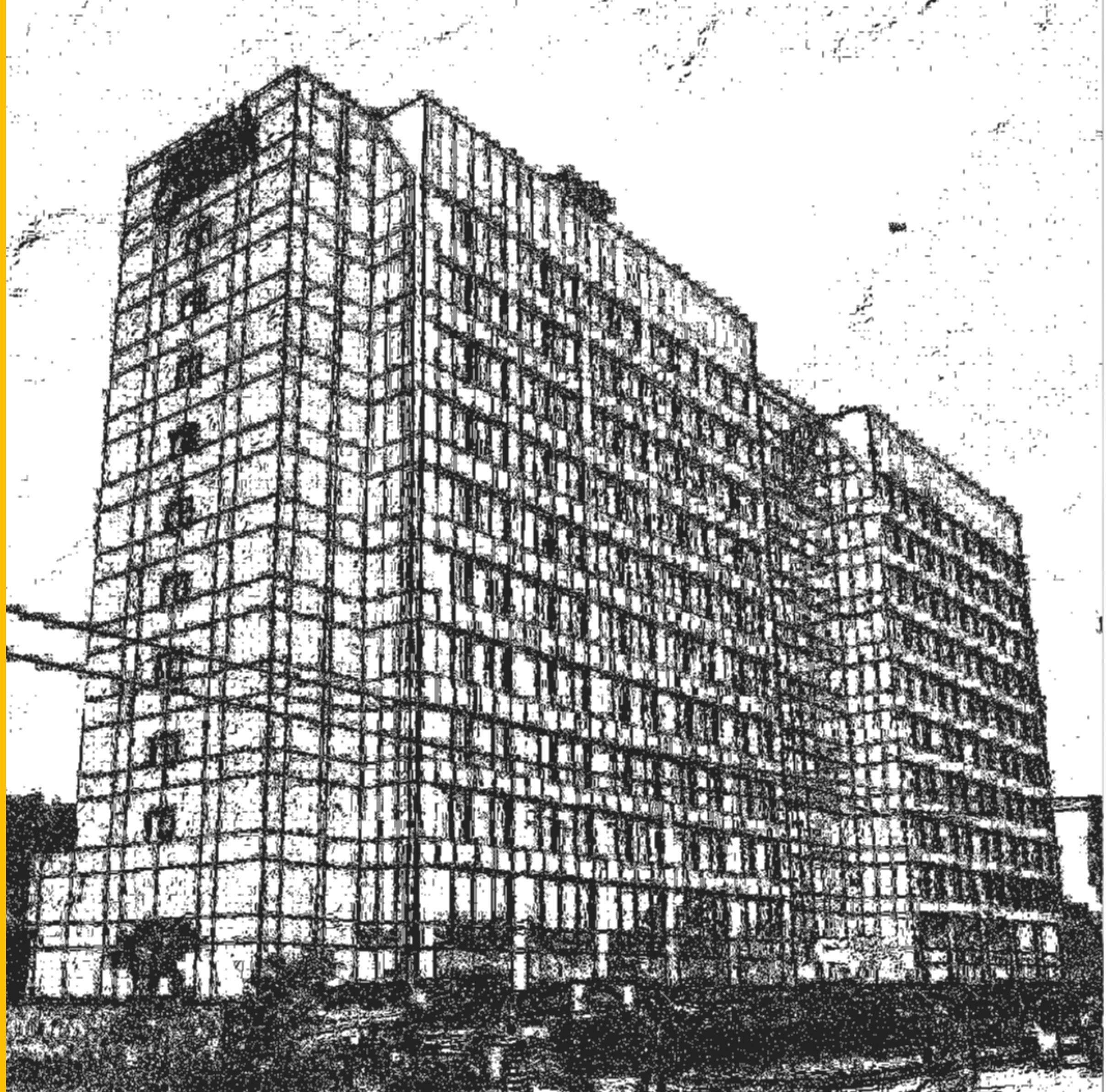

10-Storey Office Building in Karachi

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

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Summary

The building is located in a densely populated area in Karachi. It is a reinforced concrete framed building with ten storeys above ground and twelve storeys total, including two basements. The building is being used as an office building, therefore it is evaluated for the Life Safety (LS) level of seismic performance, meaning that its occupants should survive the design level earthquake and be able to exit the building safely. The reinforced concrete frame consists of flat slab with drop panel and having outer peripheral beams. The building construction was completed in 2004. Project participants selected this building as a case study because it has several potential seismic vulnerabilities common to high rise buildings in Karachi: a weak story created by open working areas, 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 building was found to be adequately designed. Some minor damage, which will not affect the stability of the building, may occur in some columns at the ends of the building. However, the building is expected to meet the Life Safety performance objective, and therefore no seismic retrofit is required.

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, and Ms. Maria Ansari, Lecturer, Department of Civil Engineering, NED University of Engineering and Technology; Mr. M. Anis Bilal, Engineering Manager, Structural Section, Mustaq & Bilal Associates; and Mr. Moinuddin Khan, Head, Structural Department, EA Consultants Pvt. Ltd.

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.

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Introduction

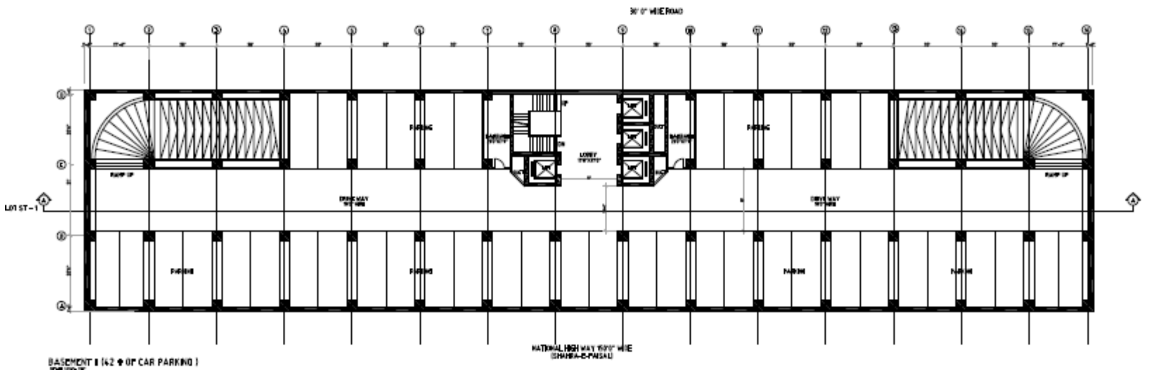
The Tier 1 vulnerability assessment exercise carried out by the team members gave them the opportunity to evaluate a real building with all the physical constraints. On the basis of the vulnerabilities found through the Tier 1 assessment, Tier 2 (linear static structural analysis) was carried out to assess the vulnerabilities and potential solutions in more detail. This gave the members the opportunity to do hands-on practice on ETABS and understand the ASCE/SEI 31-03 and FEMA documents.

Building Information

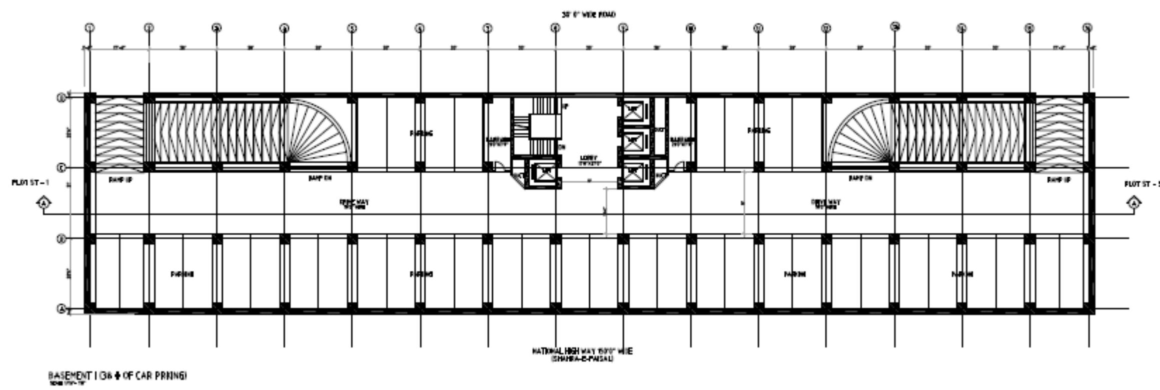
The building, shown in Figure 1, is a ten storey office building, with two basements, a ground floor and nine upper floors. The building's overall dimensions are 65'-0" wide by 301'-0" long, and it is approximately 90 feet tall from ground level. The building system consists of flat slabs with drop panels and outer peripheral beams. RCC wall lift cores are eccentrically placed at the back side of the building (way from the street shown below). The foundations are reinforced concrete isolated spread footings with retaining walls on the periphery. The building is relatively new and is in reasonably good condition. No condition assessments or repairs have been made.

Figure 1. Front elevation view

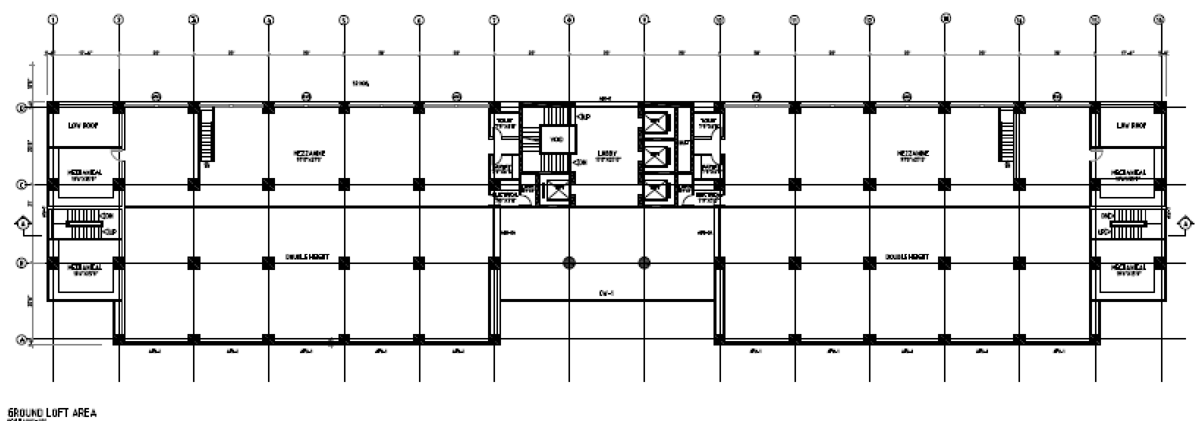
The building's architectural and structural drawings are shown in Figure 2 through Figure 6. Original design calculations show that the building was designed according to ACI-99 and earthquake analysis was carried out using UBC-97. Typical storey height is 10'-0". The strength of concrete and reinforcement are taken as: f'c = 4,000 psi (slabs, beams and columns (from 3rd floor to roof)), f'c = 5,000 psi (Columns up to 2nd floor), fy= 60,000 psi.



