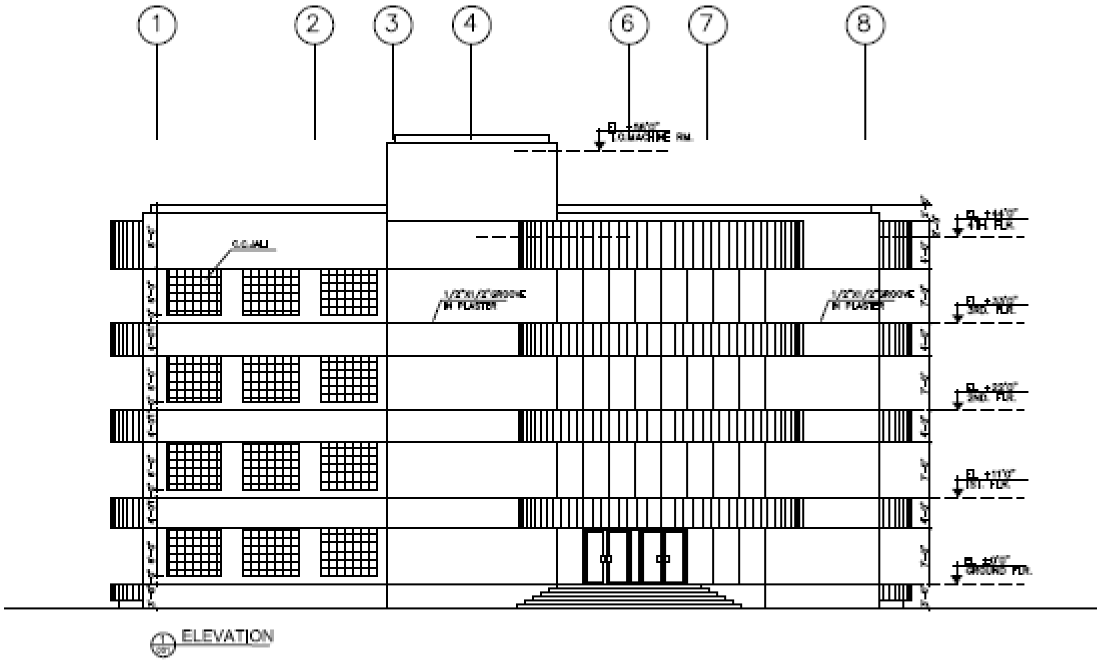



**Figure 7. Architectural elevation**

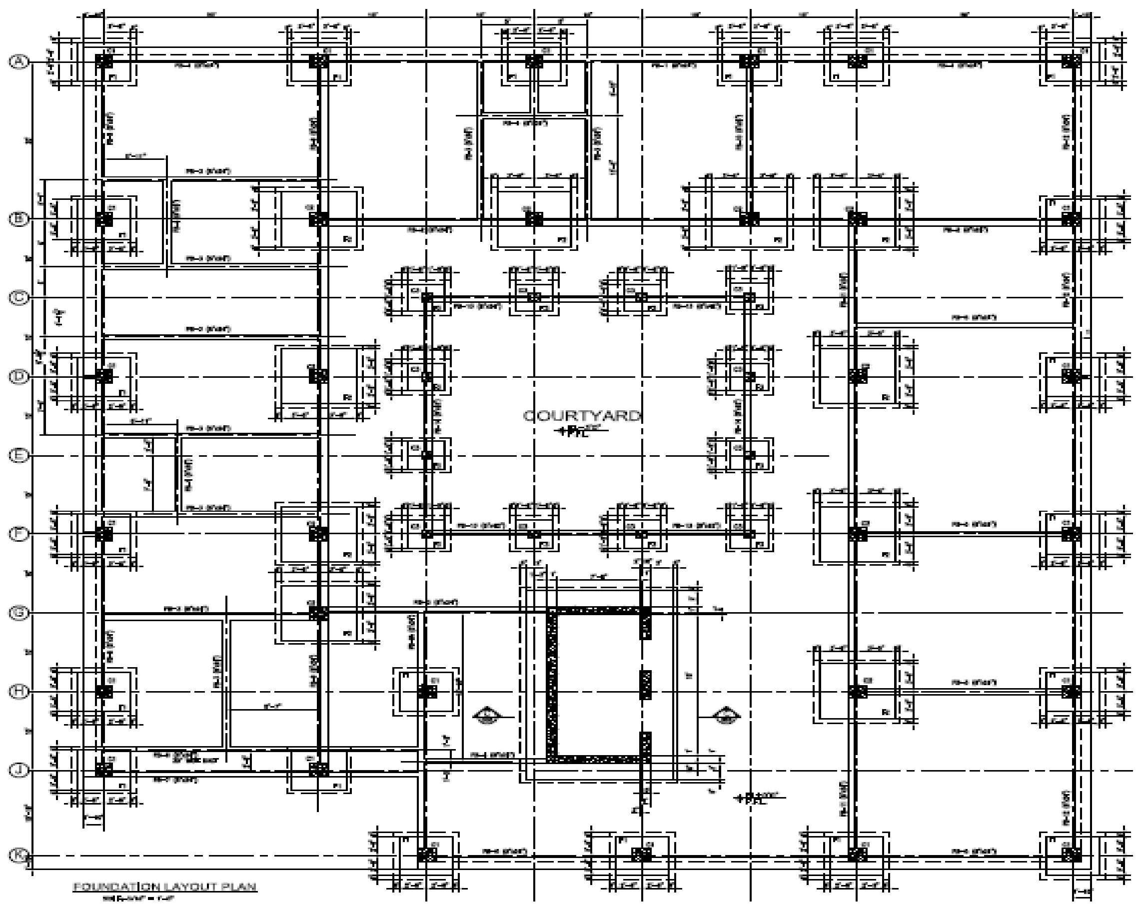



**Figure 8. Foundation layout plan**

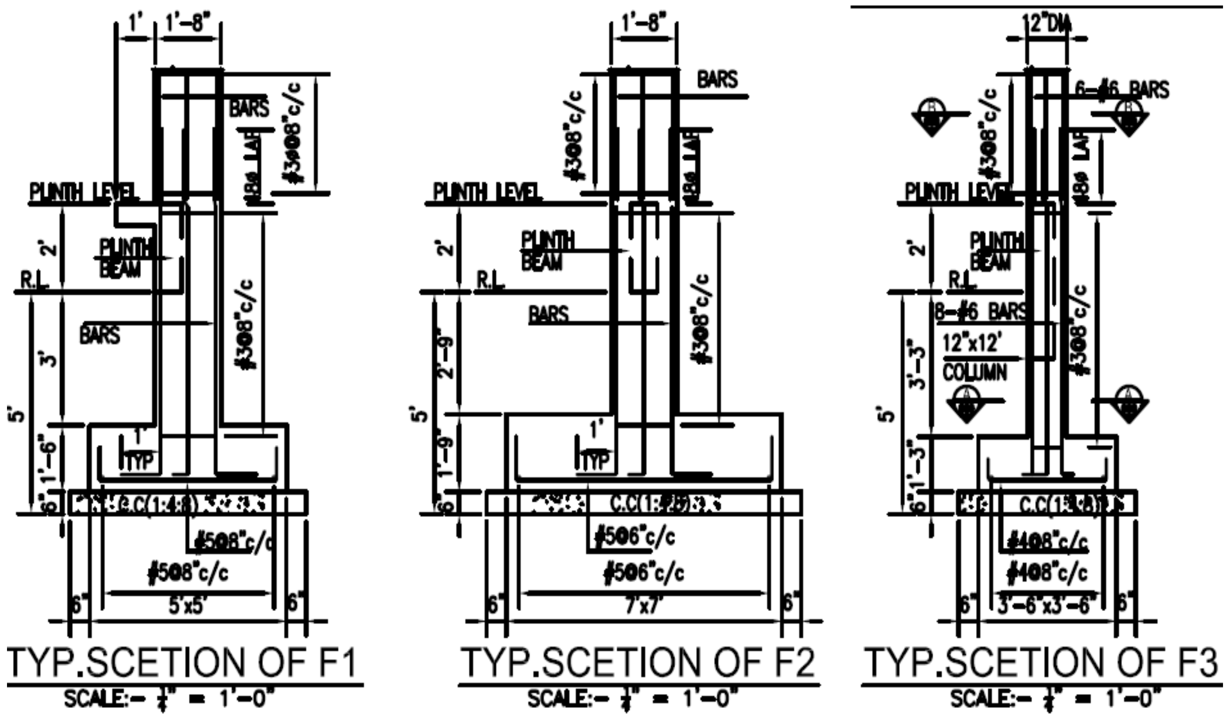



**Figure 9. Sections of footings and columns**

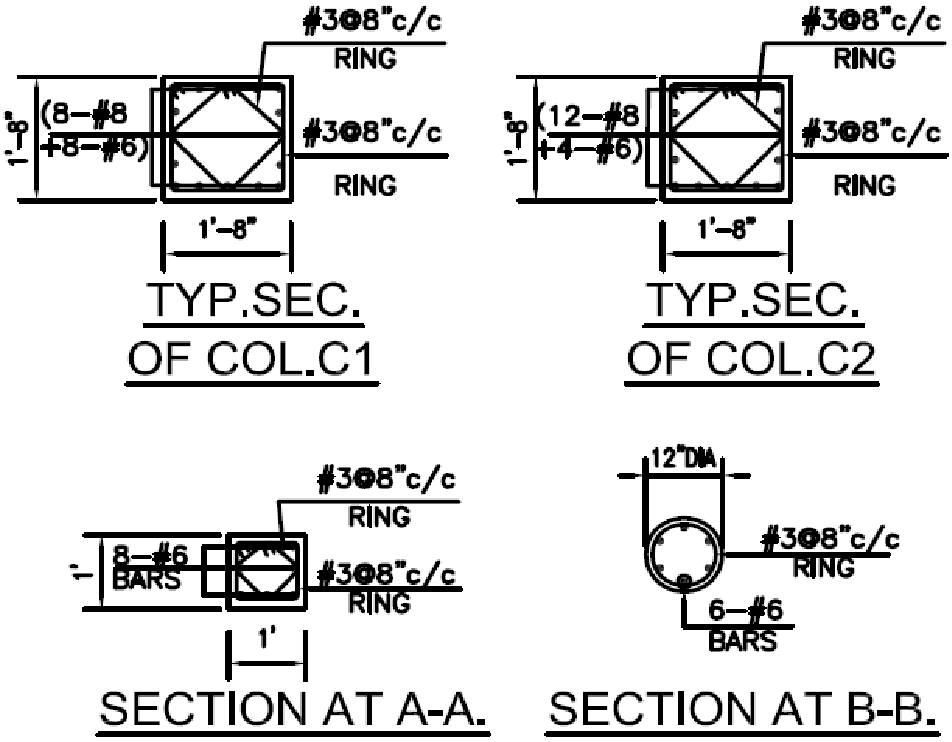



**Figure 10. Typical column sections**

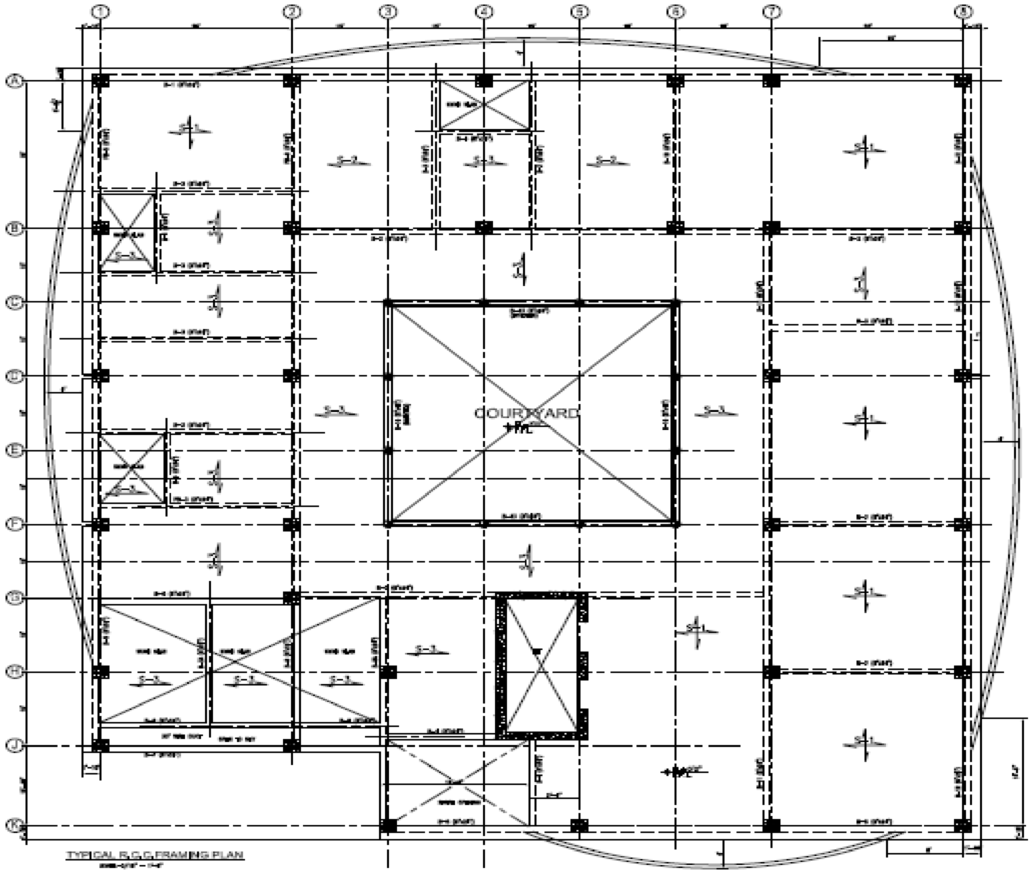



**Figure 11. Typical framing plan**

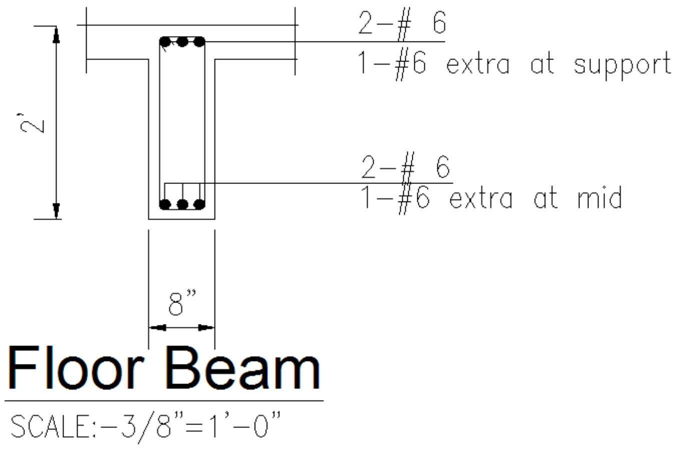



**Figure 12. Typical floor beam section**

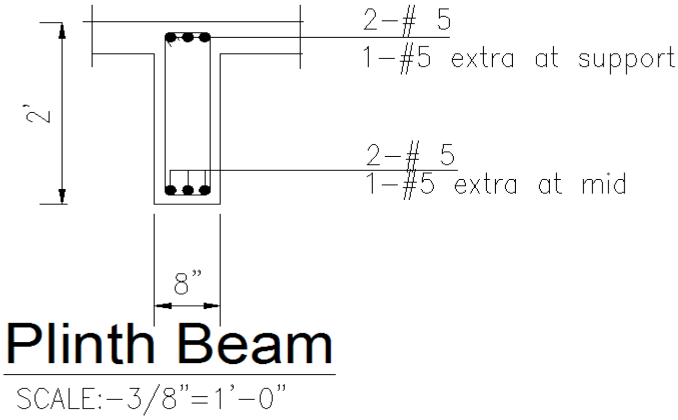



**Figure 13. Typical plinth beam section**

## 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). Site Class $S_B$ was used in the analyses.

## 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 Table 1.

**Table 1. Summary of Tier 1 Assessment**

| Checklist | Tier 1<br>Non-compliant Items |
|---|---|
| Building System | Weak Storey<br>Mass irregularity<br>Torsion irregularity |
| Lateral Force-resisting System | None |
| Geologic Hazards and Foundation | None |

## Linear Evaluation

For Tier 2 linear elastic analysis, a 3-D model of the building was developed in ETABS Nonlinear version 9.7.0, which is shown in Figure 14. The beams and columns were modeled with linear beam-column elements, and the infill walls were modeled with single linear compression struts. The reinforced concrete walls were modeled as membrane area elements. The linear analysis results show that there no columns with demand/capacity ratios (DCRs) greater than one, so the building is expected to respond within the linear range. Table 2 shows ETABS modeling parameters. Table 2 shows ETABS loading input parameters. Please see Appendix B for linear analysis results.

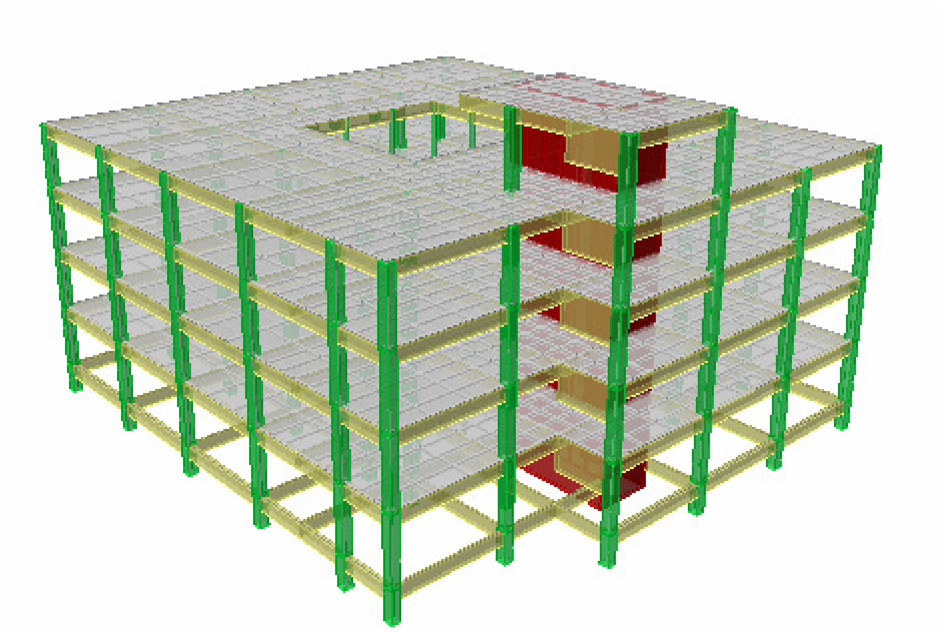


**Figure 14. Rendering of linear ETABS model of the building**

**Table 2. ETABS loading input parameters**

| | |
|---|---|
| Dead load<br><br><br><br><br>Live load | Self weight.<br>6" thick wall load.<br>24 psf finishes<br>165 psf sunk<br>12" RCC wall<br>According to usage of area (UBC-97) |
| Earthquake load<br>Z<br>$C_a$<br>$C_v$ | <br>0.4g<br>0.4 with $N_a$ = 1.0<br>0.4 with $N_v$ = 1.0 |
| Soil type | $S_B$ |

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 3), soft storey (shown in Table 4), and storey drift (shown in Table 5) and mass irregularity (shown in Table 6).

**Table 3. Torsion irregularity check**

| Story | Diaphragm | XCM | YCM | XCR | YCR | % diff X (allow 20%) | % diff Y (allow 20%) |
|---|---|---|---|---|---|---|---|
| ROOF | D1 | 514.243 | 581.899 | 440.73 | 374.837 | 6.8 | 17.3 |
| 3RD | D1 | 518.298 | 577.781 | 446.22 | 344.532 | 6.7 | 19.4 |
| 2ND | D1 | 519.204 | 578.073 | 453.375 | 312.726 | 6.1 | 22.1 |
| 1ST | D1 | 506.671 | 601.024 | 470.539 | 289.313 | 3.3 | 26.0 |
| GF | D1 | 513.189 | 604.559 | 507.337 | 330.447 | 0.5 | 22.8 |

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

There is a torsion irregularity at ground, first and second floor levels.

