G.Muttrah Commercial & Residential Complex

Muscat, Sultanate of Oman

Technical Report I

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Structural Option

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Executive Summary

The following report will analyze the existing structural condition of the G.Muttrah Commercial and Residential Complex in Muscat, The sultanate of Oman. Buildings in the sultanate of Oman are structurally designed according to the British Standards; however the analysis in this report will be performed using U.S standards and codes. Concrete is the leading material used for construction in Oman which makes the G.Muttrah Complex a concrete building as well. The structure is a reinforced concrete moment frame with 8 stories excluding the parking in the basement level. The building will incorporate retail spaces, offices and residential apartments.

Since the British Standards direct the design, the metric unit was used in the original design of the G.Muttrah building. This report will however analyze the building using United States Customary System (English units). The conversions will be accurately approximated and also increased or decreased depending on the calculation in order to obtain a conservative result. Values will hence be reported in English units.

The codes used for the analysis are the ASCE 7-05 and ACI 318-08. All the relative loads in the building will be analyzed and compared to the existing design. The wind loads are calculated using the Main wind Force Resisting System, but the loads used in the original design are not available for comparison. Additional lateral system analysis will be conducted in future reports. The seismic load was calculated using the minimum design allowed in the United States, category A in the Seismic Design Criteria, due to the fact that the Sultanate of Oman is considered in a seismic safe zone and there are no local seismic design requirements.

Gravity systems were also examined in order to compare the assumed loads to the original design. Two types of columns were analyzed where the majority (square columns), were adequate and did not differ substantially from the assumed loads. The other long narrow columns seemed to have a greater strength than required. These columns might be intended to help the lateral force resisting system. Further analysis will be required to confirm this hypothesis. Flat plate slabs were also checked and strength was sufficient to carry the assumed loads while the increase in amount of steel in some parts of the slab might have been used to have a uniform distribution for ease of construction.

Further details and analysis in the report will help gain a better understanding of the G.Muttrah Complex's structural system.

Introduction

The G.Muttrah Commercial & Residential Complex is a mixed use building in a commercially developing region in the city of Muscat, Sultanate of Oman. Covering an area of approximately 280,000 square feet, the reinforced concrete building will consist of eight floors excluding the parking at the basement level. Retail space will occupy the ground floor, offices in the second floor and 96 apartments in the rest of the 6 floors. The parking garage in the basement will serve 115 slots for the tenants due to the limited parking spaces in the area. More parking spaces will be available around the perimeter of the building which will only provide space for 63 cars.

The typical floor height is 10 ft for the basement level, 14 ft for the retail, 12 ft for the offices and 10 ft on the rest of the residential floors. A flat roof is used to place all the HVAC equipment. The plot has a slope of about 10 ft from the northwest corner to the southeast corner. This slope is used to incorporate the basement level as a parking garage. The ground level is set at 2.6 ft cm below grade while the basement level floor is constructed at 12 ft below grade (Figure 1). Like a typical parking garage, the concrete reinforced columns are placed in a rectangular grid in order to accommodate all the spaces and for ease of transportation.

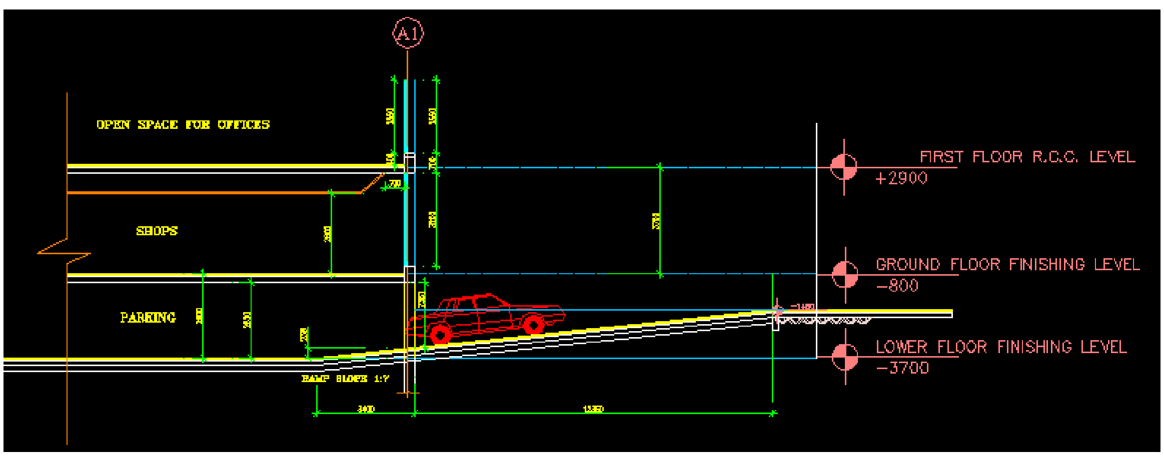


Figure 1: A section showing the entrance of the garage level

Structural System Overview

Summary

The G.Muttrah Commercial & Residential Complex consists of a reinforced concrete frame, shear walls and a combination of reinforced concrete flat plate slabs on some floors and typical two way slabs on beam frame system on the others. The dimensions of the building plan are about 300ft by 132ft. The typical roofing/floor system span is between 10ft and 30 ft. The material strength used is approximately 5,700 psi strength concrete and 65,000 psi steel strength. Finally, the roof of the building is a 6 in thick slab that only has to carry the loads from the mechanical equipment on the rooftop. There are no snow loads for this building since the weather statistics show that the chances of snow in Oman are slim to none.

Floor Slabs & Beams

The second and third floor of the G.Muttrah complex consists of a flat plate slab system with drop panels. The floors have 2 varying slab thickness; One at 10in slab thickness with a drop panel of 14in and reinforcement of # 3's and #4's in U.S standard. The second is at 14in slab thickness with a drop panel of 22in and reinforcement of #5's (see figure 2). The rest of the floors have a typical two-way slab system with slabs thickness varying from 6in to 8in. The slabs are supported by the usual rectangular beams that range from 6in x 20in to 32in x 20in.

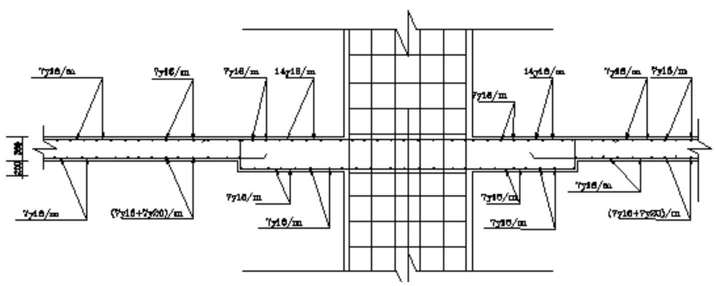


Figure 2: Flat plate slab and column on the second floor

Foundation & Columns

As for the foundation, a 4 ft thick mat slab is used to carry the loads from the different columns. The mat slab is reinforced with 2 layers of #20's and 2 layers of # 10's mesh running both ways. Gravity loads from the building are carried down through reinforced concrete columns that are aligned together in a simple grid, with the majority running throughout the entire building. The columns have a base at the foundation slab level (see figure 3) and range between 14in x 21in to 28in x 47in.