BASEMENT II ARCHITECTURAL PLAN

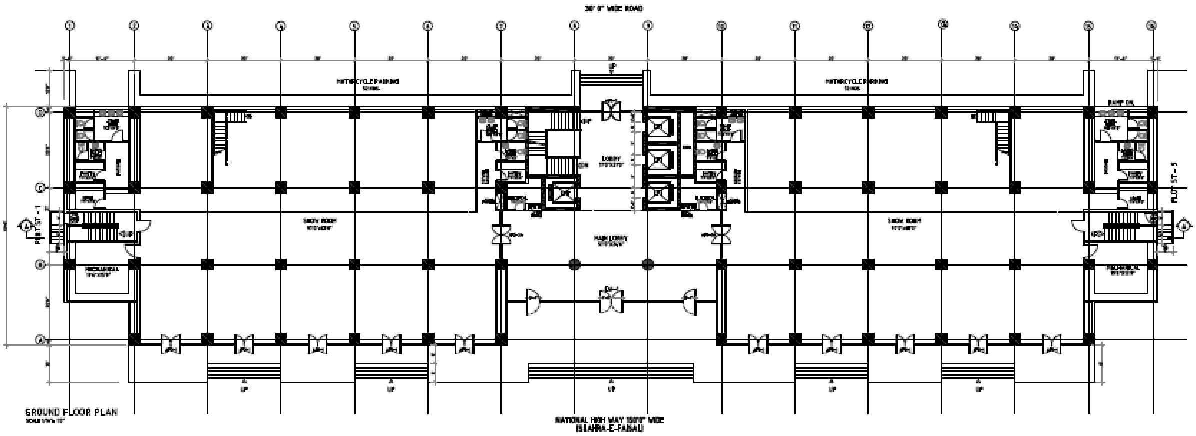


BASEMENT I ARCHITECTURAL PLAN

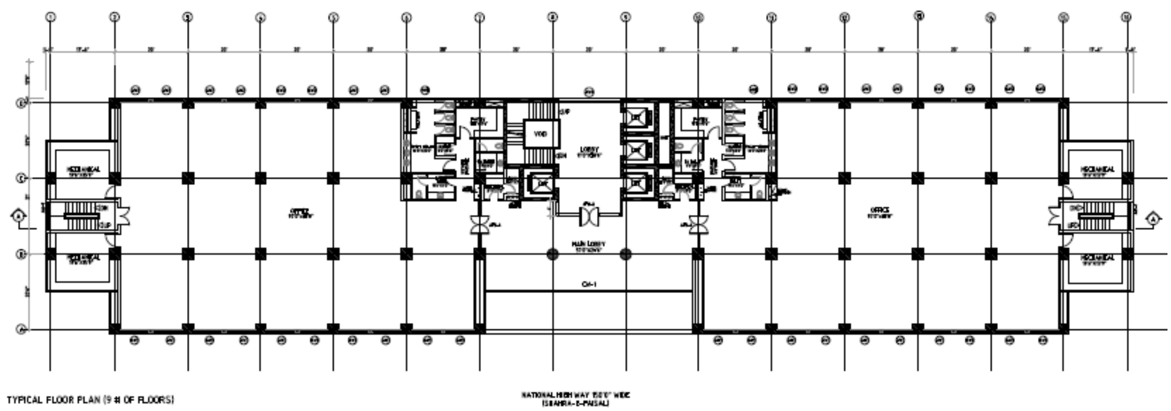


GROUND LOFT AREA ARCHITECTURAL PLAN

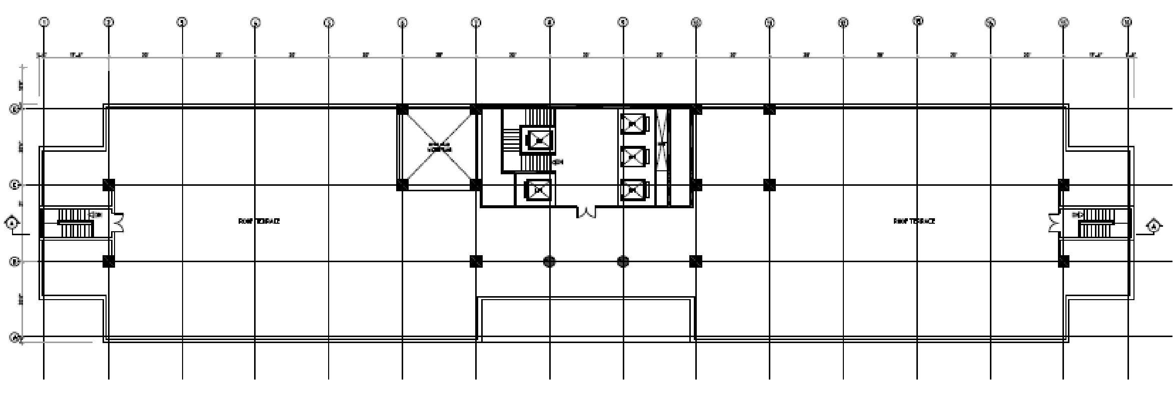
Figure 2. Architectural plans of the building: basement and ground loft areas



GROUND FLOOR ARCHITECTURAL PLAN

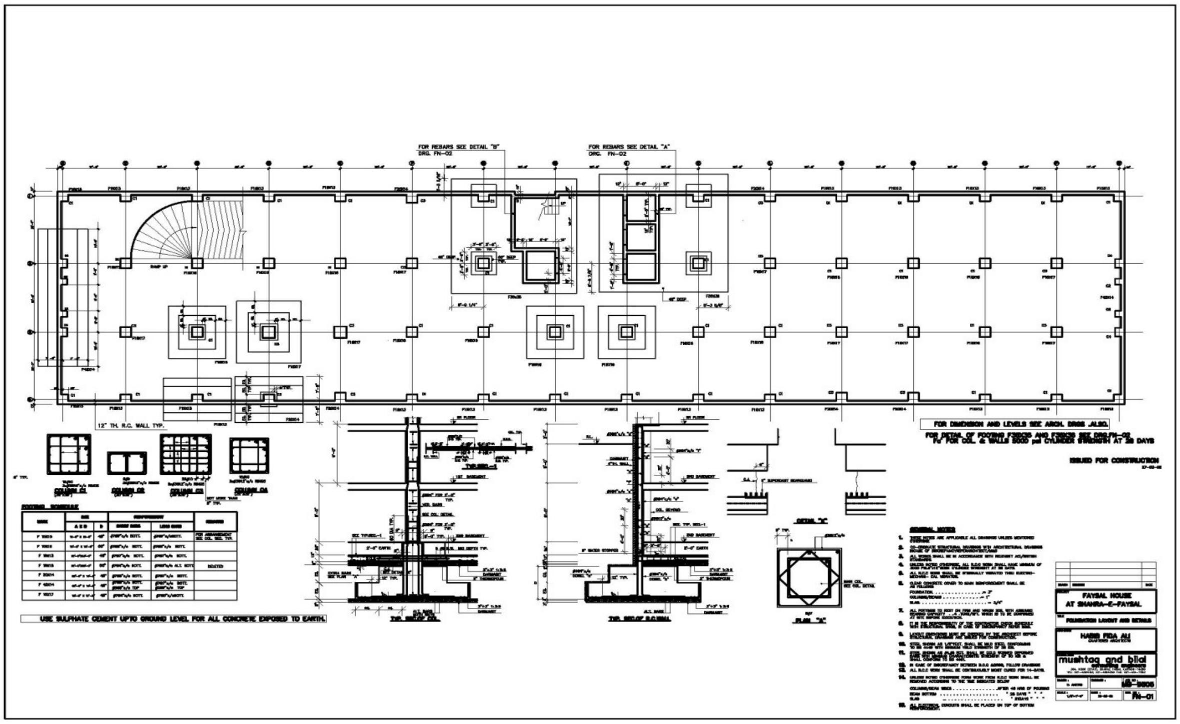


TYPICAL FLOOR ARCHITECTURAL PLAN



ROOF PLAN

Figure 3. Architectural plans of the building: ground floor, typical floor and roof