**Table 4. Soft storey check**

| | | | | | % diff in K (30% allow) | |
|---|---|---|---|---|---|---|
| Story | Load | storey force | Total Displacement | Stiffness | % difference compare to | |
| | | kips | inches | kip/in | Above storey | Below storey |
| ROOF | EX | 386 | 1.0564 | 365.39 | ---- | 19.6 |
| 3RD | EX | 381 | 0.8388 | 454.22 | 24.3 | 3.4 |
| 2ND | EX | 271 | 0.5766 | 470.00 | 3.5 | 16.4 |
| 1ST | EX | 169 | 0.3005 | 562.40 | 19.7 | 12.8 |
| GF | EX | 30 | 0.0465 | 645.16 | 14.7 | ---- |

| | | | | | % diff in K (30% allow) | |
|---|---|---|---|---|---|---|
| Story | Load | storey force | Total Displacement | Stiffness | % difference compare to | |
| | | kips | inches | kip/in | Above storey | Below storey |
| ROOF | EY | 420 | 0.3828 | 1097.18 | ---- | 21.3 |
| 3RD | EY | 414 | 0.297 | 1393.94 | 27.0 | 6.7 |
| 2ND | EY | 295 | 0.1974 | 1494.43 | 7.2 | 23.2 |
| 1ST | EY | 183 | 0.094 | 1946.81 | 30.3 | 0.9 |
| GF | EY | 33 | 0.0171 | 1929.82 | 0.9 | ---- |

The first storey is the only one which is slightly exceeding the allowable limit.

**Table 5. Storey drift check**

| Story | Etab Drift X | Code Modified Drift | Etab Drift Y | Code Modified Drift |
|---|---|---|---|---|
| | $\Delta_S$ | $\Delta_M$ | $\Delta_S$ | $\Delta_M$ |
| ROOF | 0.002434 | 0.00937 | 0.001243 | 0.00479 |
| 3RD | 0.003437 | 0.01323 | 0.001506 | 0.00580 |
| 2ND | 0.004121 | 0.01587 | 0.001619 | 0.00623 |
| 1ST | 0.003823 | 0.01472 | 0.001292 | 0.00497 |
| GF | 0.001536 | 0.00591 | 0.000514 | 0.00198 |

The drifts both in x and y directions are within limit of 0.02.

**Table 6. Mass irregularity check**

| | | % diff in Mass (50% allow) | |
|---|---|---|---|
| | | % difference compare to | |
| Story | MassX | Above storey | Below storey |
| ROOF | 3.3894 | --- | 21 |
| 3RD | 4.3153 | 27 | 0 |
| 2ND | 4.3241 | 0 | 5 |
| 1ST | 4.5313 | 5 | 77 |
| GF | 2.567 | 43 | ---- |

There is no mass irregularity in the building.

## Hand Calculation Checks

The case study team checked the demand/capacity ratios for floor and plinth beams by hand, and also checked the joint shear capacity with hand calculations based on the methods American Concrete Institute (ACI) Standard 352-02. The check of shear demand versus capacity at critical joints shows that the joints have adequate strength.

**Demand Capacity Ratios for Beams**

**For Floor Beams**

Required Reinforcement at mid span =0.82 in2
Provided Reinforcement at mid span =1.32 in2
Ratio = 0.62

Required Reinforcement at support =0.90 in2
Provided Reinforcement at support =1.32 in2
Ratio = 0.0.69

**For Plinth Beams**

Required Reinforcement at mid span =0.53 in2

Provided Reinforcement at mid span =0.918 in2

Ratio = 0.58

Required Reinforcement at support =0.65 in2

Provided Reinforcement at support =0.918 in2

Ratio = 0.71

**Joint Shear Check**

The following equations from ACI 352-02 were applied to the joint shown in Figure 15, using the value of γ from ACI 352-02 Table 6-10, shown in Figure 16.

$$M_{pr,b} = A_s \alpha f_y \left(d - \frac{a}{2}\right) \qquad T_u = A_s \alpha f_y \qquad V_u = T_u - V_{col}$$

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

$$V_n = \gamma \sqrt{f'_c}\, b_j h_c$$

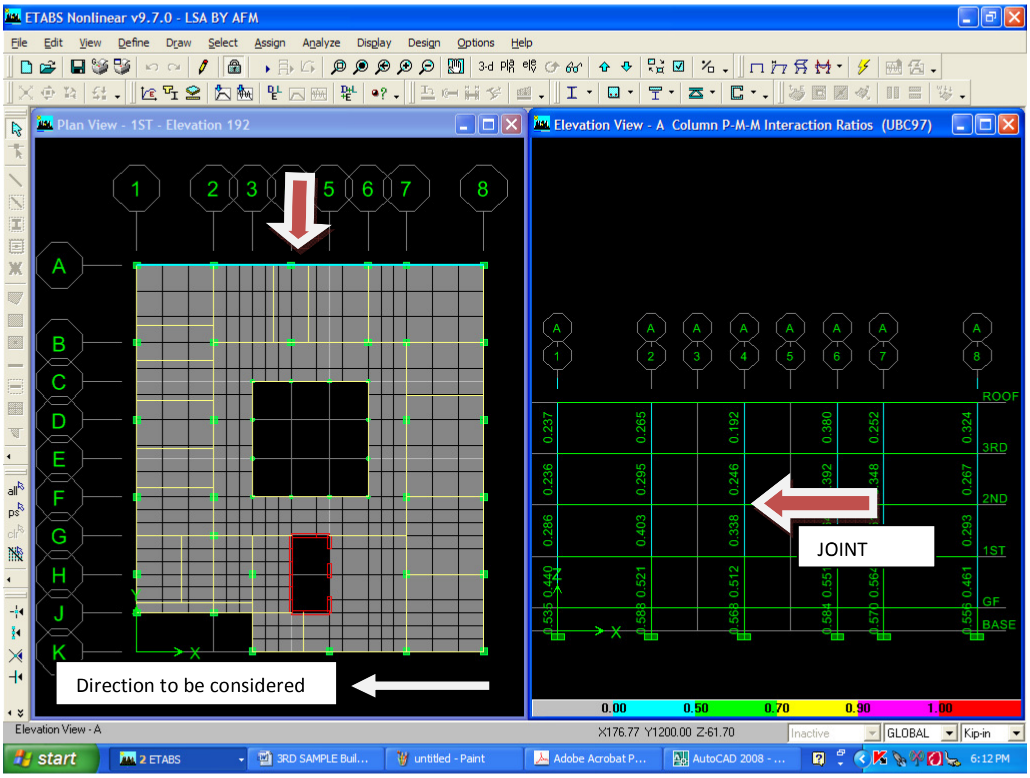



**Figure 15. Joint checked for shear by hand calculation**

**Table 6-10. Values of $\gamma$ for Joint Strength Calculation**

| | Value of $\gamma$ | | | | |
|---|---|---|---|---|---|
| $\rho''$[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 16. Selected value of γ for joint shear check (from ACI 352-02)**

Using the above equations gives:

a = 1.32x60/(0.85x2.5x8) = 3.90

$M_{pr,b}$ = 2x3x0.44x1.0x60(21.5-3.90/2) = 3096.72kip-in (because of two beams)

$T_u$ = 3x0.44x1.0x60 = 79.20 kips

$V_{col}$ = 3096.72/(11x12) = 23.46 kips

$V_u$ = 79.20-23.46 = 55.74 kips

$b_j$ = 20 inches    $h_c$ = 20 inches

$V_n$ = 10 x (3000)^0.5 x20x20/1000 = 219.10 kips

**So joint shear strength is greater than the demand, and there is no need to retrofit the joint.**

## Results Summary

The case study results can be summarized as follows:

- 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 in one direction at the first floor level but the drifts are low so the formation of a soft storey mechanism is unlikely. Differences in stiffness are due to differences in infill wall distribution.
- Based on the Tier 2 analysis, all columns have demand capacity ratios less than one, and are thus expected to remain elastic in the design earthquake.
- Tier-2 results show that there exists torsion irregularity in ground, first and second floors. However, because the building is expected to remain elastic these irregularities are not likely to cause life-threatening damage.

- Joints have no reinforcing - column ties and beam ties are closely spaced at ends but do not continue through joint. However, a hand check of shear demand versus capacity at critical joints shows that the joints have adequate strength.

- Because the building was built after the 2005 earthquake, some seismic design requirements were followed. This helps explain the building's relatively good behavior.