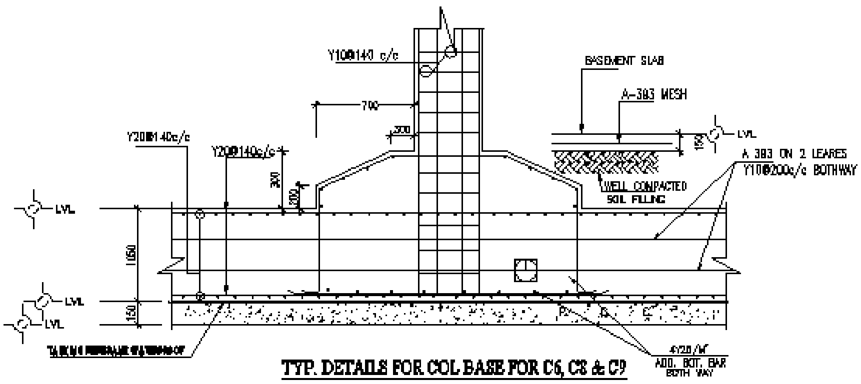


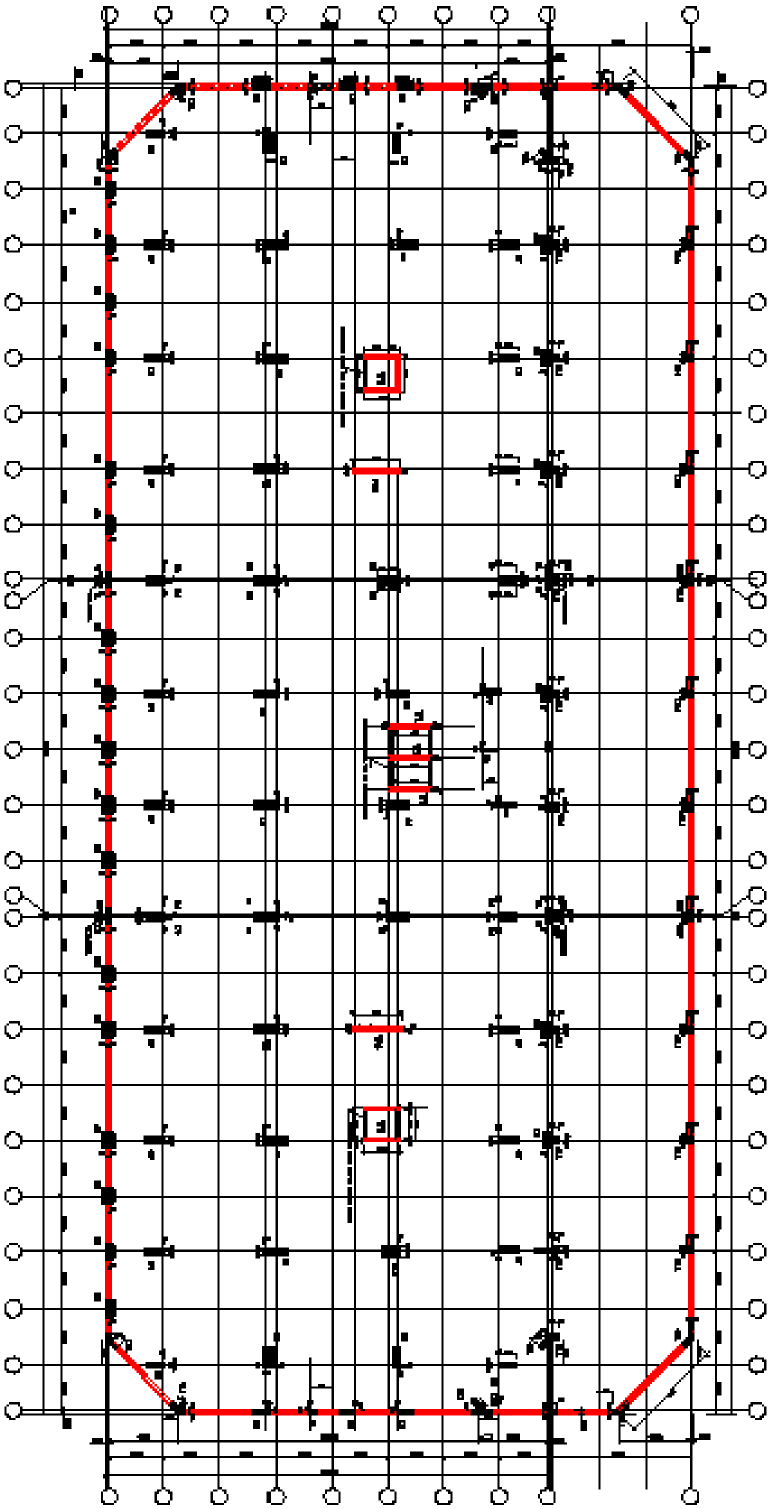
Figure 3: Typical column base at foundation level

Lateral System

Shear walls are used to resist the lateral force in the G.Muttrah complex. The major shear walls are located around the perimeter of the building and start at a thickness of 14in at the basement and decrease to 8in as they reach the roof. The rest of the shear walls, total of 9, are interior walls that run in the north-south direction. This is expected since the north-east axis is the weaker axis due to the wind direction and exposure to a larger surface area. The interior shear walls also run to the eighth floor and only cover a span of 12ft.

The lateral load is transformed through the diaphragm and beams to the shear walls where the load is carried down to the foundation. The following plans highlight the shear walls within the building:

Figure 4: Frame plan showing shear walls



Code & Design Standards

Applied to original design:

BS8110-British Standard for the design and construction of reinforced and prestressed concrete structures, structural design.

Substituted for analysis:

American Society of Civil Engineers (ASCE 7-05), Minimum Design Loads for Buildings and other Structures, 2005

American Concrete Institute (ACI 318-08), Building code Requirements for Structural Concrete

Material Strength Requirement Summary:

Cast-in-place Concrete

- Foundations: 5700 psi
- Formed Slabs: 5700 psi
- Columns & Walls: 5700 psi
- Reinforcement: 65000psi

Required Loads

The codes for the original design of the building are from The British Standards (BS8110). The codes used by the engineer are currently unavailable for comparison; however, below is a list of the loads from ASCE 7-05 which were used in this analysis of this report.

Live Loads:

Occupancy	Load (psf)
Parking	40
Entry	100
Office	50
Retail	100
Residential	40
Corridor	100
Restrooms	100
Roof	20
Stairs	100
Ramps (vehicle)	250
Sidewalk	250
Exterior	100

Dead Loads

Material/Occupancy	Load (psf)
Normal Weight Concrete	150 pcf
Floor Superimposed	15 psf
Roof Superimposed	30 psf
Facade	30 psf

Lateral Loads

Wind Loads

The wind loads used for analysis in this report are according to ASCE7-05. However, the loads used for the original design are not available for comparison. The method used to determine the wind load is the Main Wind Force Resisting System. For simplification, the curves around the edge of the building are ignored and the building is assumed to have a rectangular shape. The height of the building is 96ft to the roof and 102 ft to the parapet. The length and breadth of the building are 300ft and 132ft respectively.

North-South:

The wind loads in the north-south govern the design which was expected from the additional shear walls running in the given direction. The pressure at the bottom of the building in the windward starts at 7.8 PSF and gradually increases to 10.1 PSF as you move up the building. Although the difference in pressure is not as large it cannot be ignored and should be designed accordingly. The pressure in the leeward side is constant at 7.1 PSF for all heights. (See figure 6). Refer to Appendix-A for detailed calculations.