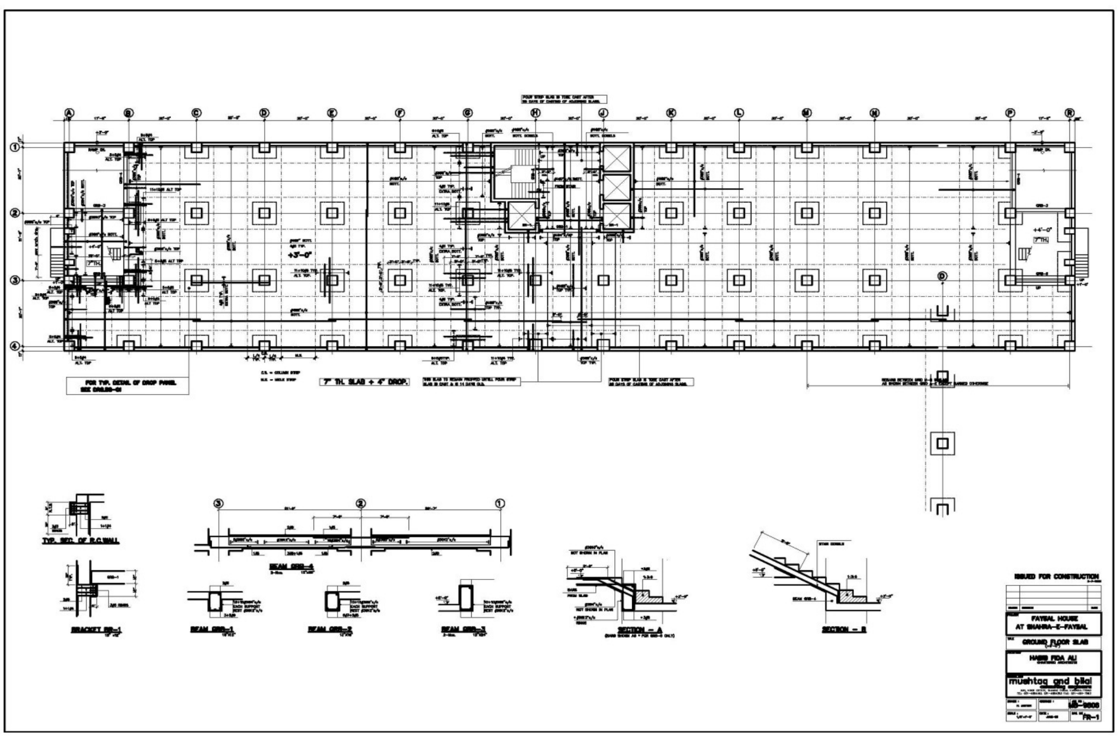


Figure 4. Structural framing plans: basement (top) and ground floor (bottom)

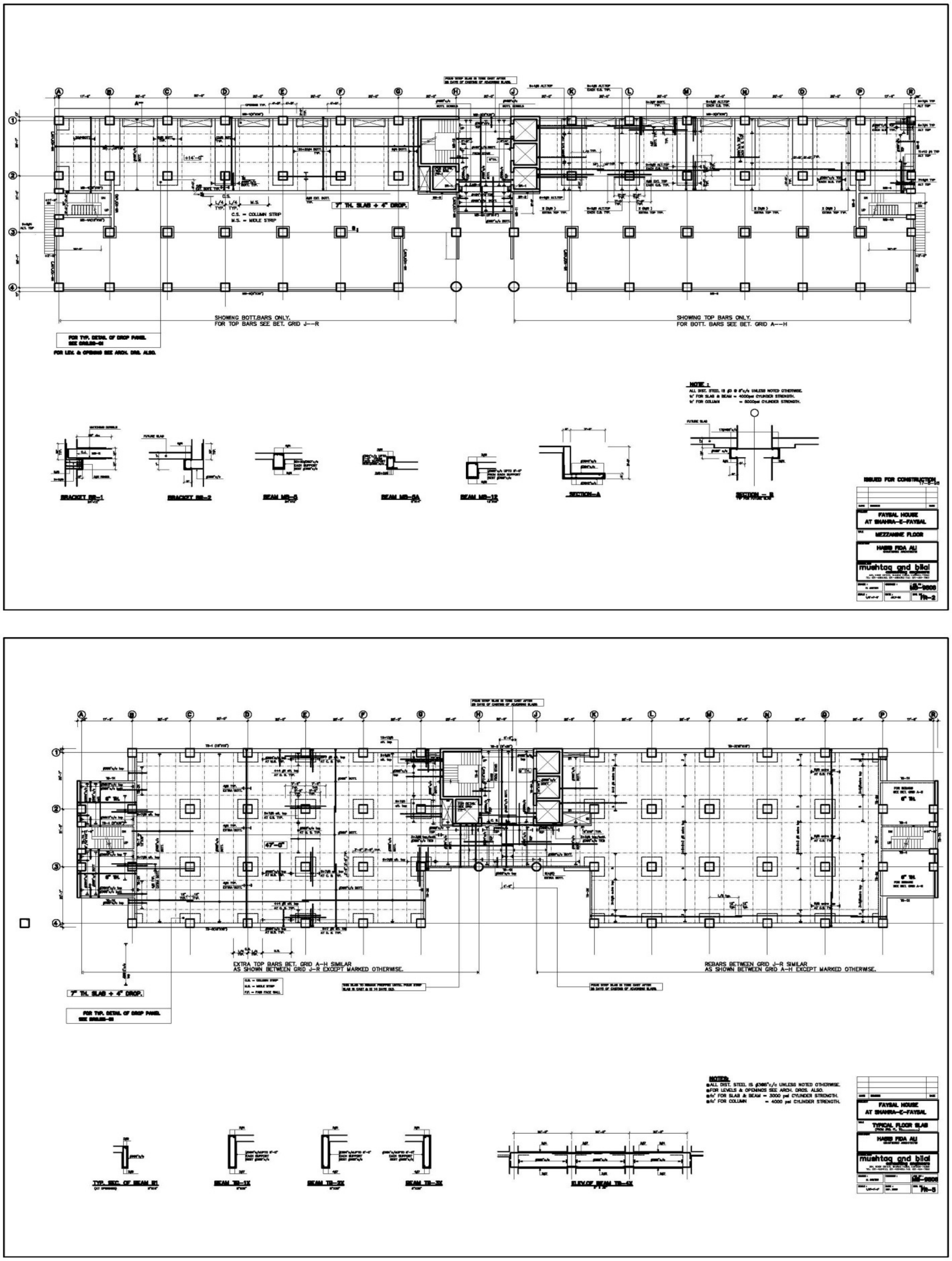


Figure 5. Structural framing plans: mezzanine (top) and typical floor (bottom)

MARK	f'c	C1 SIZE	C1 REBARS	C2 SIZE	C2 REBARS	C3 SIZE	C3 REBARS	C4 SIZE	C4 REBARS	C5 SIZE	C5 REBARS	C3A SIZE	C3A REBARS	C1A SIZE	C1A REBARS
ROOF															
9 TH. FLOOR		28"x28"	#3@12"c/c 12#10	18"x24"	#3@12"c/c 6#8	28"x28"	#3@12"c/c 20#10	26"x26"	#3@12"c/c 8#9			30"x28"	#3@12"c/c 24#10		
8 TH. FLOOR		30"x30"	#3@12"c/c 12#10	18"x24	#3@12"c/c 6#8	30"x30"	#3@12"c/c 20#10	28"x28"	#3@12"c/c 8#9			32"x30"	#3@12"c/c 24#10		
7 TH. FLOOR		30"x30"	#3@12"c/c 12#10	18"x26"	#3@12"c/c 8#8	30"x30"	#3@12"c/c 20#10	28"x28"	#3@12"c/c 8#9			32"x30"	#3@12"c/c 24#10		
6TH. FLOOR	4000 Psi.	32"x32"	#3@12"c/c 12#10	18"x26"	#3@12"c/c 8#8	32"x32"	#3@12"c/c 20#10	30"x30"	#3@12"c/c 12#9	SAME AS COL - C3	SAME AS COL - C3	34"x32"	#3@12"c/c 24#10	SAME AS COL - C3	SAME AS COL - C3
5TH. FLOOR		32"x32"	#3@12"c/c 12#10	18"x28"	#3@12"c/c 8#9	32"x32"	#3@12"c/c 20#10	30"x30"	#3@12"c/c 12#9			34"x32"	#3@12"c/c 24#10		
4 TH. FLOOR		34"x34"	#3@12"c/c 16#10	18"x28"	#3@12"c/c 8#9	34"x34"	#3@12"c/c 24#10	32"x32"	#3@12"c/c 12#9			36"x34"	#3@12"c/c 28#10		
3 RD. FLOOR		34"x34"	#3@12"c/c 16#10	18"x30"	#3@12"c/c 8#9	34"x34"	#3@12"c/c 24#10	32"x82"	#3@12"c/c 12#9			36"x34"	#3@12"c/c 28#10		
2ND. FLOOR		34"x34"	#3@12"c/c 16#10	18"x30"	#3@12"c/c 8#9	34"x34"	#3@12"c/c 24#10	32"x32"	#3@12"c/c 12#9	34		36"x34"	#3@12"c/c 28#10		
1ST. FLOOR		34"x34"	#3@12"c/c 16#10	18"x30"	#3@12"c/c 8#9	34"x34"	#3@12"c/c 24#10	32"x32"	#3@12"c/c 12#9	42"DIA	#3@12"c/c 24#10	36"x34"	#3@12"c/c 28#10		
MEZZ. FLOOR		36"x36"	#3@12"c/c 16#10	20"x34"	#3@12"c/c 10#9	36"x38"	#3@12"c/c 24#10	34"x34"	#3@12"c/c 12#10	42"DIA	#3@12"c/c 24#10	36"x38"	#3@12"c/c 28#10	36"x36"	#3@12"c/c 24#10
GR. FLOOR	5000 Psi.	36"x36	#3@12"c/c 16#10	20"x34"	#3@12"c/c 10#9	36"x38"	#3@12"c/c 24#10	34"x34"	#3@12"c/c 12#10	42"DIA	#3@12"c/c 24#10	36"x38"	#3@12"c/c 28#10	36"x36"	#3@12"c/c 24#10
BASEMENT-1		36"x36"	#3@12"c/c 24#10	20"x34"	#3@12"c/c 12#9	36"x38"	#3@12"c/c 32#10	34"x34"	#3@12"c/c 16#10	42"x42"	#3@12"c/c 24#10	36"x38"	#3@12"c/c 32#10	36"x36"	#3@12"c/c 24#10
BASEMENT-2		36"x36"	#3@12"c/c 24#10	20"x34"	#3@12"c/c 12#9	36"x38"	#3@12"c/c 32#10	34"x34"	#3@12"c/c 16#10	42"x42"	#3@12"c/c 24#10	36"x38"	#3@12"c/c 32#10	36"x36"	#3@12"c/c 24#10

Figure 6. Column reinforcement

Site Information

The building is located in an area with very dense soil and soft rock (Soil Profile Type is Sc). No known active faults pass through or near the site. The bearing capacity of the soil is 3.0 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=0.56. 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	Tier 1 Non-compliant Items
Building System	Torsion irregularity Mass irregularity Story drift Soft storey
Lateral Force-resisting System	Interfering wall Proportions of infill walls Flat slabs frames Beam bar splices Column tie spacing Joint reinforcement Joint eccentricity
Geologic Hazards and Foundation	Ties between foundation elements

Please see Appendix A for the full Tier 1 Checklist results.