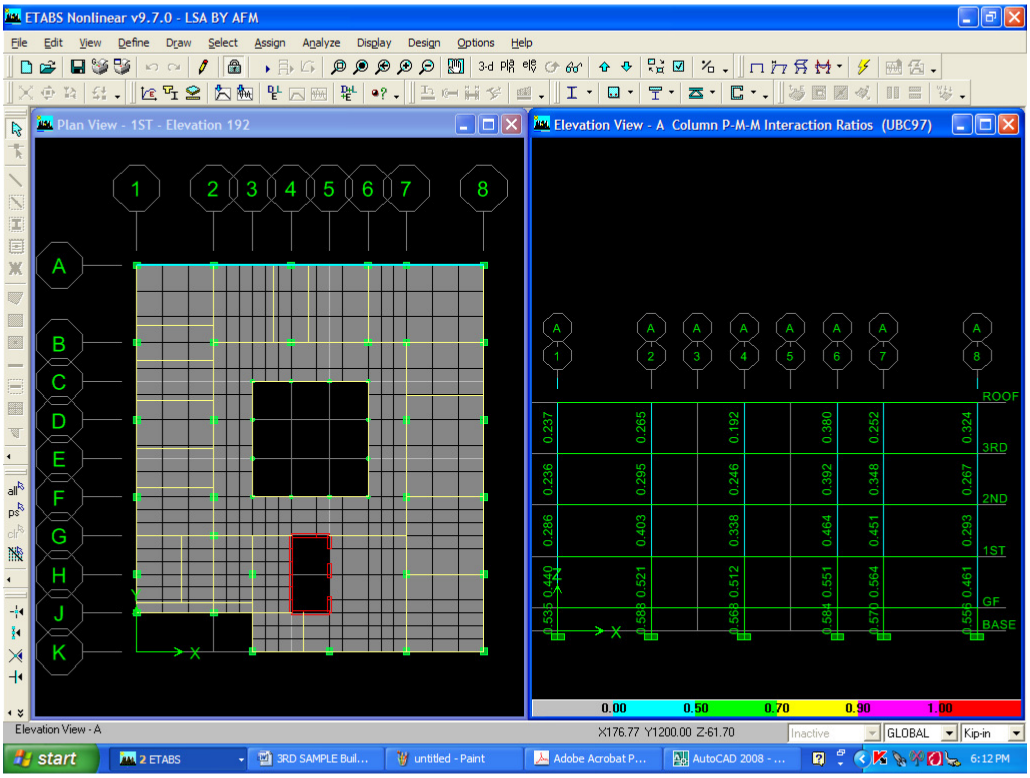
## Appendix A: Tier 1 Checklists

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

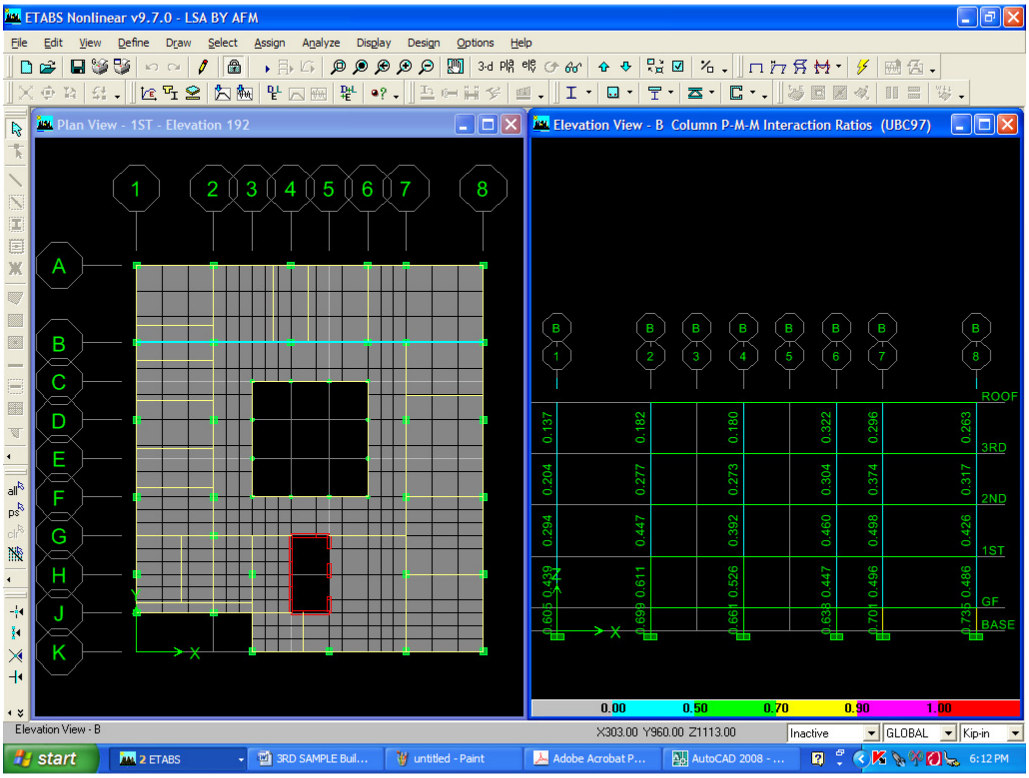
| LATERAL-FORCE RESISTING SYSTEM | |
|---|---|
| Redundancy | C |
| Wall Connections | C |
| Shear Stress Check | C |
| Axial Stress Check | C |
| flat Slab Frames | NA |
| Pre Stressed Frames | NA |
| Captive Column | C |
| No Shear Failure | C |
| Strong Columns/ Weak Beams | C |
| Beam Bars | C |
| Columns Bar Splices | C |

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

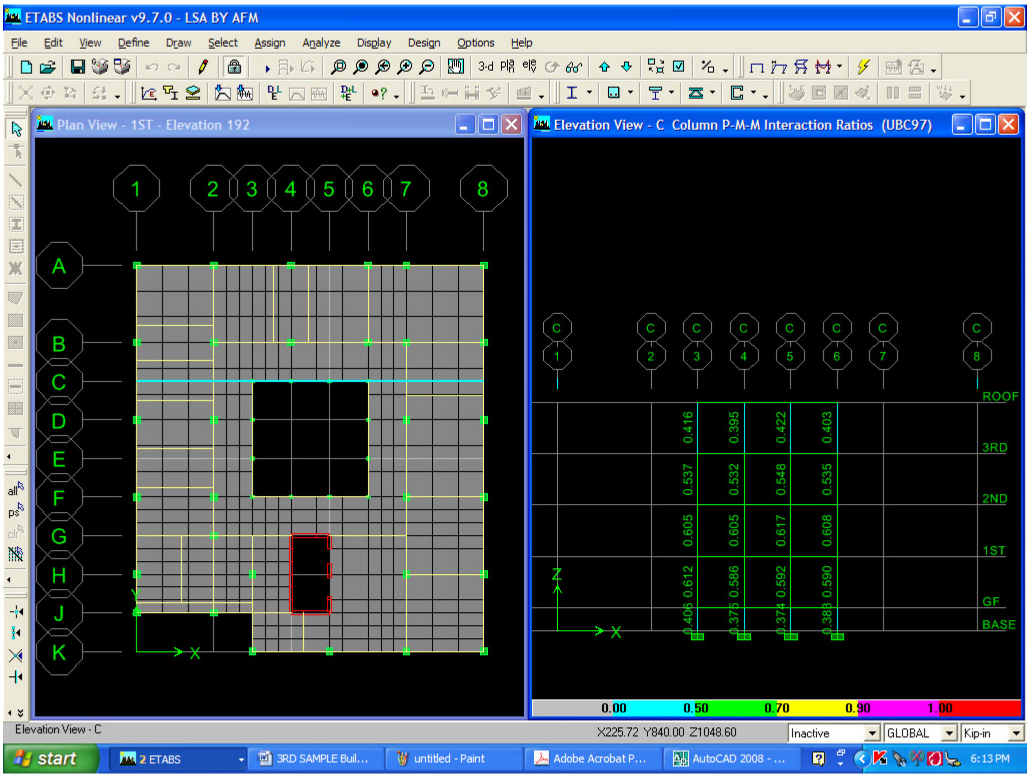
# Appendix B: Linear Analysis (Tier 2) Results