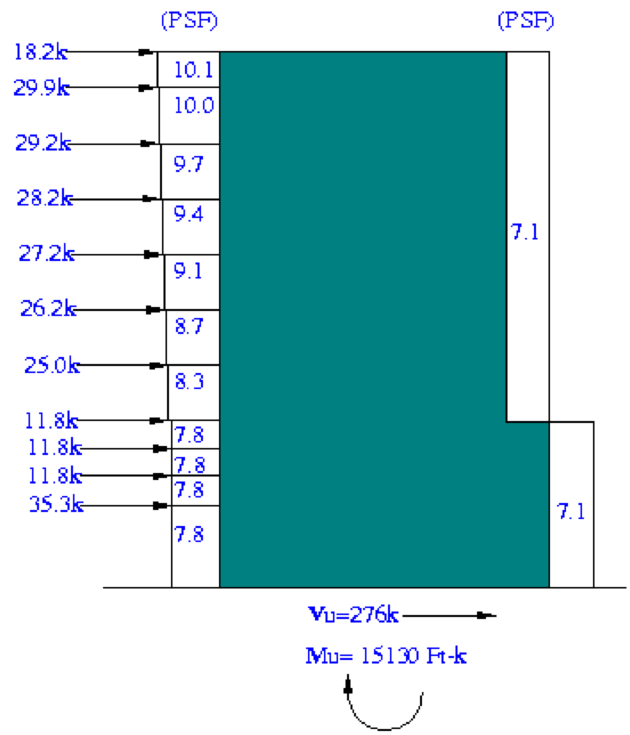


Figure 6: Wind load on north-south face

East-West:

The wind loads in the east-west direction are very close to the pressures in the north-south direction. Nevertheless, the small surface area which it acts upon does not create an impact as great as the north-south direction. The leeward pressure, on the other hand, is significantly larger for the east-west direction. The overall shear at the base and moment are still larger for the north-south direction. (See figure 7).

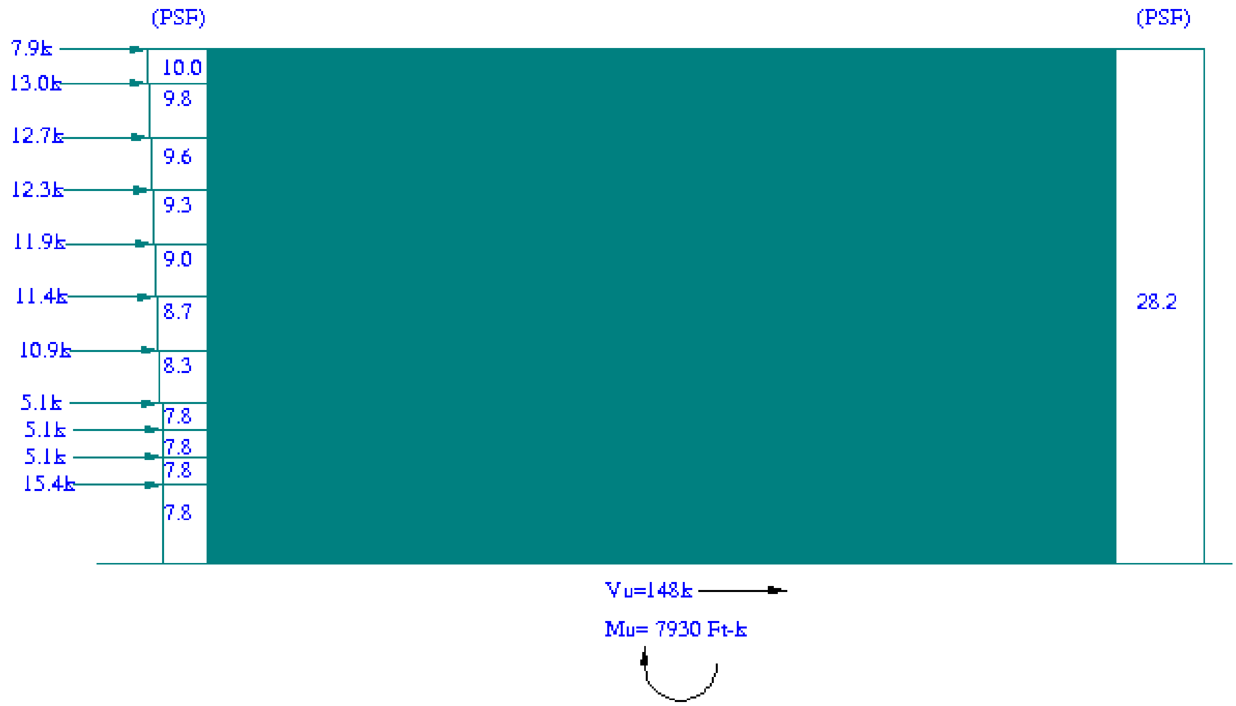


Figure 7: Wind load on east-west face

Additional analysis and design of the lateral system in future technical reports will provide an estimate of the loads used in the original design and will help compare the different methods in design. For specific variables and calculations, refer to Appendix A.

Seismic Loads

Data provided by the engineers working on the project confirms that there are no requirements for seismic design in the local area. The Sultanate of Oman is considered a seismic safe zone, where the wind loads are the loads resisted by the lateral system. For the purpose of this analysis, minimum seismic loads are going to be applied to the building according to the United States standards (ASCI 7-05). The minimum design falls under category A in the Seismic Design Criteria. The following figure 8 shows the shear force at the base and different levels of the building.

Design Category	A
Cs	0.01

Table 1: Design criteria

		Weight					
		Column	**Slab(w/ superimposed)**	**Beams**	**Walls & Façade**	**Total**	**Shear (K)**
Floor	**B**	949440	0	0	1516320	2465760	24.6576
	1	1329216	4143750	2359119	2304288	10136373	101.36373
	2	1075230	7706250	0	1710720	10492200	104.922
	3	1971255	6956250	0	1425600	10353105	103.53105
	4	597909	3187500	2359119	1205280	7349808	73.49808
	5	530934	3187500	2359119	1205280	7282833	72.82833
	6	433899	3187500	2359119	997920	6978438	69.78438
	7	354654	3187500	2359119	997920	6899193	68.99193
	8	304959	3187500	2359119	997920	6849498	68.49498
	R	0	3937500	2359119	0	6296619	62.96619
	Total	7547496	38681250	16513833	12361248	75103827	751.03827