Linear Evaluation

Figure 7 shows the 3-D model of the building generated in ETABS Nonlinear version 9.7.0. The peripheral 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 column strip approach was used to model typical floor beams (120"x7"). The 7 inch thick slab was modeled as membrane area element. An over strength factor (R) of 5.5 (for concrete building frame system having shear wall) was used for seismic analysis. The analysis results show that there are a number of columns with demand/capacity ratios (DCRs) greater than one, so the building is expected to respond in the nonlinear range. Please see Appendix B for linear analysis results.

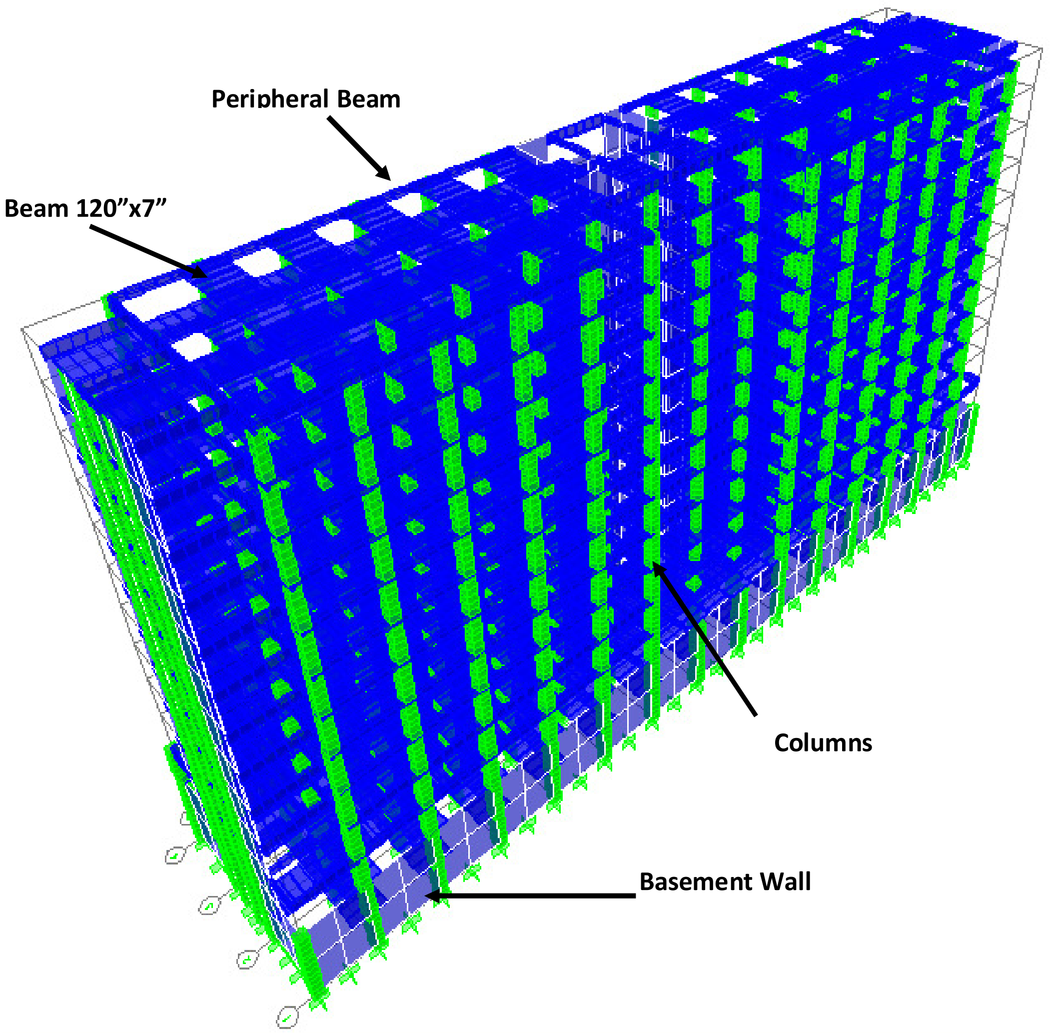


Figure 7. Rendering of linear ETABS model of the building

The team also conducted the other checks mandated in ASCE 31 for Tier 2 analysis based on the Tier 1 Checklist results. Despite using a modified FEMA 310 Tier 1 Checklist there was enough correspondence between items in the ASCE 31 Tier 1 Checklist and the modified FEMA 310 checklist to use ASCE 31’s Tier 2 checks directly. For this building, the required Tier 2 checks were for torsion irregularity (shown in Table 1), mass irregularity (shown in Table 2), soft storey (shown in Table 3) and storey drift (shown in Table 4).

Table 1. Torsion irregularity check

Story	Diaphragm	XCM (in.)	YCM (in.)	XCR (in.)	YCR (in.)	% diff. X (20% Allowed)	% diff. Y (20% Allowed)
ROOF	D1	1757.262	375.293	1731.063	442.105	1.51	15.11
NINE	D1	1750.652	381.595	1731.582	446.308	1.10	14.50
EIGHT	D1	1750.821	381.437	1732.175	449.16	1.08	15.08
SEVENTH	D1	1750.447	381.117	1732.69	451.412	1.02	15.57
SIXTH	D1	1750.503	380.419	1733.234	454.206	1.00	16.25
FIFTH	D1	1750.515	380.246	1733.974	457.71	0.95	16.92
FOURTH	D1	1750.114	380.404	1734.931	462.058	0.88	17.67
THIRD	D1	1750.876	380.209	1736.481	467.42	0.83	18.66
SECOND	D1	1750.783	384.87	1739.452	475.343	0.65	19.03
FIRST	D1	1751.016	383.957	1741.78	474.706	0.53	19.12
MEZZNIANE	D1	1758.418	484.991	1746.738	442.07	0.67	9.71
GROUND	D1	1751.49	368.228	1751.726	360.157	0.01	2.24
BASEMENT-1	D1	1751.658	357.595	1750.267	361.322	0.08	1.03

From the above data it is clear that there is no torsion irregularity.

Table 2. Mass irregularity check

Story	MassX (k)	MassY (k)	Mass < 150% of below story	Mass < 150% of above story
ROOF	12.7014	12.7014	OK	-
NINE	10.8385	10.8385	OK	OK
EIGHT	10.9089	10.9089	OK	OK
SEVENTH	10.9842	10.9842	OK	OK
SIXTH	11.0509	11.0509	OK	OK
FIFTH	11.1201	11.1201	OK	OK
FOURTH	11.2366	11.2366	OK	OK
THIRD	11.2265	11.2265	OK	OK
SECOND	11.3066	11.3066	OK	OK
FIRST	11.3552	11.3552	OK	OK
MEZZNIANE	11.5241	11.5241	OK	OK
GROUND	13.4101	13.4101	OK	OK
BASEMENT-1	16.5975	16.5975	-	OK

Heavy mass on roof

There is a heavy mass on the roof, but it is not more than 150% of the mass in the story below, so there is no mass irregularity.

Table 3. Soft storey check

Story	Load	Story Force kips	Total Displacement inches	Stiffness K kip/in	Soft Story Check: $K_{Below} < 0.7 \times K_{Above}$	Soft Story Check: $K_{Below} < 0.8 \times K$ Avg.of 3 above stories
ROOF	EX	864.01	7.734	111.72		-
NINE	EX	389.05	6.9639	55.87	SOFT STORY	-
EIGHT	EX	355.88	6.1646	57.73	OK	-
SEVENTH	EX	322.51	5.3392	60.40	OK	OK
SIXTH	EX	288.43	4.4966	64.14	OK	OK
FIFTH	EX	253.95	3.6487	69.60	OK	OK
FOURTH	EX	219.95	2.8171	78.08	OK	OK
THIRD	EX	182.6	2.0258	90.14	OK	OK
SECOND	EX	146.74	1.3077	112.21	OK	OK
FIRST	EX	111.26	0.7042	157.99	OK	OK
MEZZNIANE	EX	47.03	0.2509	187.45	OK	OK
GROUND	EX	43.81	0.0128	3422.66	OK	OK

Story	Load	Story Force kips	Total Displacement in.	Stiffness K kip/in	Soft Story Check: $K_{Below} < 0.7 \times K_{Above}$	Soft Story Check: $K_{Below} < 0.8 \times K$ Avg.of 3 above stories
ROOF	EY	923.92	3.1671	291.72		-
NINE	EY	435.86	2.8214	154.48	SOFT STORY	-
EIGHT	EY	398.7	2.4694	161.46	OK	-
SEVENTH	EY	361.31	2.1149	170.84	OK	OK
SIXTH	EY	323.13	1.7632	183.26	OK	OK
FIFTH	EY	284.5	1.4192	200.47	OK	OK
FOURTH	EY	246.41	1.0907	225.92	OK	OK
THIRD	EY	204.58	0.7847	260.71	OK	OK
SECOND	EY	164.38	0.5163	318.38	OK	OK
FIRST	EY	124.66	0.3121	399.42	OK	OK
MEZZNIANE	EY	52.68	0.1514	347.95	OK	OK
GROUND	EY	49.09	0.0506	970.16	OK	OK

The above data show that a soft storey may exist on the 9th floor.