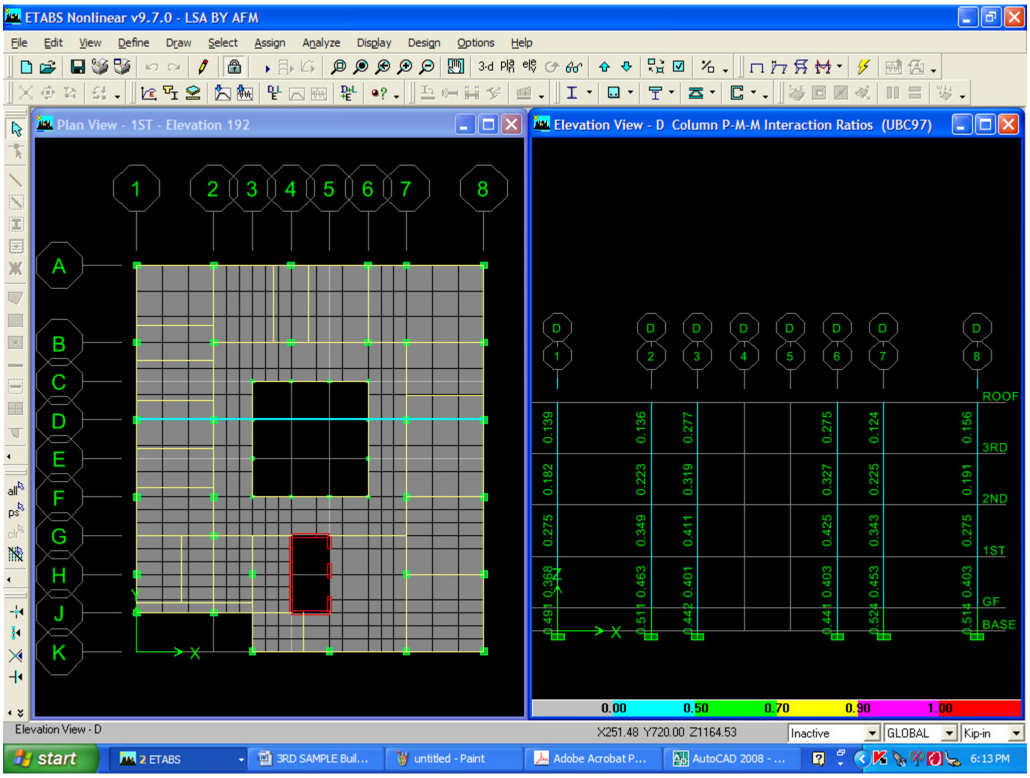

**Demand/Capacity Ratios for Frame at Grid-A**



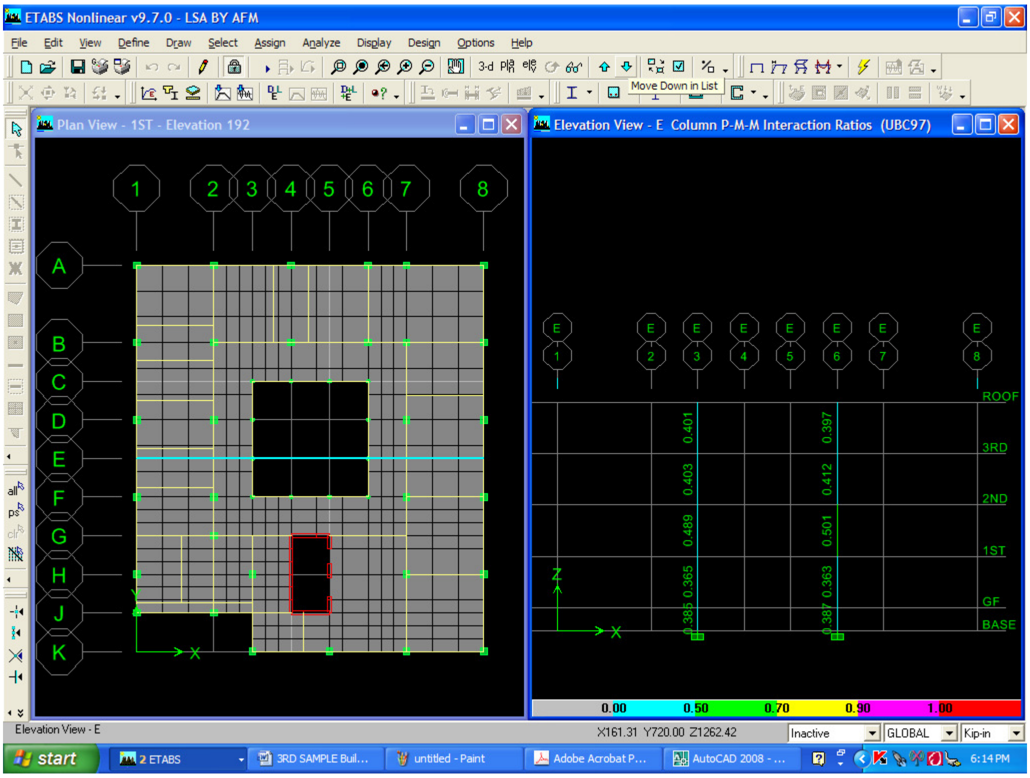

**Demand/Capacity Ratios for Frame at Grid-B**



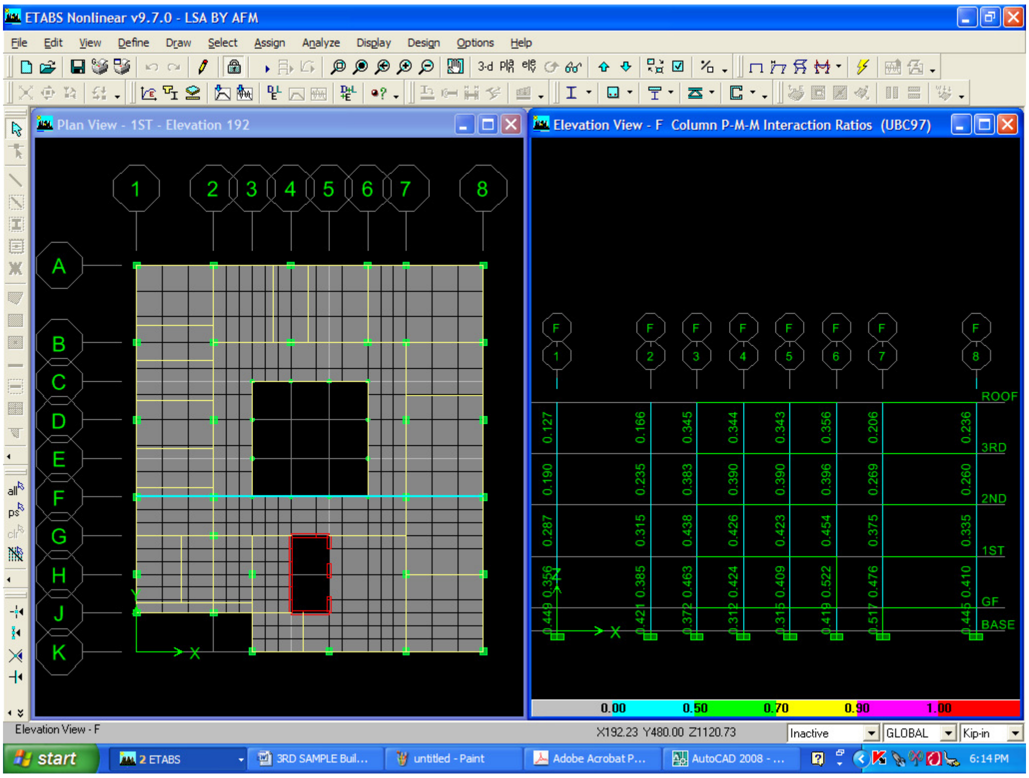
**Demand/Capacity Ratios for Frame at Grid-C**



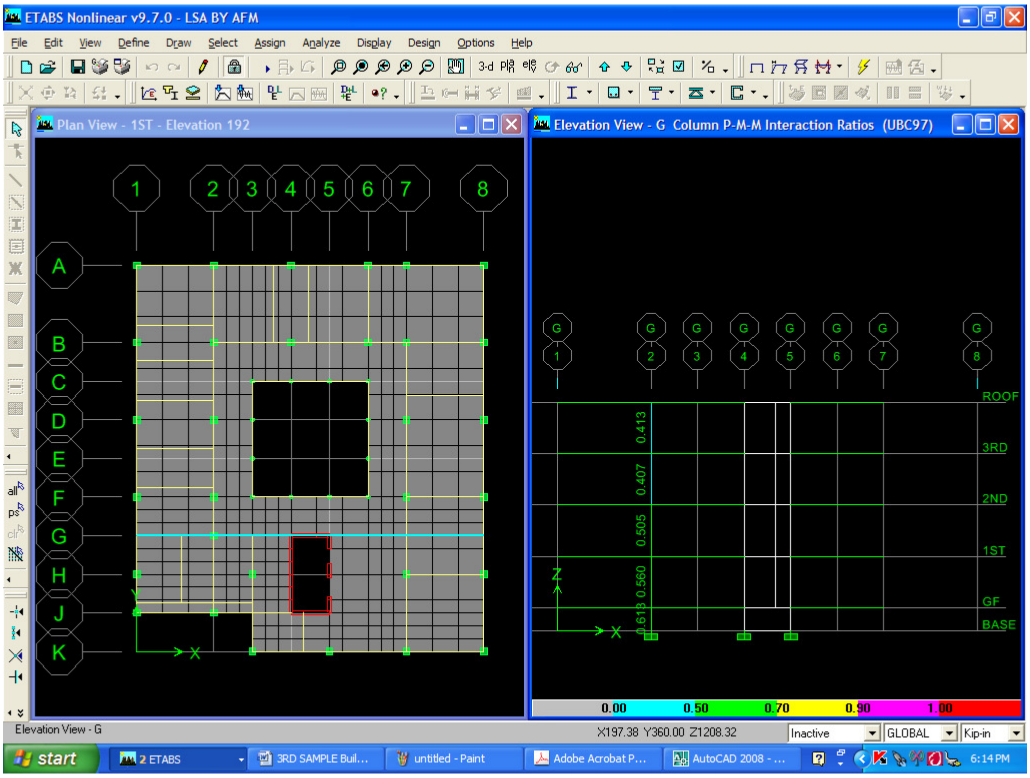
**Demand/Capacity Ratios for Frame at Grid-D**