Table 2: Building Weight Summary

Base V_u= 751 Kips

M_u= 33,050 Ft-Kips

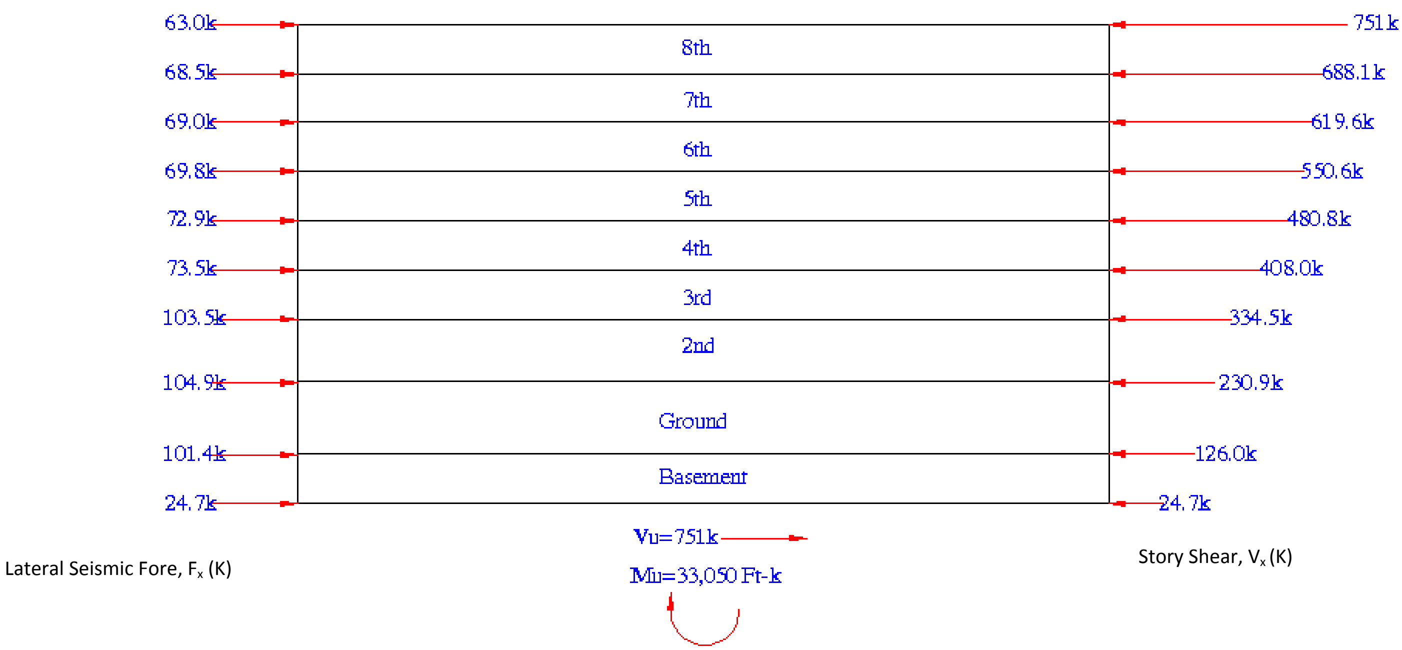


Figure 8: Seismic load on building

The resultant seismic shear load is 751Kips which is relatively high compared to the wind load calculated earlier. This is due to the following reasons:

- The method used for calculating the weight of the building gives a general approximation. The windows in the façade were not taken into consideration while an assumption of same spans for all beams was used to simplify the calculation. It was also assumed that the weights of the slabs are the same in all the floors. These along with other assumptions would give a greater value for the total weight of the building. Such assumptions are valid since they would result in a conservative seismic load.
- The wind load in the sultanate of Oman is generally low, given at 75mph. This speed is much lower than the values expected in the United States of America. Hence the minimum seismic load from the U.S standards could possibly be greater than wind loads in such an area as Oman.

Analysis & Conclusions

Spot Check of Typical Gravity Load Areas

Two spot checks were performed to typical gravity members in order to compare the strengths of the members designed to the loads that were assumed to apply on the building in this report. The two checks were a column check and a slab check.

Column Check

Two columns were examined since the columns in the building were divided into two categories; long narrow columns and square columns. The long narrow column examined was a 20in x 55in reinforced concrete column with (38) #8's rebar's. The strength of the column was calculated at 3641 K while the required strength was a low 570 K. A possible reason for the substantial difference in loads could be that the columns are also used, along with shear walls and moment frame, to resist the lateral loads. The long narrow columns are aligned with the shear walls in the north-south direction which happens to be the weaker axis of the building. Such columns could add little strength to the lateral system. Further analysis would be required in order to confirm such an assumption.

The square column, on the other hand, was a 20in x 20in column with (16) #6's rebar's. The calculated strength of the column is 1196 K compared to the required load of 973 K. Hence the square columns are at a reasonable size and the assumed loads on the building are acceptable. Further details in calculation are found in Appendix-A

Slab Check

The slab chosen for analysis was the flat plate with drop panels slab in the second floor. Reinforced with (6) # 4's and (18) # 5's, the 10 in slab was adequate to carry the load. The amount of steel used was greater in some parts of the slab while sufficient in others. However, the minimum amount of steel was provided and the uniform use of steel is probably used to ease the construction process. Punching shear was also checked and the slab thickness exceeded the 8.5in minimum required for deflection calculations. Further details in calculation are found in Appendix- A.

Appendix A: Calculations

Wind

Table A-1:

Mean Velocity(mph)	75	Provided by engineer
Occupancy Category IBC	II	IBC
Exposure Category	B	
Directionality Factor K_d*	0.85	ASCE 7-05
Importance Factor. I	1	ASCE 7-05
Topographic Factor K_{zt}	1	ASCE 7-05
Velocity Factor $q_z=0.00256K_zk_{zt}k_dv^2I$	Table	
Velocity Coefficient K_z	Table	
α	7	
Z_g	1200	
ε	1/3.0	
ℓ	320	
c	0.3	
β	1 (Assumed)	
b	0.45	
Building Frequency η_1	0.980	Structure is flexible
Peak Factors g_q	3.4	
Peak Factors g_v	3.4	
Peak Factors g_R	4.18	
Turbulence Factor Z	57.6	$>z_{min}= 30'$
Intensity of Turbulence I_z	0.273	
Integral Length L_z	385	
Background Response Q	0.83	
Mean Wind Speed V	56.8	
Reduced Frequency N1	6.64	
Rn	0.042	
Rh	0.123	for η=7.62
Rb	0.091	for η=10.5
RL	0.12	for η=79.7
Resonant Response	0.0166	(N-S)
Resonant Response	0.0188	

Gust Effect Factor	0.83	(N-S)
Gust Effect Factor	0.82	
Building is Enclosed		
kp	0.99	
qp	12.12	
GCpn	1.5 Windward	
GCpn	(-1.0) Leeward	
Pp	18.18	windward
Pp	-12.12	Leeward
High-Rise building		
GCpi	0.18 or -0.18	
External Pressure Coefficient		
Windward Cp	0.8	
Leeward (N-S) Cp	-0.5	L/B=.44
Leeward (E-W) Cp	-2.65	L/B=2.27
Sidewall Cp	-0.7	

North-South

Height= 96 ft

B= 300

L= 132

Table A-2:

Location	Height (Ft)	K_z	q_z	P_z (psf)	P_z (Kips)	Overturning Moment, M_o (ft-kips)
Windward	0-15	0.7	8.568	7.848	35.317	529.757
	20	0.7	8.568	7.848	11.772	235.448
	25	0.7	8.568	7.848	11.772	294.309
	30	0.7	8.568	7.848	11.772	353.171
	40	0.76	9.302	8.336	25.008	1000.307
	50	0.81	9.914	8.742	26.227	1311.339
	60	0.85	10.404	9.067	27.202	1632.124
	70	0.89	10.894	9.392	28.177	1972.415
	80	0.93	11.383	9.718	29.153	2332.211
	90	0.96	11.750	9.961	29.884	2689.569
	96	0.98	11.995	10.124	18.223	1749.412
Leeward	ALL	0.98	11.995	-7.137	-21.400	-1027.0

V_u= 276K, M_u= 15130 Ft-K

East-West

Height= 96 ft
B= 132
L= 300

Table A-3:

Location	Height (Ft)	K_z	q_z	P_z (psf)	P_z (Kips)	Overturning Moment, M_o (ft-kips)
Windward	0-15	0.7	8.568	7.780	15.404	231.0573
	20	0.7	8.568	7.780	5.135	102.6921
	25	0.7	8.568	7.780	5.135	128.3652
	30	0.7	8.568	7.780	5.135	154.0382
	40	0.76	9.302	8.261	10.905	436.2058
	50	0.81	9.914	8.663	11.435	571.7545
	60	0.85	10.404	8.984	11.859	711.5426
	70	0.89	10.894	9.305	12.283	859.8099
	80	0.93	11.383	9.626	12.707	1016.5562
	90	0.96	11.750	9.867	13.025	1172.2427
	96	0.98	11.995	10.028	7.942	762.4452
Leeward	ALL	0.98	11.995	-28.200	-37.200	-1785.6