Table 4. Storey drift check

Story	DriftX	Code Modified	DriftY	Code Modified
	Δs	ΔM = 0.7RΔs	Δs	ΔM = 0.7RΔs
ROOF	0.005377	0.0207	0.002505	0.0096
NINE	0.005607	0.0216	0.002567	0.0099
EIGHT	0.005799	0.0223	0.002592	0.0100 Max.
SEVENTH	0.005922	0.0228 Max.	0.002577	0.0099
SIXTH	0.005962	0.0230	0.002525	0.0097
FIFTH	0.005849	0.0225	0.002411	0.0093
FOURTH	0.005567	0.0214	0.002241	0.0086
THIRD	0.005043	0.0194	0.001942	0.0075
SECOND	0.004218	0.0162	0.001443	0.0056
FIRST	0.003166	0.0122	0.001133	0.0044
MEZZNIANE	0.00167	0.0064	0.000708	0.0027
GROUND	0.000067	0.0003	0.000237	0.0009
BASEMENT-1	0.000023	0.0001	0.000116	0.0004

The allowable drift value is 0.025 as per UBC 97 for a fundamental time period (T_a = 1.247 sec). Therefore computed drifts do not exceed the allowable.

Detailed Evaluations of Existing Building

The linear static analysis of this building and the checks for the building system required per ASCE 31 Tier 2 (mass irregularities, torsion etc.) based on the non-compliant items from the Tier 1 visual inspection, showed that all checks came out to be compliant, except for the soft story check. It is also observed that all columns connected to slab directly have DCR < 1. Similarly, internal columns in the end framing bents are failing but they are connected to stairs and/or peripheral beams (see structural drawings) and their demand/capacity ratio (DCR) is greater than 1 but less than 2. For these reasons, nonlinear analysis was deemed unnecessary, and only simple hand calculation checks for the punching shear capacity were performed during the detailed analysis phase.

Hand Calculation Checks

Building system is consisting of flat slab system which according to ASCE 31-03 Tier 2: Sec.4.4.1.4.3 is not recommended; therefore punching shear capacity of the flat slab and slab-column connections are required to be checked.

Required Maximum reinforcement has been observed at @ Roof Level:

Required maximum reinforcement at top of beam = 6.682 in^2

Required maximum reinforcement at bottom of beam = 5.813 in2

Provided reinforcement at support (4+4 #8) = 6.284 in2

Provided reinforcement at mid span (4+4 #8) = 6.284 in2

Demand capacity ratio at top of the beam = 6.682/6.284 = 1.0633 which is slightly greater than 1

Demand capacity ratio at bottom of the beam = 5.813/ 6.284 = 0.925 which is less than 1

These results are satisfactory.

Results Summary

Following conclusions can be made from this seismic evaluation:

1. The building was originally designed for Seismic Zone 2B as per UBC. In general performance of this building seems sufficient and building seems stable to resist seismic forces.

2. The building structural system is a flat slab system, which according to ASCE 31-03 Tier 2: Sec.4.4.1.4.3 is not recommended; therefore punching shear capacity of the flat slab and slab-column connections are required to be checked. The building's flat slab system has been designed according to the shear design provisions of ACI 318 code. Punching shear provisions were followed to estimate shear strength and to provide necessary shear reinforcement in flat slab system. Punching shear capacity of one of the internal columns has been checked manually and the result is satisfactory.

3. According to the framing drawings, the bottom flexural bars in slabs are not passing through the columns and are extended up to the center of the column; however top reinforcement is passing through the column up to a length of L/4 on either side to avoid punching.

4. According to linear static analysis of this sample building, all columns connected to slab directly have demand/capacity ratio (DCR) < 1. Internal columns in the end framing bents have DCRs greater than 1, but they are connected to stairs and/or peripheral beams (please refer structural drawings) rather than the slab and have DCRs less than 2.

5. Demand capacity ratios are slightly greater than 1 for a small number of exterior columns at second floor level where a shear wall terminates. This shear wall extends from ground to below 2nd floor. Also many columns on grid A above the shear walls have DCRs greater than 1, but these columns are small. Most other columns are okay.

6. Beams seem okay - demand capacity ratios are just over 1 at the roof.

7. Building seems okay except for columns at ends. The low level of nonlinear behavior means that there is unlikely to be a problem for building stability.

Retrofit Solution

Conceptual Solutions Considered

The damage to columns having 1< DCR<2 will just be cosmetic and not affect the structural stability since there seems to be sufficient strength/stiffness in other bays to resist the seismic forces. But, some thought should be given to whether the column damage would be acceptable to building occupants. If it would not be acceptable, then some retrofit measures could be considered.

Appendix A: Tier 1 Checklists

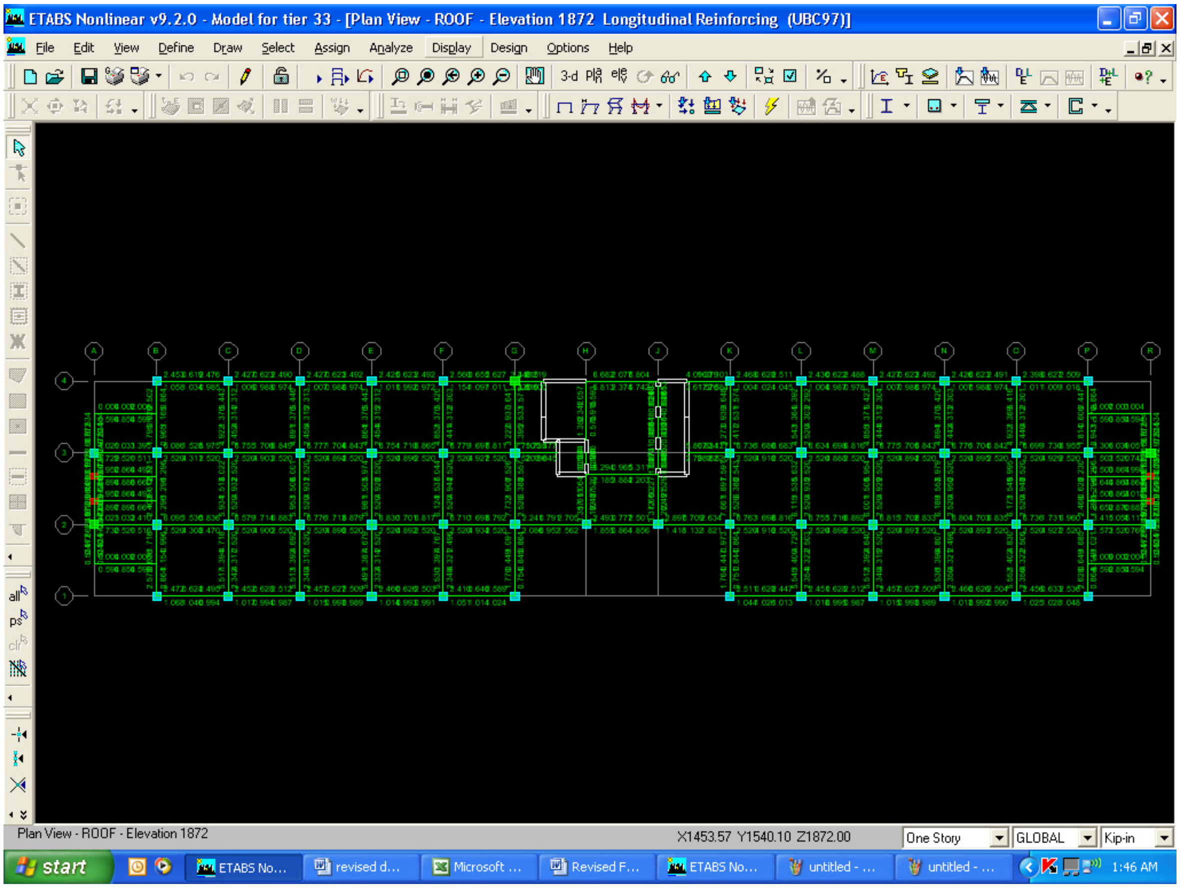
BUILDING SYSTEM	
Load Path	C
Adjacent Building	C
Mezzanine	NA
Weak Story	C
Soft Story	C
Geometry	C
Vertical Discontinuities	C
Mass Irregular	C
Torsion	NC
Deterioration	C
Post Tensioning Anchors	NA

LATERAL-FORCE RESISTING SYSTEM	
Redundancy	C
Interfering Wall	NC
Shear Stress Check	C
Axial Stress Check	C
Proportion of Infill Walls	NC
Concrete Columns	C
Solid Wall	C
Over All Construction Quality	C
Flat Slab Frames	NC
Pre-stressed Frames	N/A
Captive Column	C
Column Aspect Ratio	C
No Shear Failure	C

LATERAL-FORCE RESISTING SYSTEM (cont'd)	
Stirrup and Tie Hooks	C
Deflection Compatibility	N/A
Diaphragm Continuity	C
Plan Irregularity	N/A
Diaphragm Reinforcement at openings	N/A
Transfer to Shear Walls	C
Uplift at Pile Caps	N/A
Strong Column / Weak Beam	C
Stirrup Spacing	C
Beam Bars	C
Column Bar Splices	C
Beam bar Splices	NC
Column Tie Spacing	NC
Joint Reinforcement	NC
Joint Eccentricity	NC