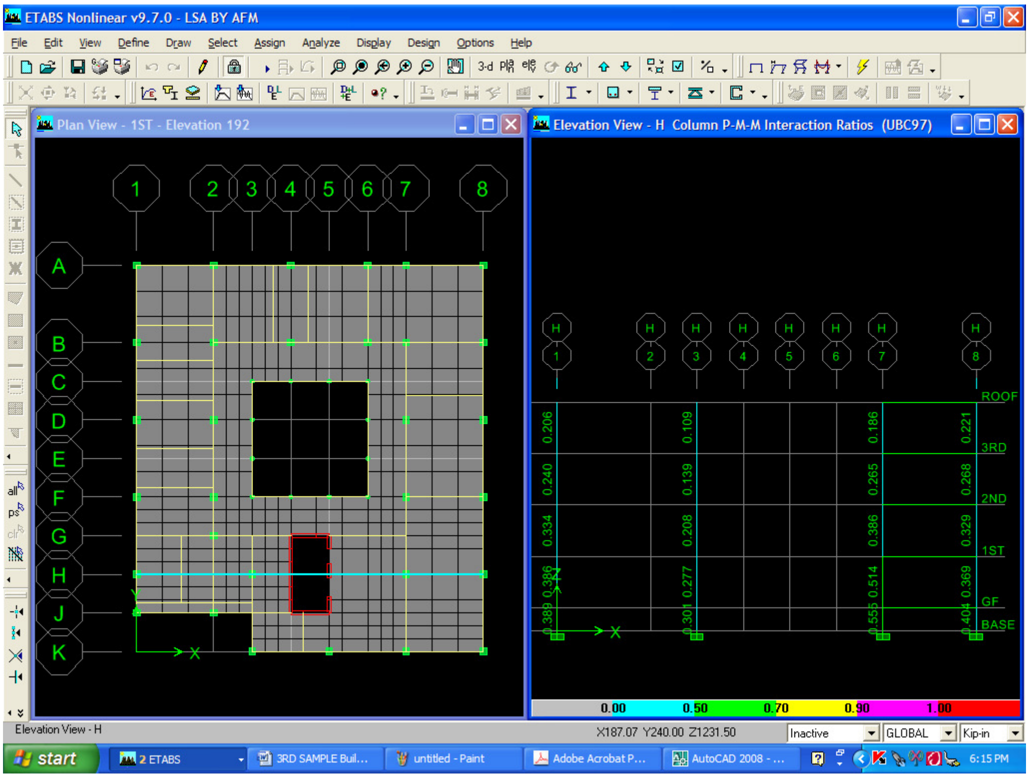

Demand/Capacity Ratios for Frame at Grid-E



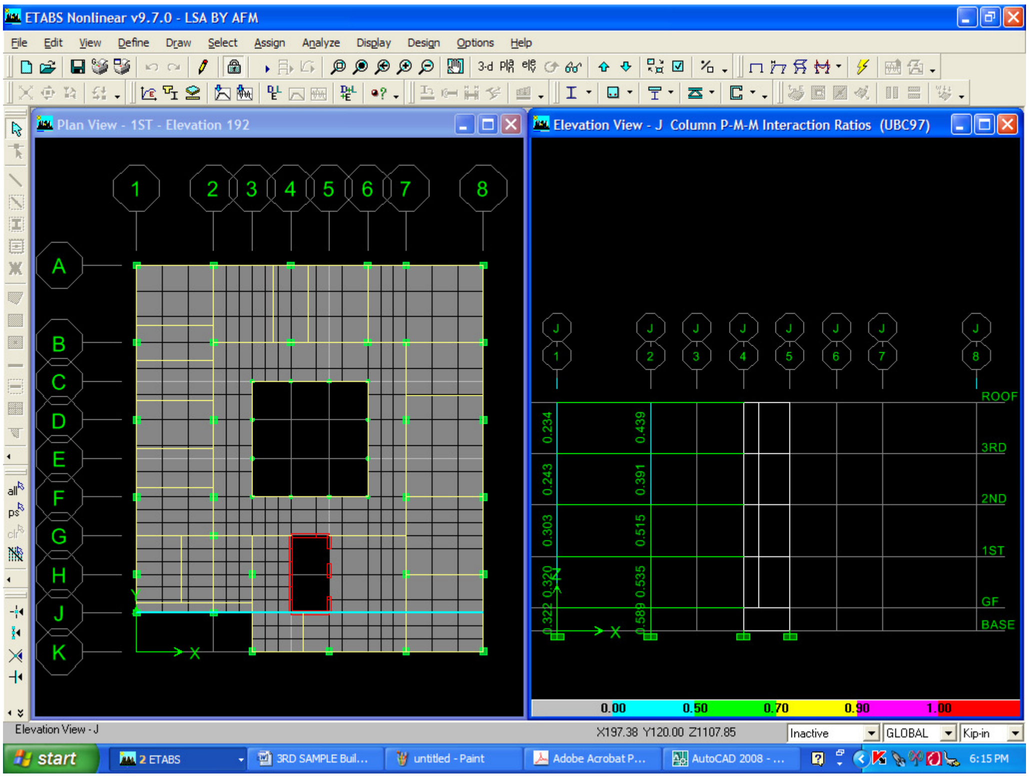

Demand/Capacity Ratios for Frame at Grid-F



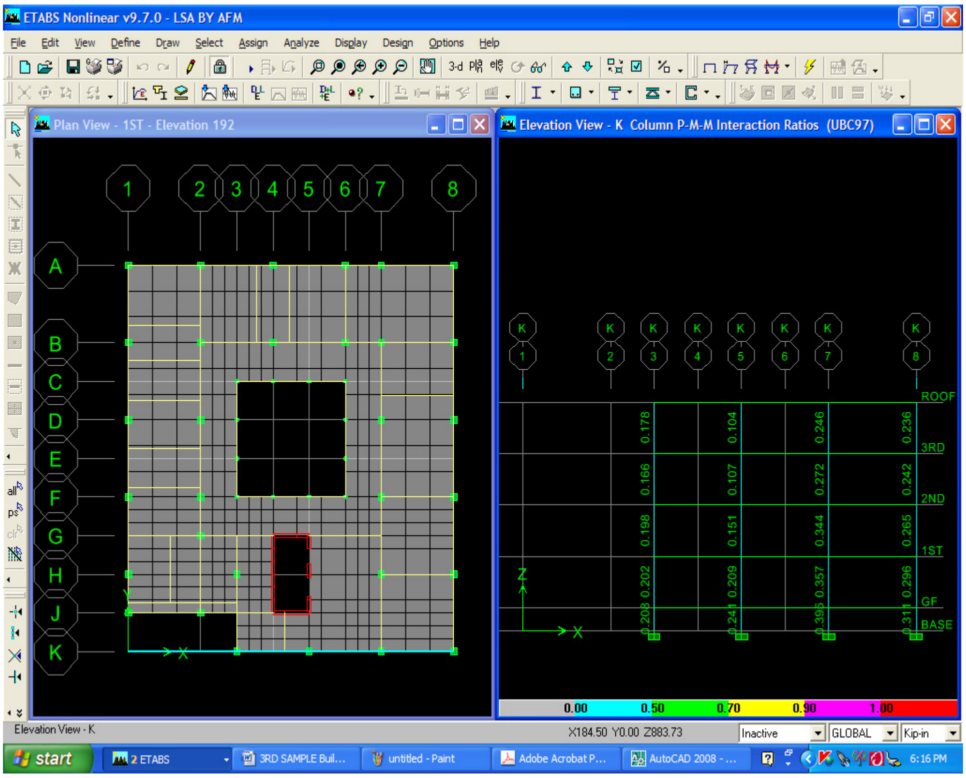
**Demand/Capacity Ratios for Frame at Grid-G**



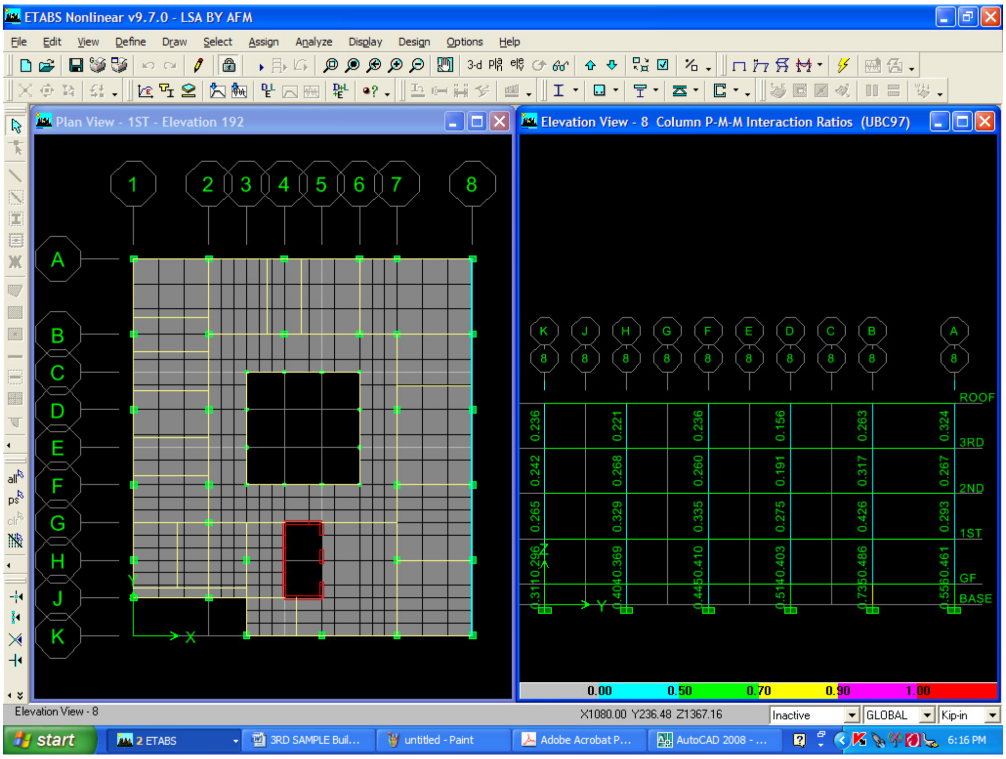
**Demand/Capacity Ratios for Frame at Grid-H**