V_u= 148K, M_u= 7930 Ft-K

Seismic

Table A-4:

Concrete	150pcf
Floor superimposed	10psf
Roof superimposed	30psf
Façade	30psf

Table A-5:

Slab	Area(sq-ft)	Slab thickness(ft)	Weight
Ground	37500	0.67	4143750
2nd	37500	1.17	7706250
3rd	37500	1.17	6956250
4th	37500	0.5	3187500
5th	37500	0.5	3187500
6th	37500	0.5	3187500
7th	37500	0.5	3187500
8th	37500	0.5	3187500
Roof	37500	0.5	3937500
		Total=	38681250

Table A-6

Beam	Quantity	Span(ft)	Area(sq-ft)	Weight
B110	23	24	4.31	356868
B107	11	24	4.31	170676
B104	8	24	2.22	63936
B106	2	24	4.31	31032
B109	16	24	4.31	248256
B111	14	5	4.31	45255
B114	2	4	2.67	3204
B203	13	24	2.72	127296
B113	12	24	4.31	186192
B112	8	24	4.31	124128
B30	2	24	2.72	19584
B29	11	24	2.72	107712
B201	24	24	2.72	235008
B202	11	30	2.72	134640
B205	12	30	6.03	325620
B101	44	12	1.56	123552
B102	16	12	1.95	56160
			Total	2359119

Table A-7:

Column	Weight
C1	10725
C2	107712
C3	94248
C4	120912
C5	408672
C6	176484
C7	100848
C8	30162
C9	154044
C10	20196
C11	80784
C12	165132
C13	252996
C14	525393
Total	2248308

Table A-8:

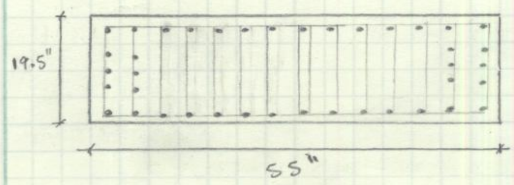
Wall	Thickness	Area	Weight
B	1.17	8640	1516320
1	1.17	12096	2304288
2	1	10368	1710720
3	1	8640	1425600
4	0.83	8640	1205280
5	0.83	8640	1205280
6	0.67	8640	997920
7	0.67	8640	997920
8	0.67	8640	997920
		Total =	12361248

Column Spot Check

1

SPOT CHECK: COMPRESSION MEMBER CHECK PER ACI 318-05

COLUMN C7 - GROUND LEVEL (INTERIOR COLUMN)



SIZE = 500 mm (19.5") x 1400 mm (55")

REINF = (38) #8's, #3 TIES

$\phi P_n = (0.80)\phi[0.85 f'_c (A_g - A_{st}) + F_y A_{st}]$ 10.3.6.2

$= (0.80)(0.65)[0.85(5.7)((19.5 \times 55)-(38 \times 0.79)) + 65(38 \times 0.79)]$

$= 3641$ KIPS

GROUND LEVEL:

TRIB AREA: $A_{TRIB} = 300" \times 150" = 45000\ in^2 = 312.5\ FT^2$

DEAD LOAD: CONC = $(\frac{8}{12}\ FT)(150\ PCF) = 100\ PSF$

MEP/PARTITIONS = 15 PSF

LIVE LOAD: RETAIL = 100 PSF

$P_D = (115\ PSF)(312.5\ FT^2) = 35.9k$

$P_L = (100\ PSF)(312.5\ FT^2) = 31.2k$

SECOND FLOOR:

TRIB AREA: $A_{TRIB} = 312.5\ FT^2$

DEAD LOAD: CONC = $(\frac{10}{12}\ FT)(150\ PCF) = 125\ PSF$

MEP/PARTITIONS = 15 PSF

LIVE LOAD: OFFICE = 50 PSF

$P_D = (140\ PSF)(312.5\ FT^2) = 43.8k$

$P_L = (50\ PSF)(312.5\ FT^2) = 15.6k$

2

THIRD FLOOR:

TRIB AREA: $A_{TRIB} = 312.5$ k

DEAD LOAD: CONC $= \left(\frac{11}{12} FT\right)(150\ PCF) = 137.5$ PSF

MEP/PARTITION: 15 PSF

LIVE LOAD: RESIDENTIAL $= 40$ PSF

$P_D = (152.5\ PSF)(312.5) = 47.7$ k

$P_L = (40\ PSF)(312.5) = 12.5$ k

FOURTH TO EIGHTH FLOOR:

TRIB AREA: $A_{TRIB} = 312.5$ k

DEAD LOAD: CONC $\left(\frac{6}{12} FT\right)(150\ PCF) = 75$ PSF

MEP/PARTITIONS $= 15$ PSF

LIVE LOAD: RESIDENTIAL $= 40$ PSF

$P_D = (90\ PSF)(312.5) = 28.1$ k

$P_L = (40\ PSF)(312.5) = 12.5$ k

ROOF LEVEL:

TRIB AREA: $A_{TRIB} = 312.5\ FT^2$

DEAD LOAD: CONC $= \left(\frac{6}{12} FT\right)(150\ PCF) = 75$ PSF

MEP/EQUIPMENT $= 30$ PSF

LIVE LOAD: ROOF $= 20$ PSF

$P_D = (105\ PSF)(312.5\ FT^2) = 32.8$ k

$P_L = (30\ PSF)(312.5\ FT^2) = 9.4$ k

TOTAL LOADS:

$P_D = 35.9 + 43.8 + 47.7 + (28.1 \times 5) + 32.8 = 300.7$

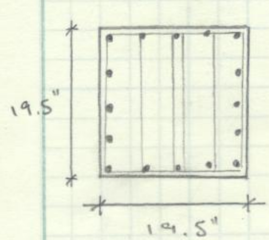
$P_L = 31.2 + 15.6 + 12.5 + (12.5 \times 5) + 9.4 = 131.2$

$P_u = 1.2D + 1.6L = 1.2(300.7) + 1.6(131.2) = 570.8$ KIPS

$\Rightarrow \boxed{\phi P_n \geq P_u \quad \therefore \underline{ok}}$

3

COLUMN C14 - GROUND FLOOR (EXTERIOR COLUMN)



SIZE = 500mm (19.5") x 500mm (19.5")

REINF. = (16) #6's , #3 TIES

A_g =

$$\phi P_n = (0.80)\,\phi\left[0.85 f'_c (A_g - A_{st}) + F_y A_{st}\right]$$

$$= (0.80)(0.65)\left[0.85(5.7)(19.5^2 - 16 \times .44) + 65(16 \times .44)\right]$$

$$= 1196 \text{ kips.}$$

GROUND LEVEL:

TRIB AREA: $A_{SLAB} = 186" \times 300" = 55\,800 \text{ in}^2$

$= 387.5 \text{ FT}^2$

$A_{WALL} = 7" \times 300" = 2100 \text{ in}^2$

$= 14.6 \text{ FT}^2$

DEAD LOAD: CONC SLAB = $(\frac{6}{12}\text{FT})(150 \text{ PCF}) = 75$ PSF

CONC WALL = (14 FT)(150 PCF) = 2100 PSF

MEP/PARTITION = 15 PSF

LIVE LOAD: RETAIL = 100 PSF

$P_D = (90 \text{ PSF})(387.5 \text{ FT}^2) + (2100 \text{ PSF})(14.6 \text{ FT}^2) = 65.5 \text{ k}$