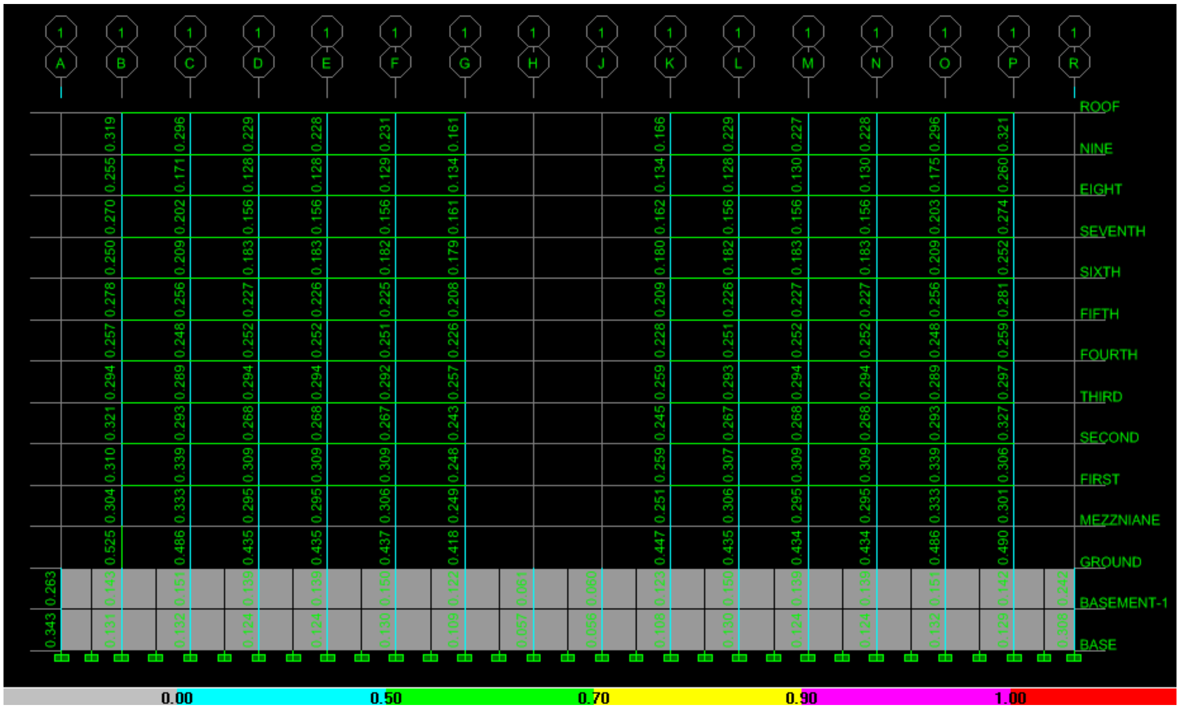
GEOLOGIC SITE HAZARDS AND FOUNDATION CHECKLIST	
Liquefaction	C
Slope Failure	C
Surface Fault rupture	C
Foundation Performance	C
Deterioration	C
Pole Foundation	N/A
Over turning	C
Ties between Foundation element	NC
Deep foundation	N/A
Sloping Sites	C

Appendix B: Linear Analysis (Tier 2) Results

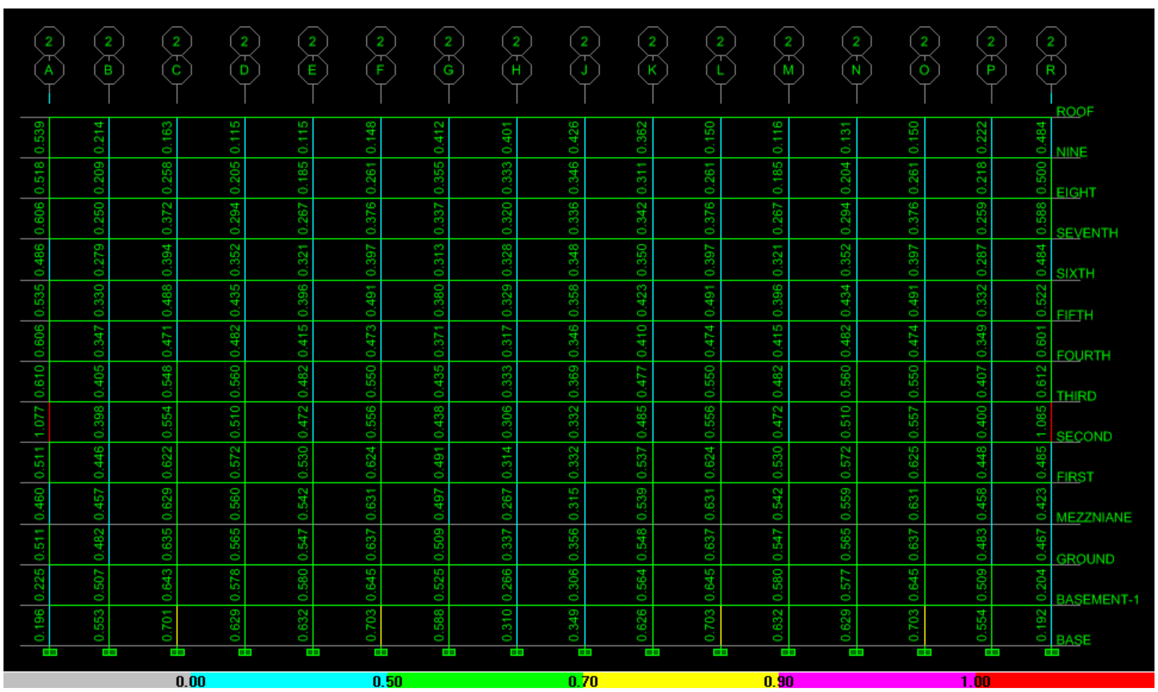


Demand/Capacity Ratios for Beams

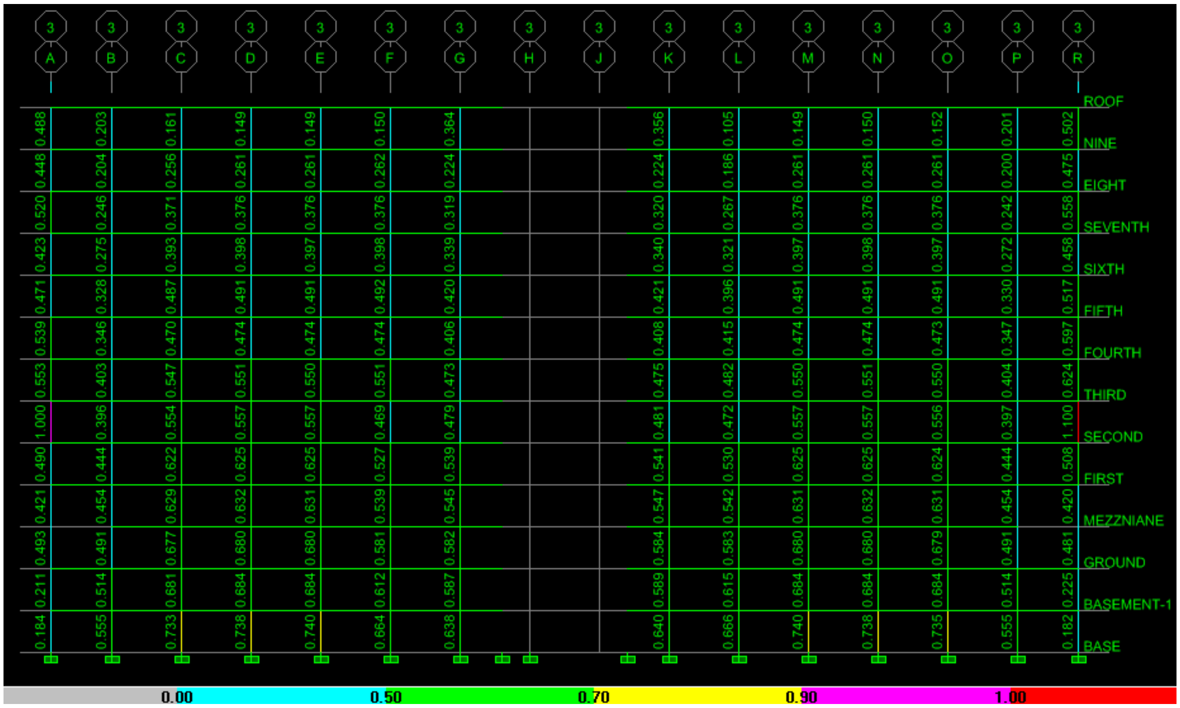
Maximum beam reinforcement is required at roof level as shown in the Plan View above.



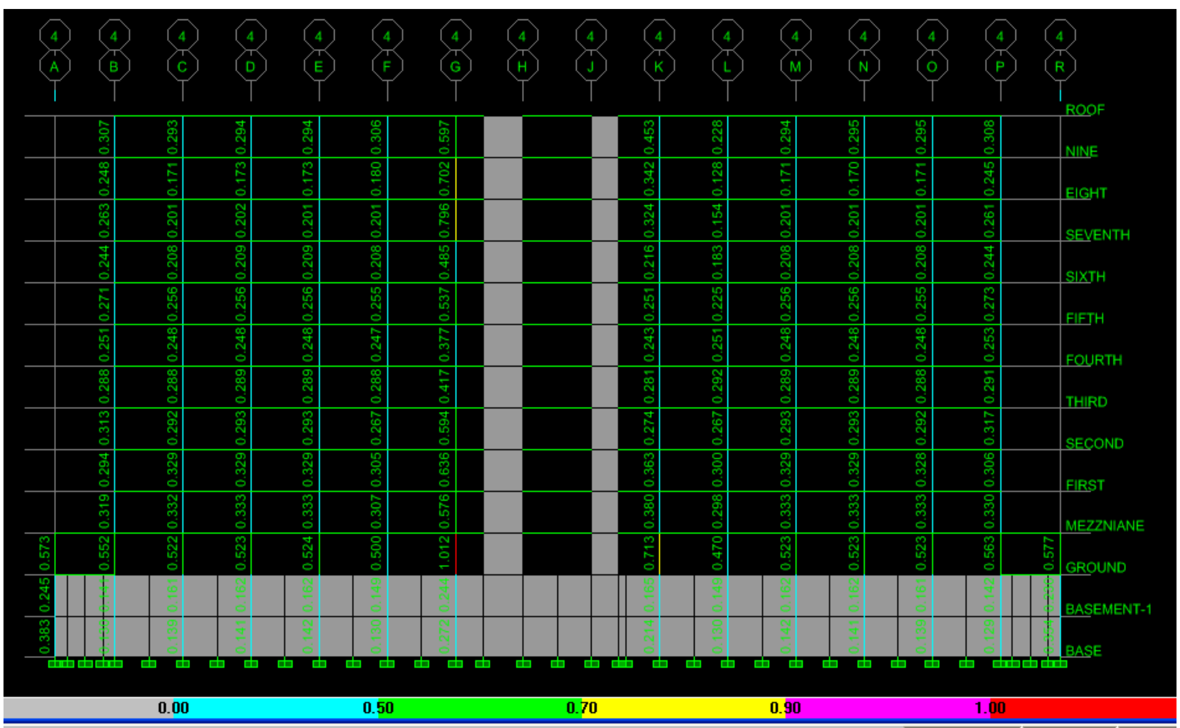
Demand/Capacity Ratios for columns at Grid-1