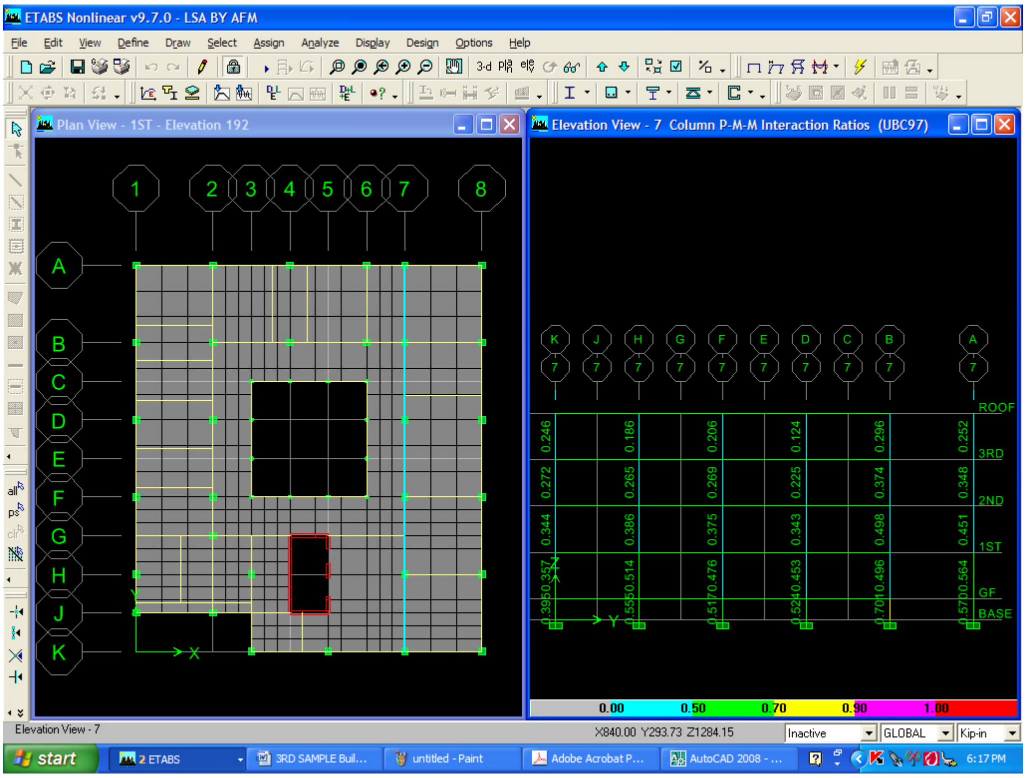

**Demand/Capacity Ratios for Frame at Grid-J**



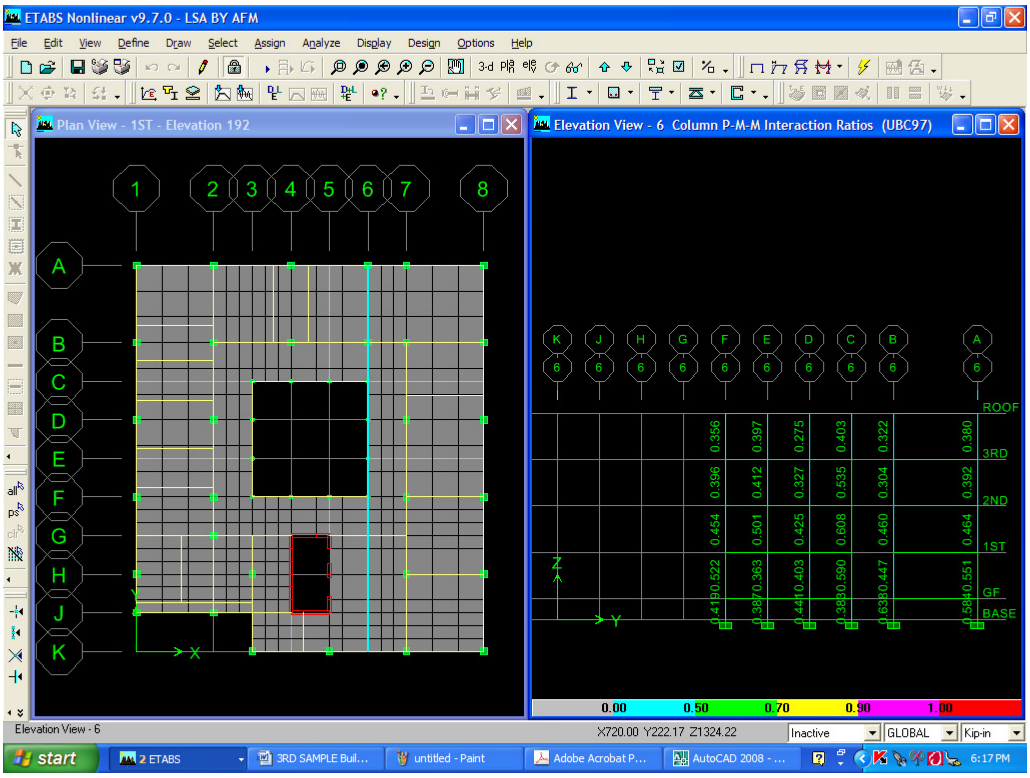

**Demand/Capacity Ratios for Frame at Grid-K**



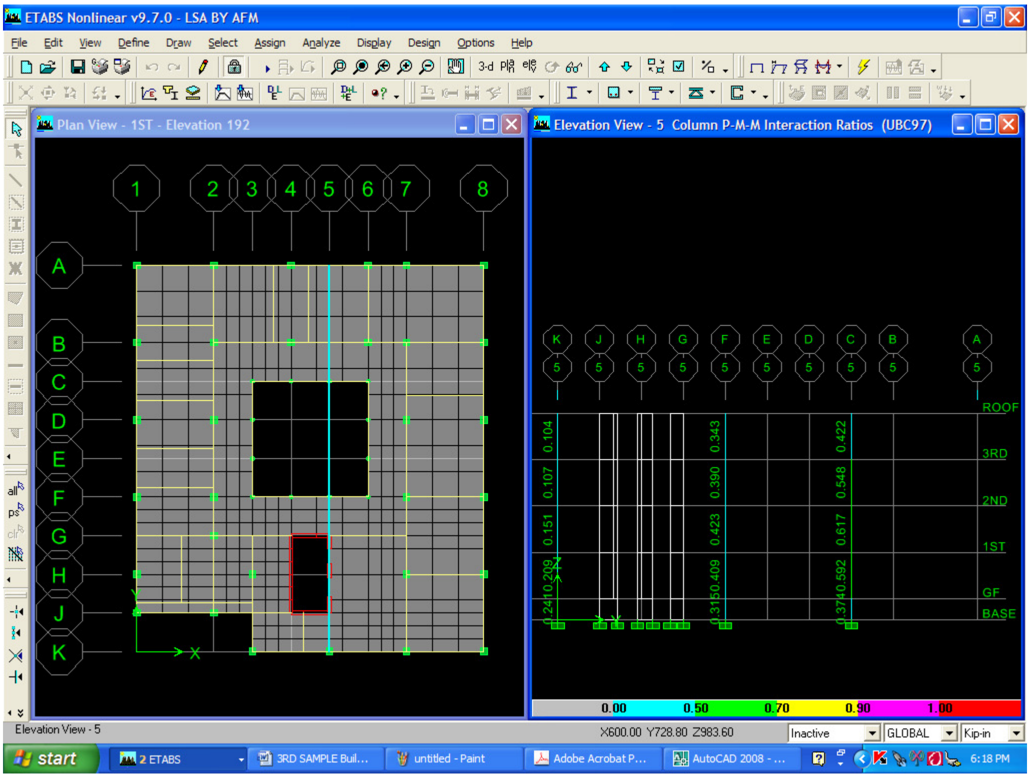

**Demand/Capacity Ratios for Frame at Grid-8**



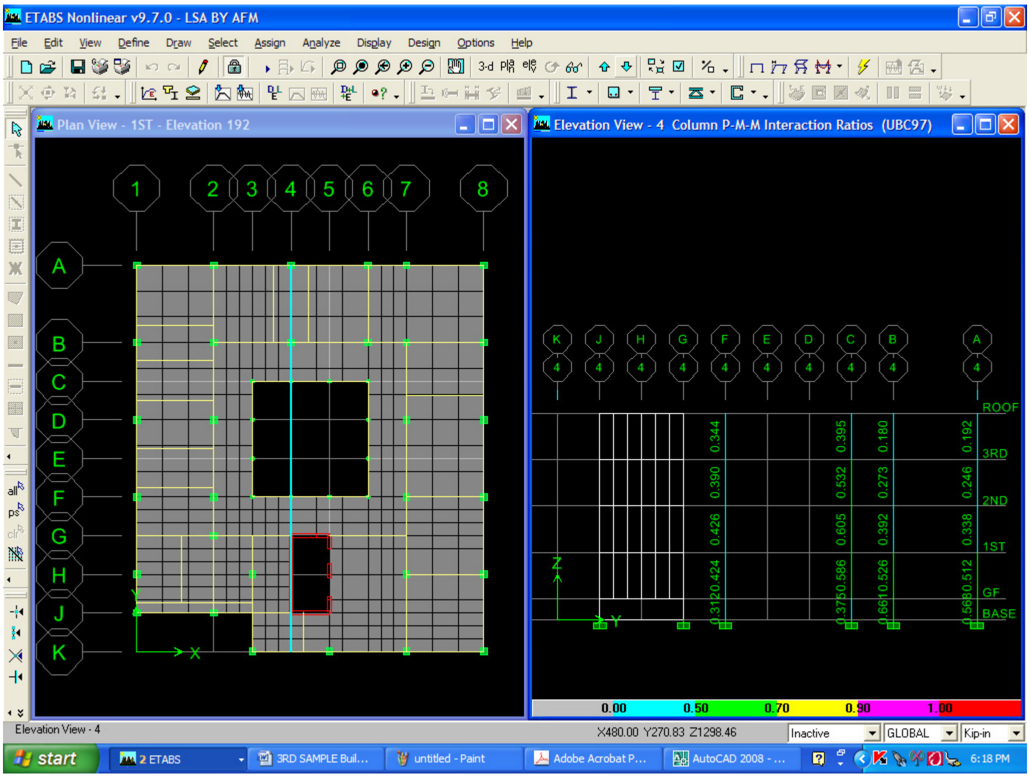

**Demand/Capacity Ratios for Frame at Grid-7**



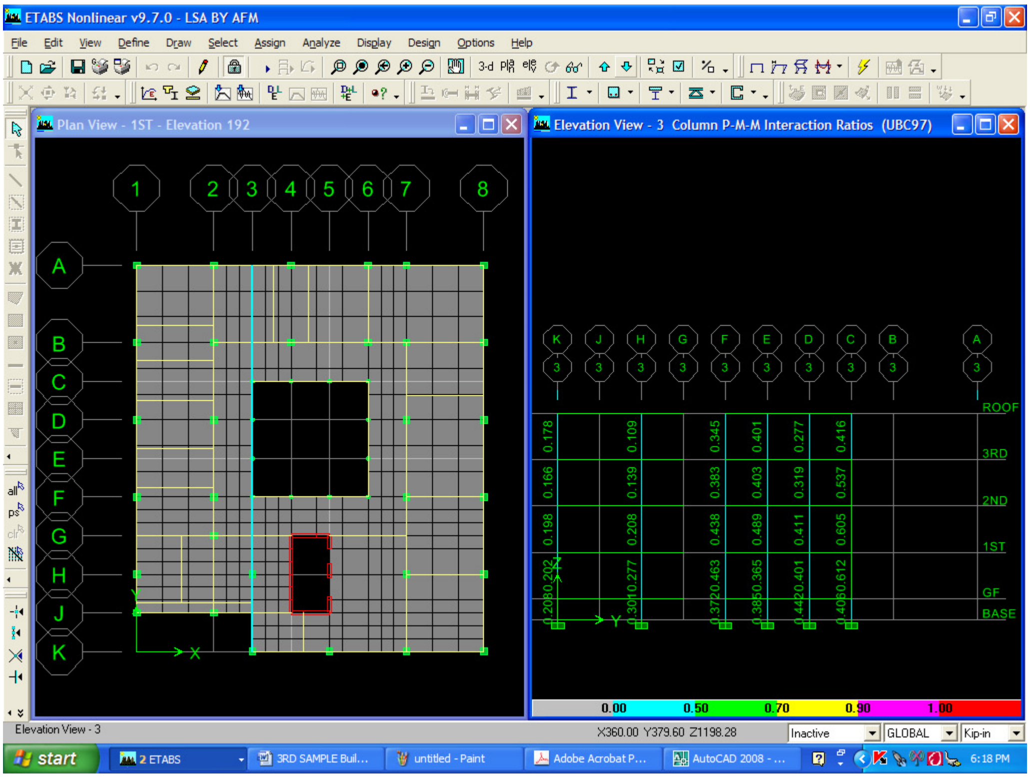