$P_L = (100 \text{ PSF})(387.5 \text{ FT}^2) = 38.8 \text{ k}$

4

SECOND FLOOR:

TRIB AREA: $A_{SLAB} = 387.5\ FT^2$

$A_{WALL} = 14.6\ FT^2$

DEAD LOAD: CONC SLAB $= (\frac{14}{12} FT)(150\ PCF) = 175\ PSF$

CONC WALL $= (12 FT)(150\ PCF) = 1800\ PSF$

MEP/PARTITION = 15 PSF

LIVE LOAD: OFFICE = 50 PSF

$P_D = (190\ PSF)(387.5\ FT^2) + (1800\ PSF)(14.6\ FT^2) = 99.9k$

$P_L = (50\ PSF)(387.5\ FT^2) = 19.4k$

THIRD FLOOR:

TRIB AREA: $A_{SLAB} = 387.5\ FT^2$

$A_{WALL} = 14.6\ FT^2$

DEAD LOAD: CONC SLAB $= (\frac{14}{12} FT)(150\ PCF) = 175\ PSF$

CONC WALL $= (10\ FT)(150\ PCF) = 1500\ PSF$

MEP/PARTITION = 15 PSF

LIVE LOAD: RESIDENTIAL = 40 PSF

$P_D = (190\ PSF)(387.5\ FT^2) + (1500\ PSF)(14.6\ FT^2) = 95.5k$

$P_L = (40\ PSF)(387.5) = 15.5k$

FOURTH TO EIGHTH FLOOR:

TRIB AREA: $A_{SLAB} = 62'' \times 300'' = 18600\ in^2 = 129\ FT^2$

$A_{WALL} = 8'' \times 300'' = 2400\ in^2 = 16.7\ FT^2$

DEAD LOAD: CONC SLAB $= (\frac{6}{12} FT)(150\ PCF) = 75\ PSF$

CONC WALL $= (10\ FT)(150\ PCF) = 1500\ PSF$

MEP/PARTITION = 15 PSF

5

LIVE LOAD: RESIDENTIAL = 40 PSF

$P_D = (90\text{ PSF})(387.5\text{ FT}^2) + (1500\text{ PSF})(16.7\text{ FT}^2) = 59.5\text{ k}$

$P_L = (40\text{ PSF})(387.5\text{ FT}^2) = 15.5\text{ k}$

ROOF LEVEL:

TRIB AREA: $A_{SLAB} = 387.5\text{ FT}^2$

DEAD LOAD: CONC SLAB = $(\frac{6}{12}\text{ FT})(150\text{ PCF}) = 75\text{ PSF}$

MEP = 30 PSF

LIVE LOAD: ROOF = 20 PSF

$P_D = (105\text{ PSF})(387.5\text{ FT}^2) = 40.7\text{ k}$

$P_L = (20\text{ PSF})(387.5\text{ FT}^2) = 7.8\text{ k}$

TOTAL LOADS:

$P_D = 65.5\text{k} + 99.9\text{k} + 95.5\text{k} + (59.5\times5)\text{k} + 40.7\text{k} = 599.1\text{k}$

$P_L = 38.8\text{k} + 19.4\text{k} + 15.5\text{k} + (15.5\times5)\text{k} + 7.8\text{k} = 159\text{k}$

$P_u = 1.2D + 1.6L$

$= 1.2(599.1) + 1.6(159)$

$= 973\text{ k}$

$\Rightarrow \boxed{\phi P_n \geq P_u \quad \therefore \underline{ok}}$

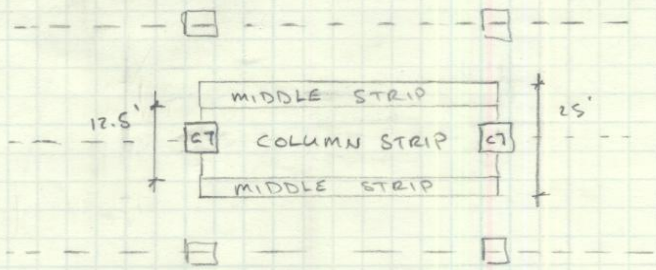
Slab Spot Check

1

SPOT CHECK: SLAB THICKNESS – DESIGN METHOD ACI 318-08 13.6

ASSUMPTIONS:

* SUCCESSIVE SPANS DO NOT DIFFER BY MORE THAN 1/3 THE LONGER SPAN.



2nd LEVEL FLAT PLATE

$L_1 = 25'$, $L_n = (25') - (20'') = 23'-4''$

$L_2 = 25'$

COLUMN STRIP $= \frac{25'}{2} = 12.5'$

MIDDLE STRIP $= 25'$

DEAD LOADS:

CONC. $= (150\ PCF)(\frac{10}{12}\ FT) = 125\ PSF$

MEP / PARTITIONS $= 15\ PSF$

LIVE LOADS:

OFFICE $= 50\ PSF$

$q_u = 1.2D + 1.6L = 1.2(140) + 1.6(50) = 248\ PSF$

$M_u = \frac{q_u L_2 L_n^2}{8} = \frac{.248(25)(23.33)}{8} = 422\ FT\text{-}K$

2

NEGATIVE FACTORED MOMENT
& POSITIVE FACTORED MOMENT

$M^- = (0.65)(422 \text{ FT-K}) = 274 \text{ FT.K}$

$M^+ = (0.35)(422 \text{ FT-K}) = 148 \text{ FT.K}$

COLUMN STRIP MOMENT

$\frac{L_1}{L_2} = \frac{26'}{25'} = 1 \quad , \quad \alpha f = 0$

$M^-_{COL\ STRIP} = (0.75)(274) = 206 \text{ FT.K}$

$M^+_{COL\ STRIP} = (0.6)(148) = 89 \text{ FT.K}$

MIDDLE STRIP MOMENTS

$M^-_{MID\ STRIP} = (0.25)(274) = 68 \text{ FT-K}$

$M^+_{MID\ STRIP} = (0.4)(148) = 59 \text{ FT.K}$

SLAB STRENGTH: ($f'_c = 5.7$ ksi, $f_y = 65$ ksi)

COLUMN STRIP:

used by engineer

$b = 150'' \quad , \quad h = 10'' \quad , \quad d = 10 - \underset{CVR}{1''} - 2(0.20) = 8.6''$

NEGATIVE MOMENT REINF

(6) #4's AND (18) #5's

$A_s = 6(0.20) + 18(0.31) = 6.78 \text{ in}^2$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{6.78(65)}{0.85(5.7)(150)} = 0.61 \text{ in} \Rightarrow c = \frac{0.61}{.765} = .8 \text{ in}$

$\Rightarrow \varepsilon_y = \frac{0.003}{.8}(8.6 - .8) = .029 > 0.005 \Rightarrow \phi = 0.9$

✓ o.k

$\phi M_n = \phi A_s F_y (d - a/2) = 0.9(6.78)(65)(8.6 - \frac{.61}{2})$

$= 274 \text{ FT-K}$

$\boxed{\phi M_n > M^-_{COL\ STRIP} \ \therefore \ \text{ok}}$

3

SAME REINF. USED FOR ALL MOMENTS.

$\Rightarrow$ $\boxed{\phi Mn > \text{ALL REQUIRED MOMENTS} \therefore \underline{\text{OK}}}$

SLAB IS OK IN FLEXURE.