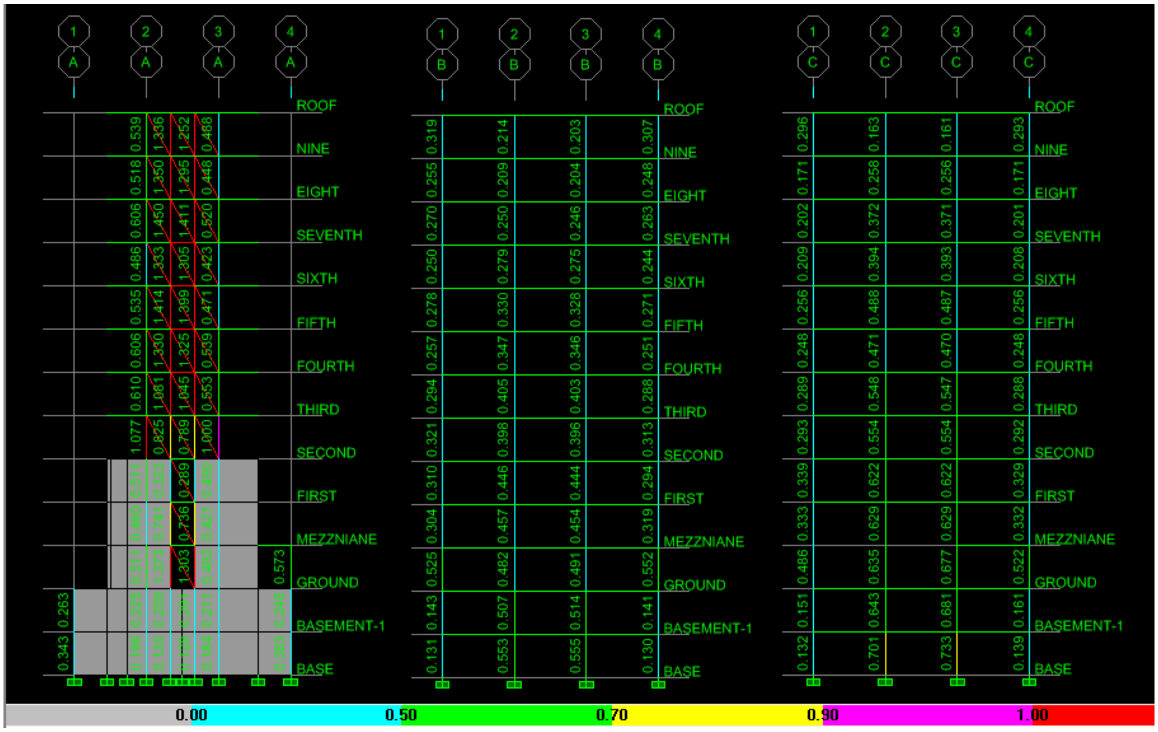
Demand/Capacity Ratios for columns at Grid-2



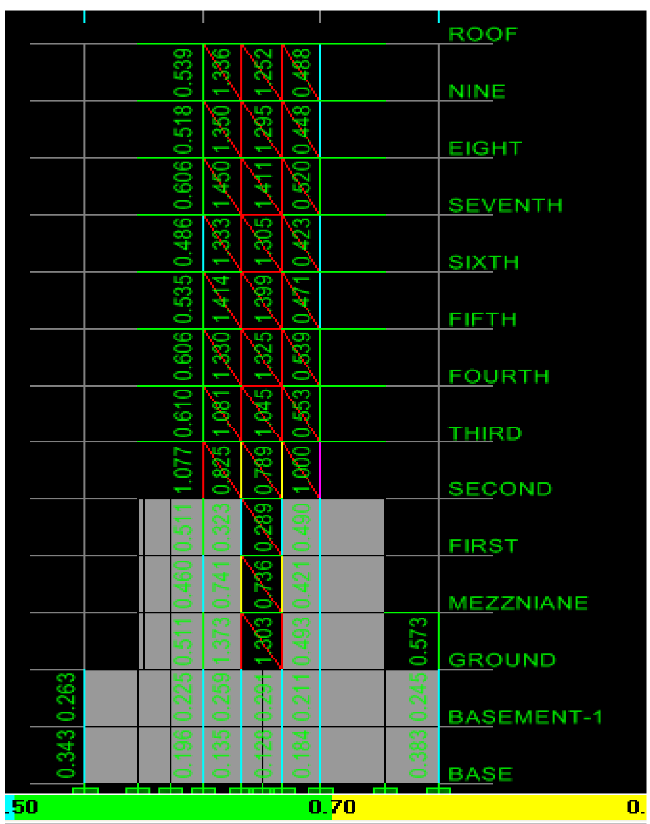
Demand/Capacity Ratios for columns at Grid-3



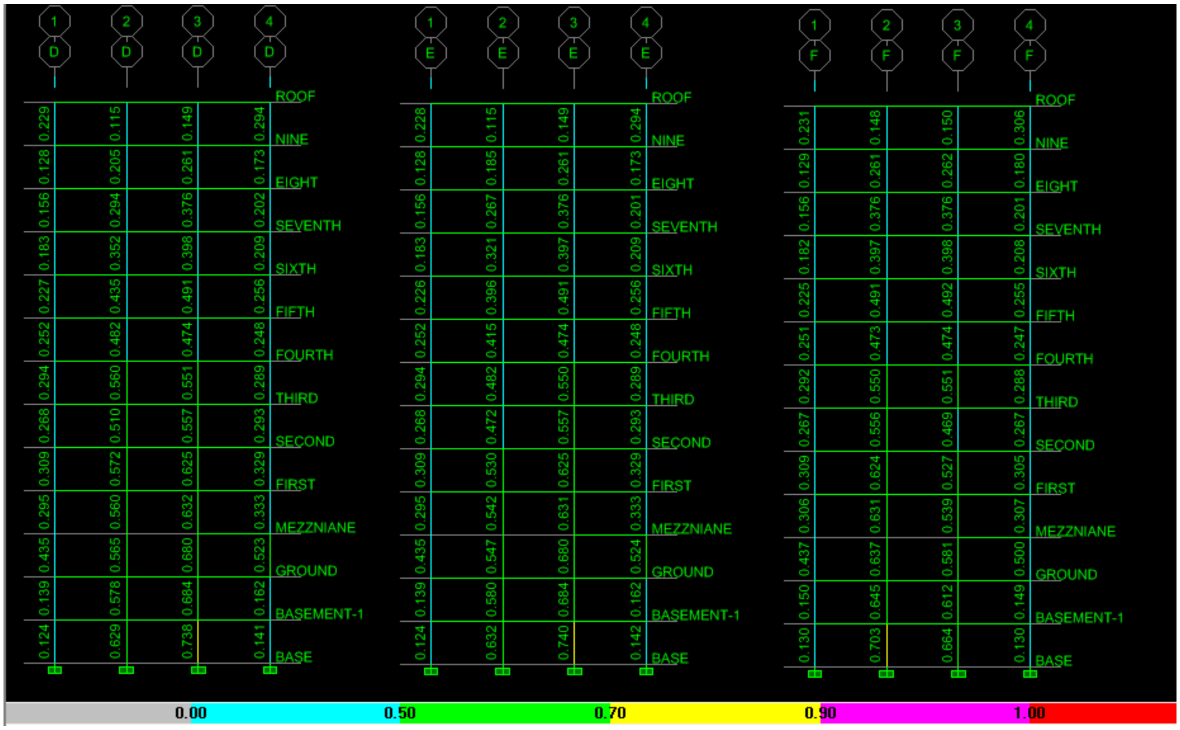
Demand/Capacity Ratios for columns at Grid-4



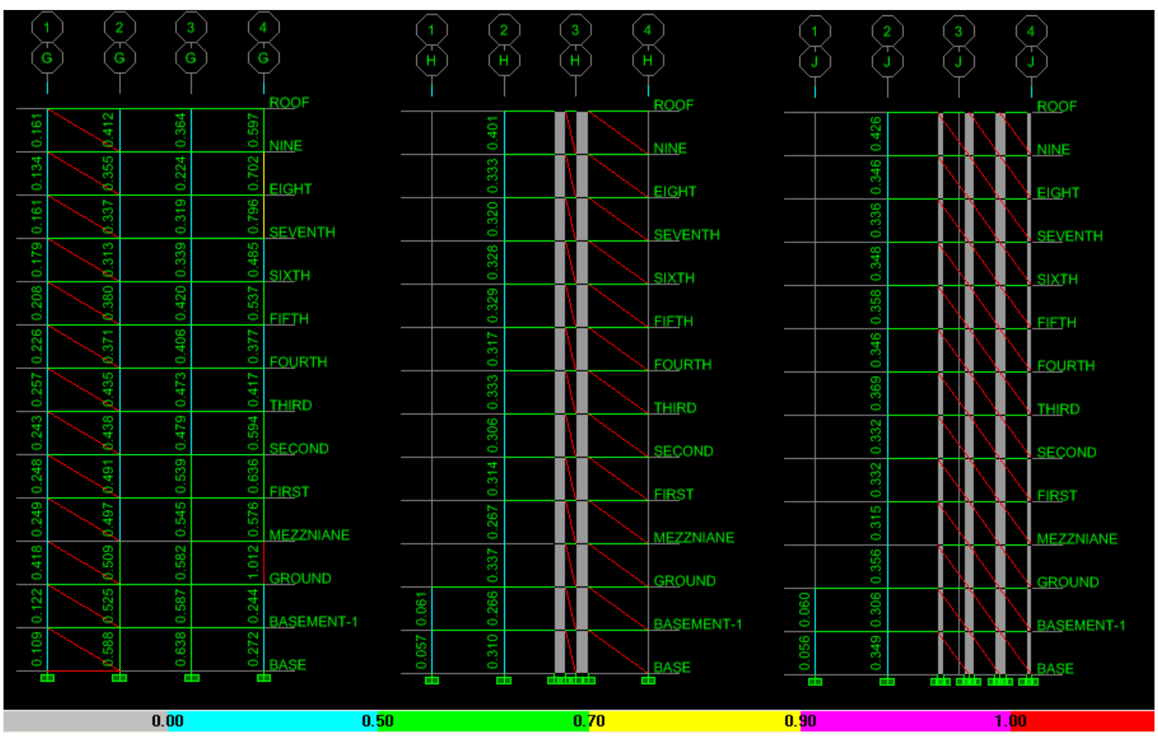
Demand/Capacity Ratios for columns at Grid-A,B and C