**Demand/Capacity Ratios for Frame at Grid-6**



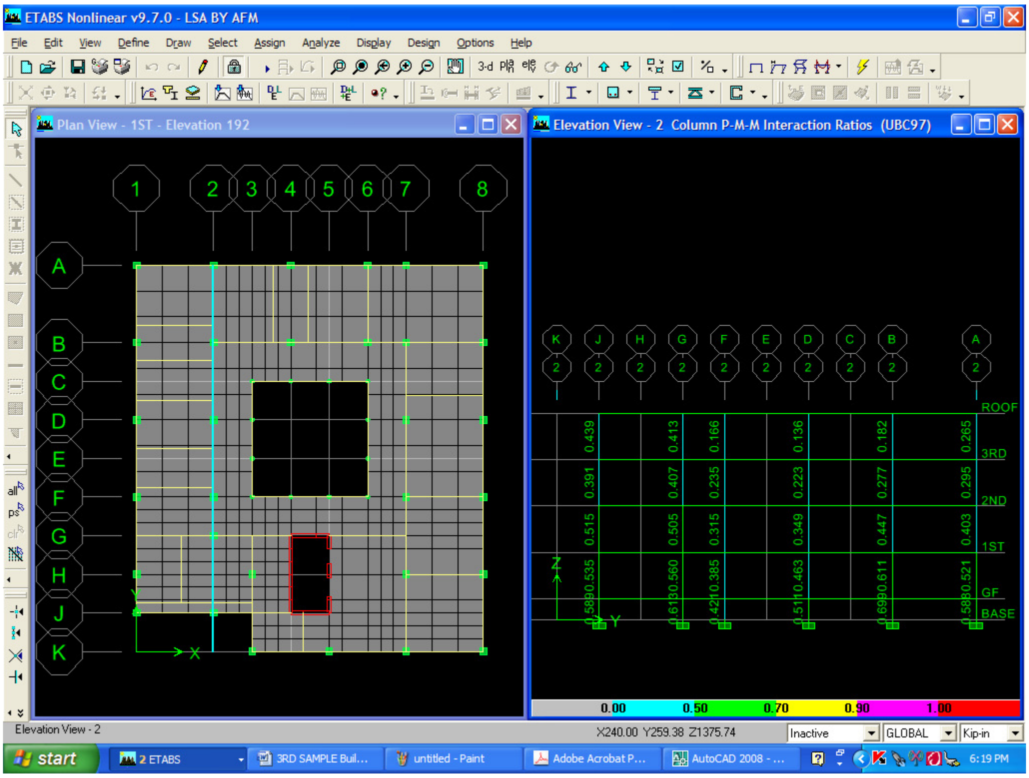

**Demand/Capacity Ratios for Frame at Grid-5**



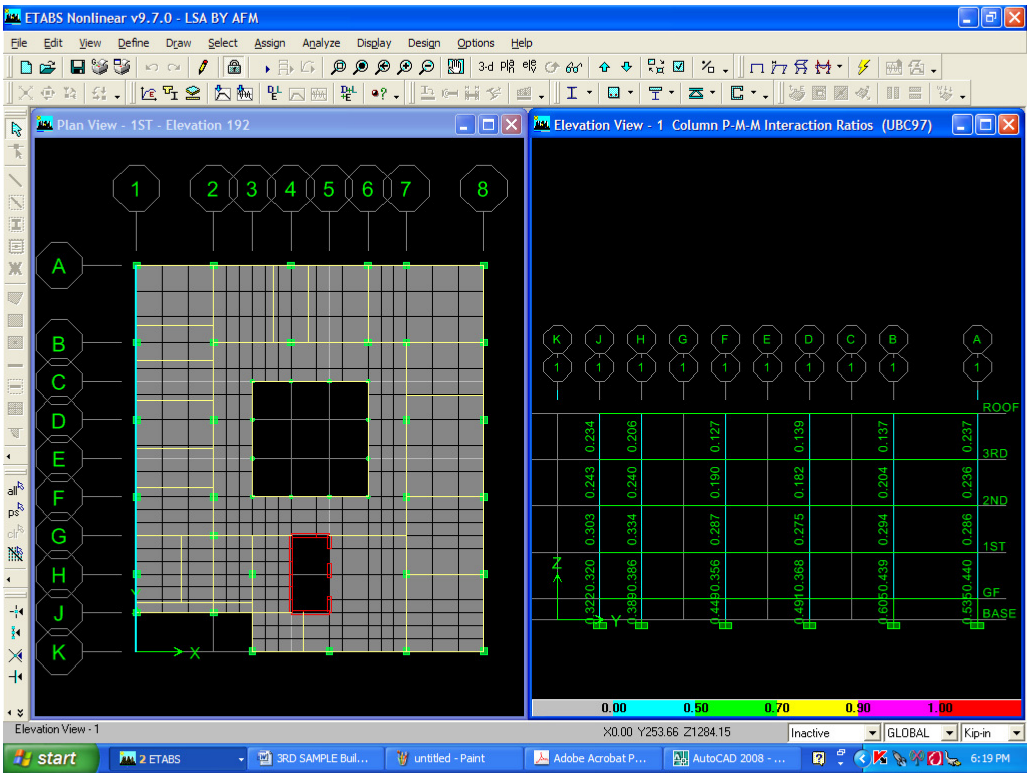

**Demand/Capacity Ratios for Frame at Grid-4**



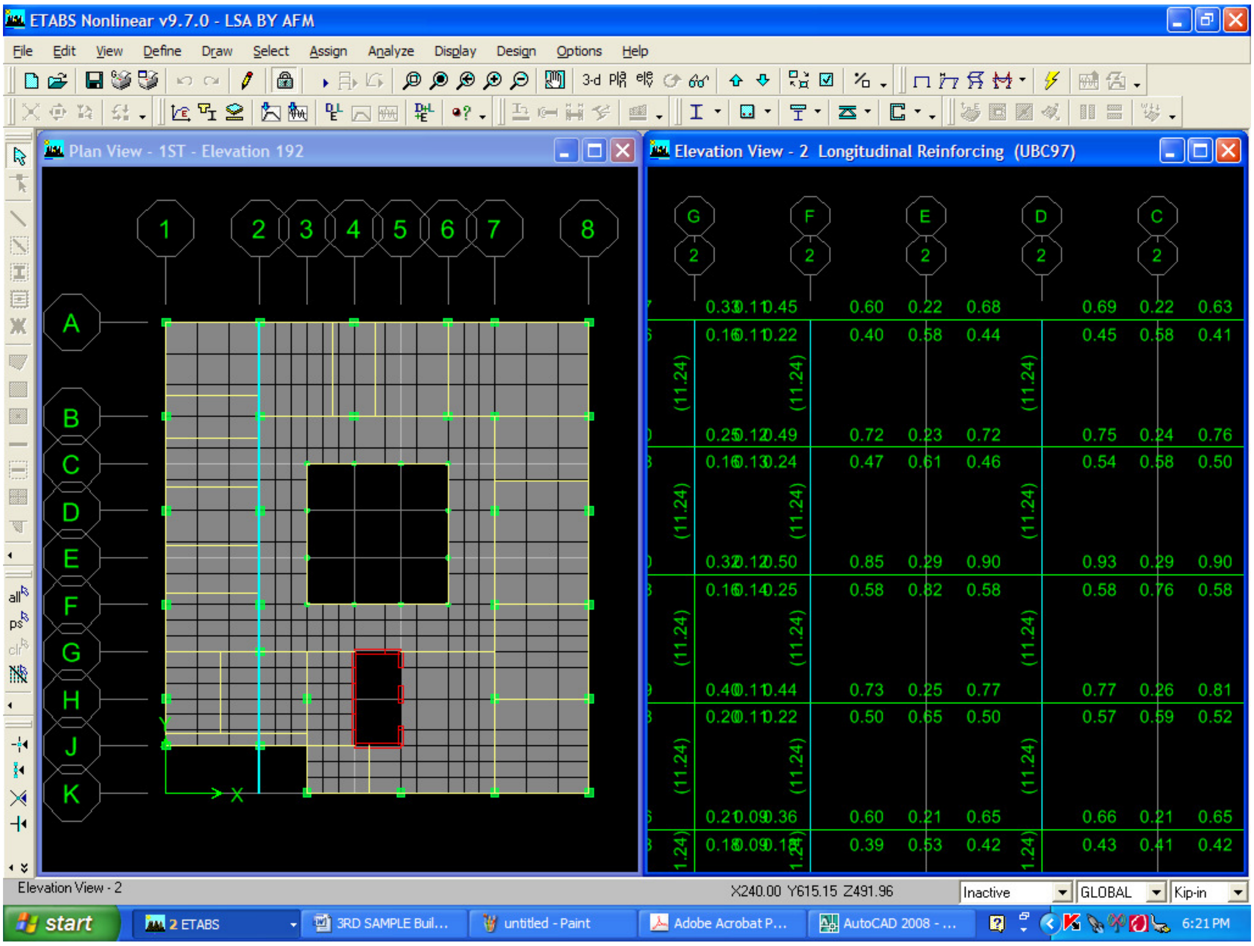

**Demand/Capacity Ratios for Frame at Grid-3**




**Demand/Capacity Ratios for Frame at Grid-2**




**Demand/Capacity Ratios for Frame at Grid-1**




**Required Flexural Reinforcement in Beams**