SLAB DEFLECTION:

$Ln = 25'$

MIN SLAB THICKNESS PER ACI 318-08 13.1.4

$t > Ln/36$ FOR 60,000 PSI
$t > Ln/34$ FOR 75,000 psi
} TABLE 9.5 (c)

interpolate for 65,000 psi

min $t = 8.5'' < 10''$ ✓ o.k.

$\boxed{\text{NO NEED TO CALCULATE DEFLECTION}}$

SHEAR (PUNCHING SHEAR):

COLUMN C7:

drop panel

$20'' \times 55''$, $d = 14'' - 1'' - 2(0.20) = 12.8''$

$b_o = (20 + \frac{14}{2})2 + (55 + \frac{14}{2})2 = 178''$

$$V_c = \min \begin{cases} (2 + \frac{4}{\beta})\sqrt{f_c}\, b_o d \\ (\frac{\alpha_s d}{b_o} + 2)\sqrt{f_c'}\, b_o d \\ 4\sqrt{f_c}\, b_o d \end{cases}$$

4

$\beta = \frac{55}{20} = 2.75$

$\alpha_s = 40$ FOR INT. COLUMNS

$- \left(2 + \frac{4}{2.75}\right)\sqrt{5700}\,(178)(12.8) = 594$ kips

$- \left(\frac{40(12.8)}{178} + 2\right)\sqrt{5700}\,(178)(12.8) = 839$ kips

$- \left(4\sqrt{57000}\right)(178)(12.8) = 688$ kips.

$\Rightarrow V_c = 594$ kips.

$A_{TRIB} = 12.5' \times 12.5' = 156.3\ FT^2$

$q_u = 248$ PSF

$V_u = 248(156.3) = 38.8$ kips.

$\phi V_c = (0.75)(594) = 446$ kips

$\boxed{\phi V_c > V_u \quad \therefore \ \text{ok}}$

BEAM ACTION:

$b_w = 150 \times 2 = 300$ in

$V_c = 2\sqrt{5700}\,(300)(12.8)$

$= 580$ kips.