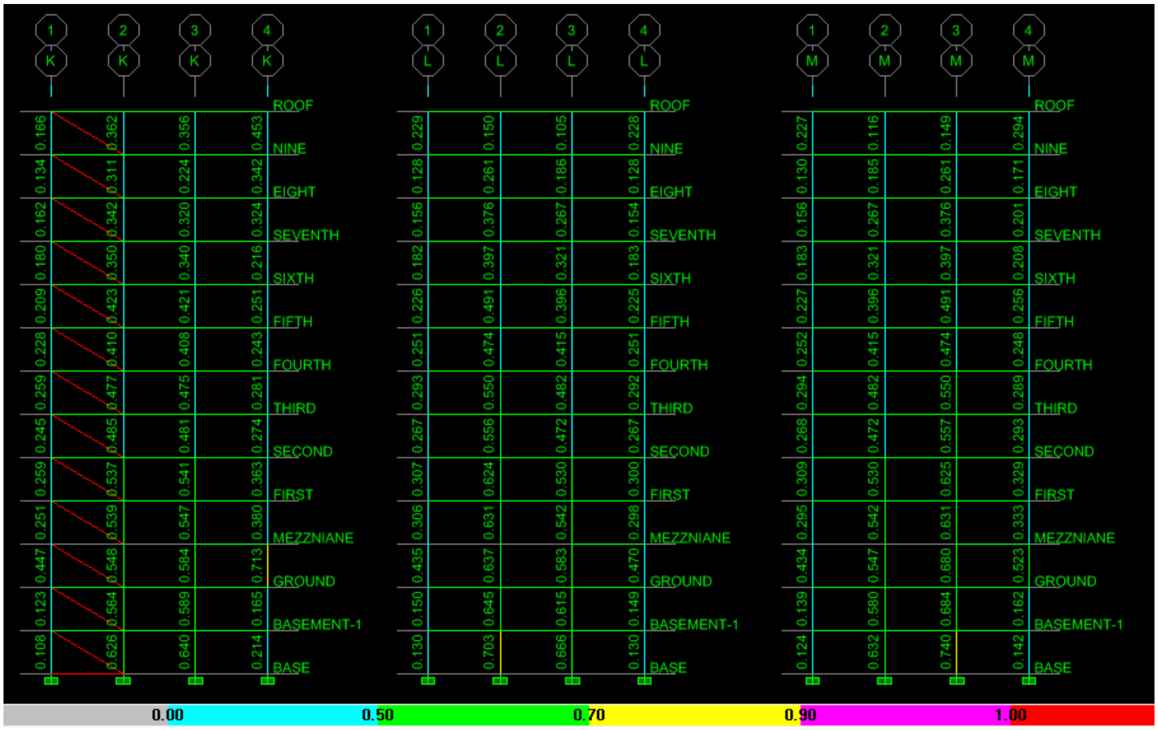
ZOOM VIEW OF Grid A



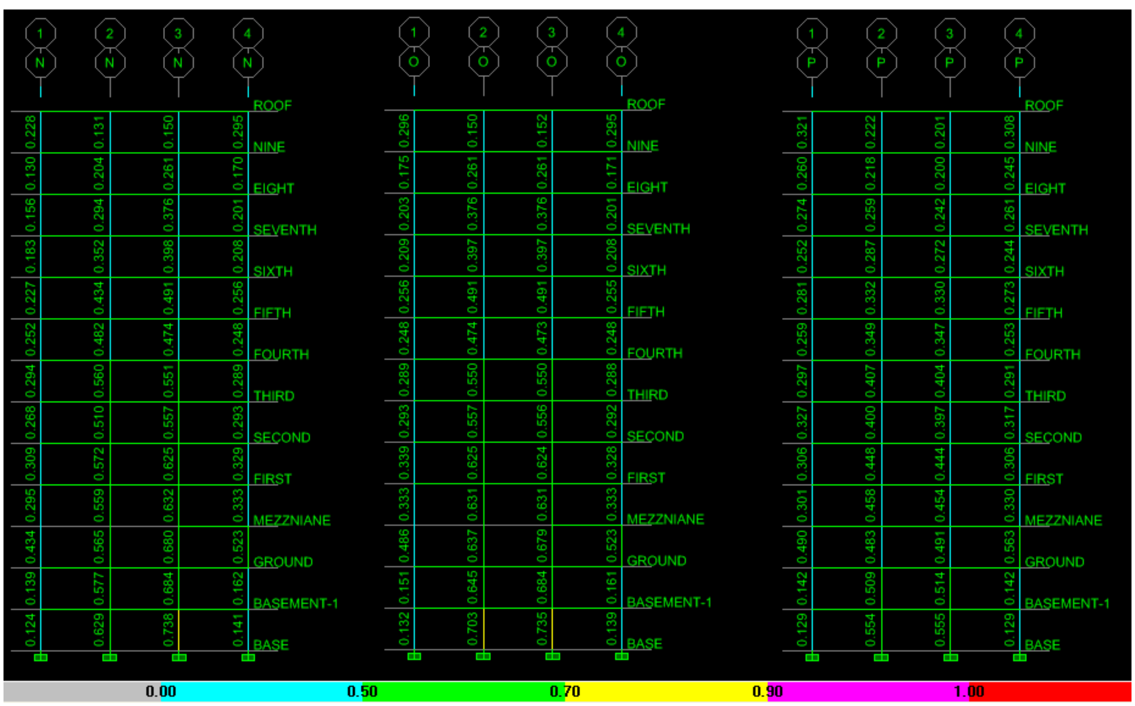
Demand/Capacity Ratios for columns at Grid-D,E and F



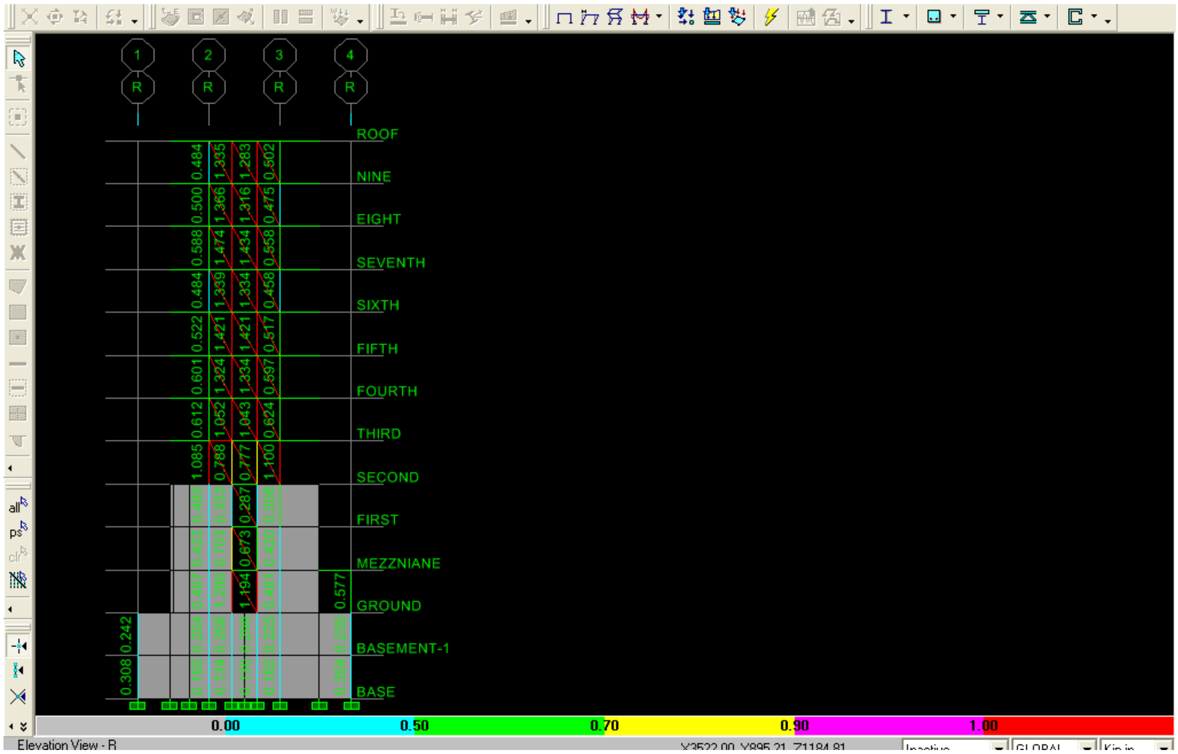
Demand/Capacity Ratios for columns at Grid-G,H and J



Demand/Capacity Ratios for Frame at Grid-K,L and M



Demand/Capacity Ratios for Frame at Grid-N,O and P



Demand/Capacity Ratios for Frame at Grid-R

ZOOM VIEW OF Grid R