$\phi V_c = (0.75)(580) = 435$ kips

$\boxed{\phi V_c > V_u = 435 \text{ kips}}$

Appendix B: Plans

Figure B-1: Site Plan

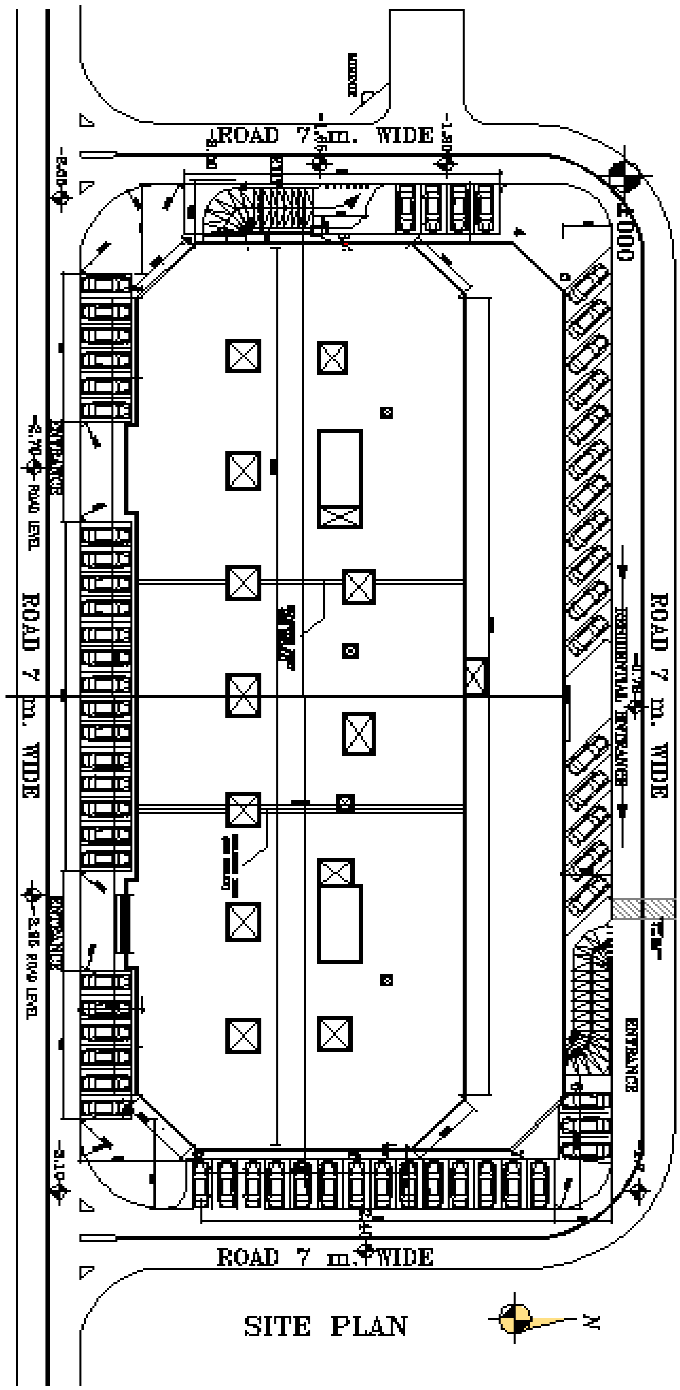


Figure B-2: Ground Floor Plan

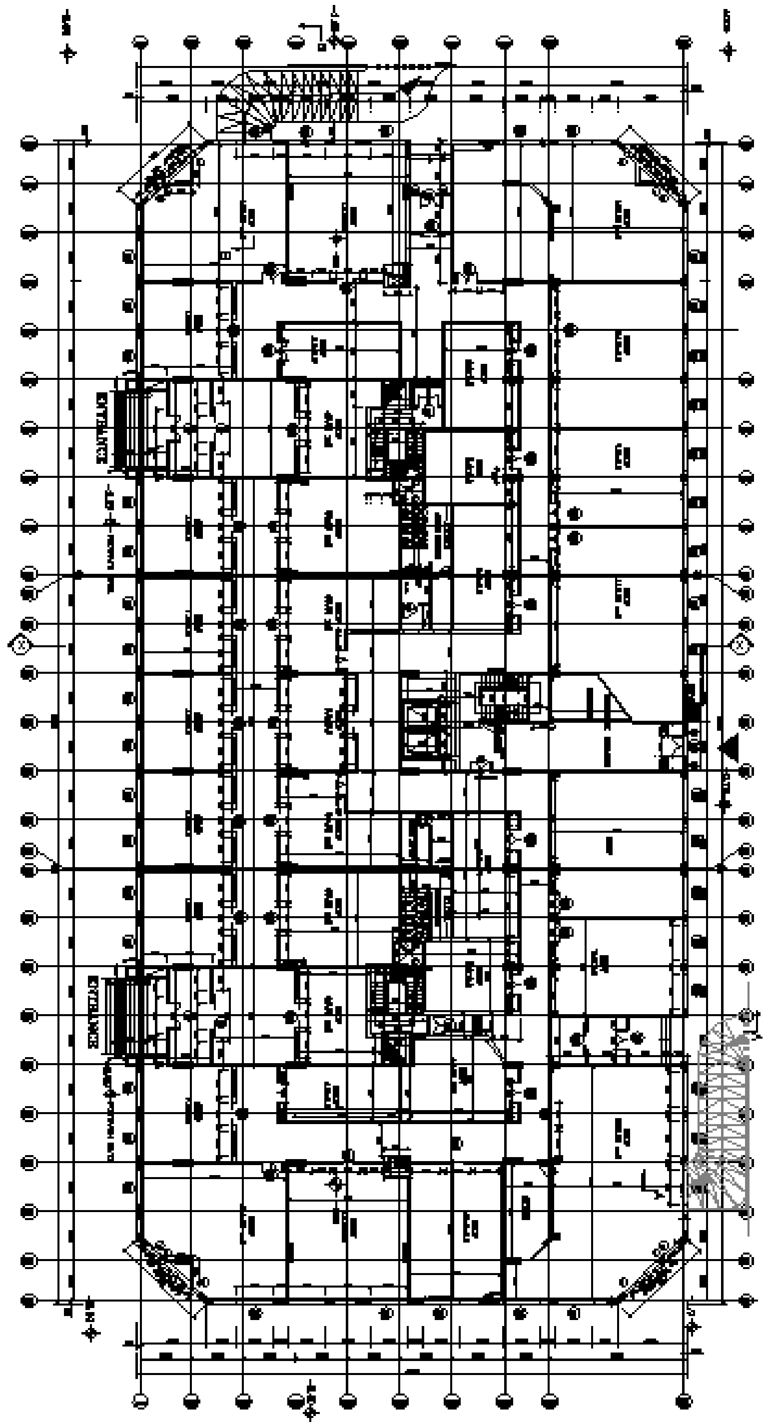
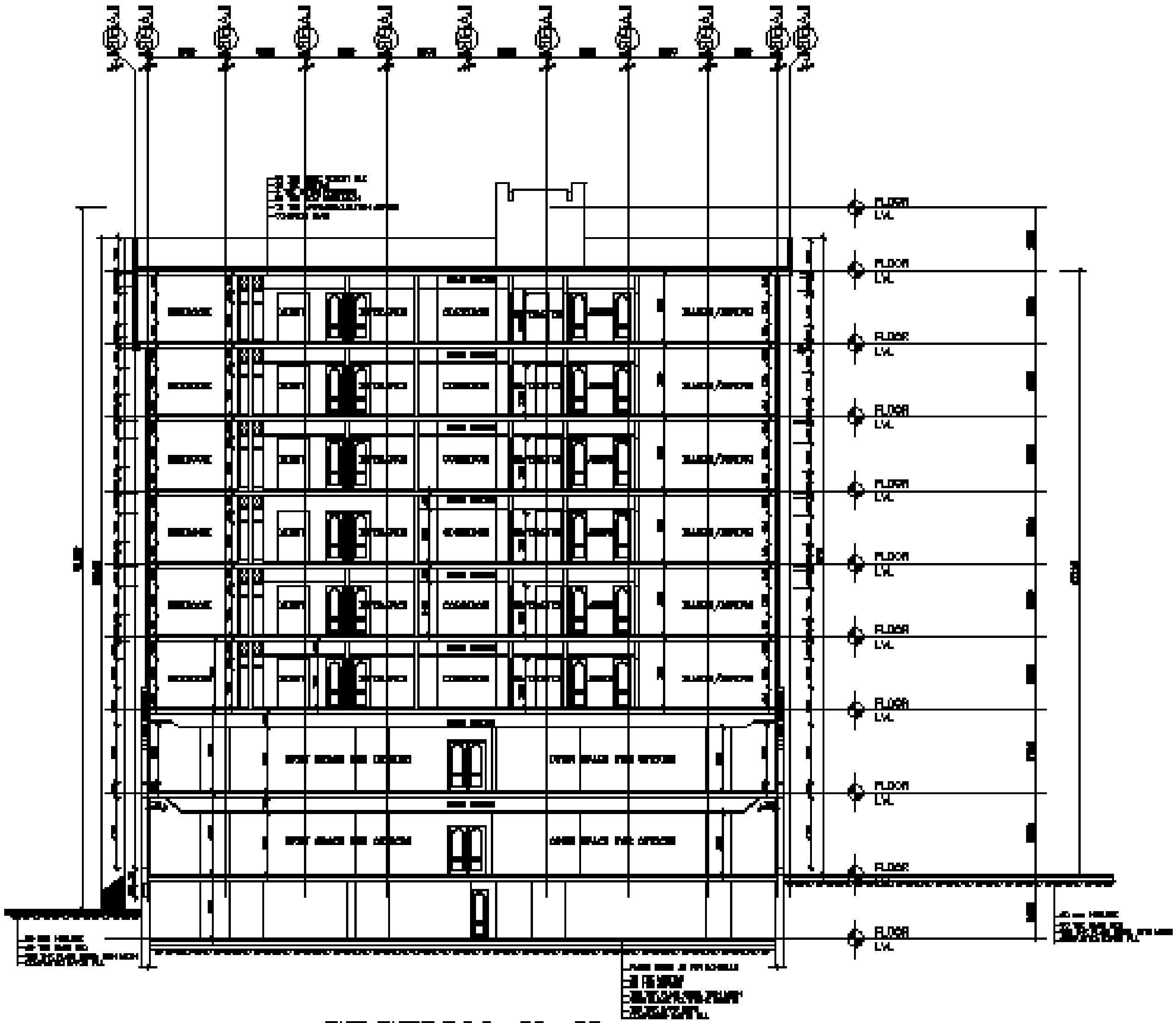
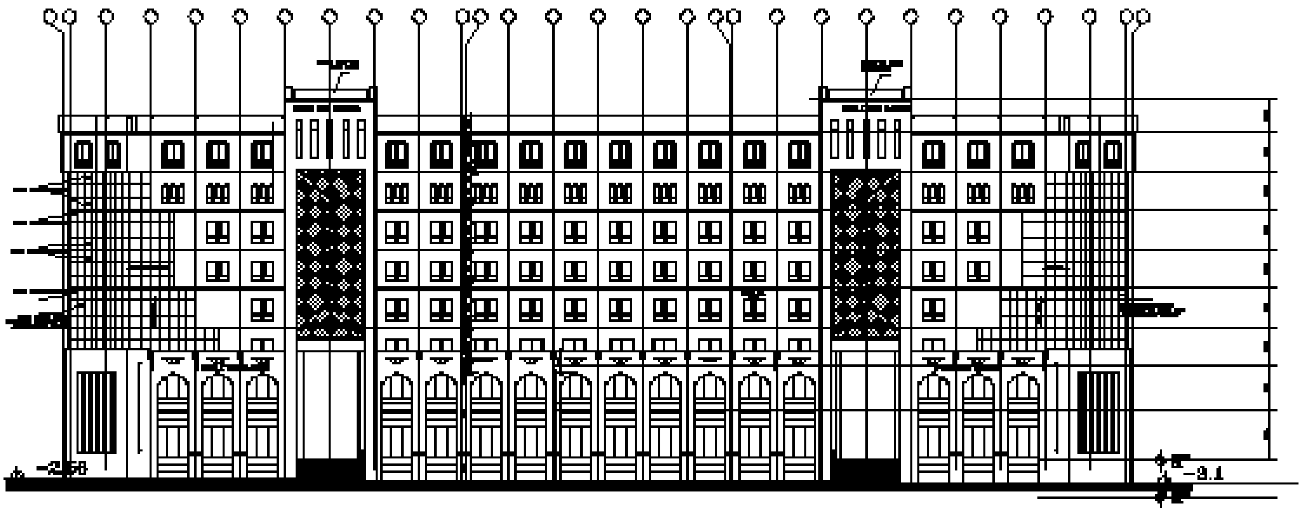


Figure B-3: Building Section (facing west)



SECTION X-X

Figure B-4: South Elevation



FRONT ELEVATION