G.Muttrah Commercial & Residential Complex

Muscat, Sultanate of Oman

Technical Report II

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Structural Option

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Executive Summary

The following report will analyze two existing and two alternative floor framing systems for the G.Muttrah Commercial and Residential Complex in Muscat, The sultanate of Oman. These systems are:

- Flat Slab System
- Two-Way Slab on Beam
- Post-Tensioned Two Way Slab
- Precast Hollow-Core Concrete Planks

The G.Muttrah complex is a reinforced concrete 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.

This report examines the four different floor framing systems while comparing their advantages and disadvantages. The main differentiating characteristics that are discussed are cost, weight, structural depth, difficulty of construction and effect on the architecture or existing conditions. After analysis and comparison it was concluded that the post-tensioned two way slab system is the more efficient alternative floor framing system for the G.Muttrah complex. This is due to its relatively shorter structural depth, lower structural weight and cost of construction. Further details and analysis in this report will help gain a better understanding of this conclusion.

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. A set back of about 35 feet from the north side starts from the fourth floor onwards. 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.

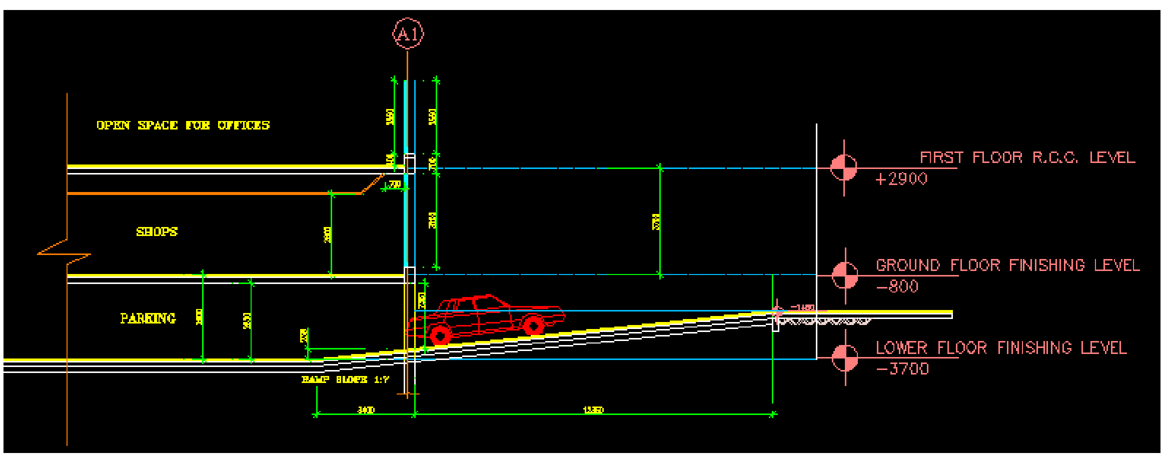

Figure 1: A section showing the entrance of the garage level

Structural System Overview

Summary

The G.Muttrah Commercial & Residential Complex is a reinforced concrete frame building with shear walls. The flooring system consists of 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.

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 2) and range between 14in x 21in to 28in x 47in.

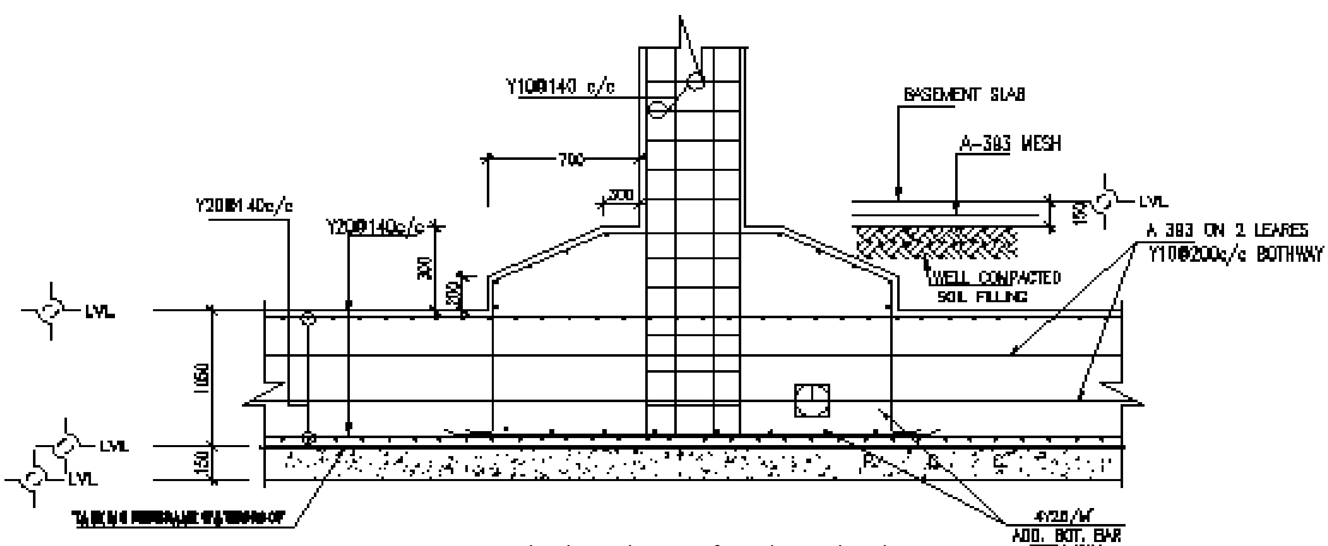

Figure 2: Typical column base at foundation level

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(figure 3) highlight the shear walls within the building:

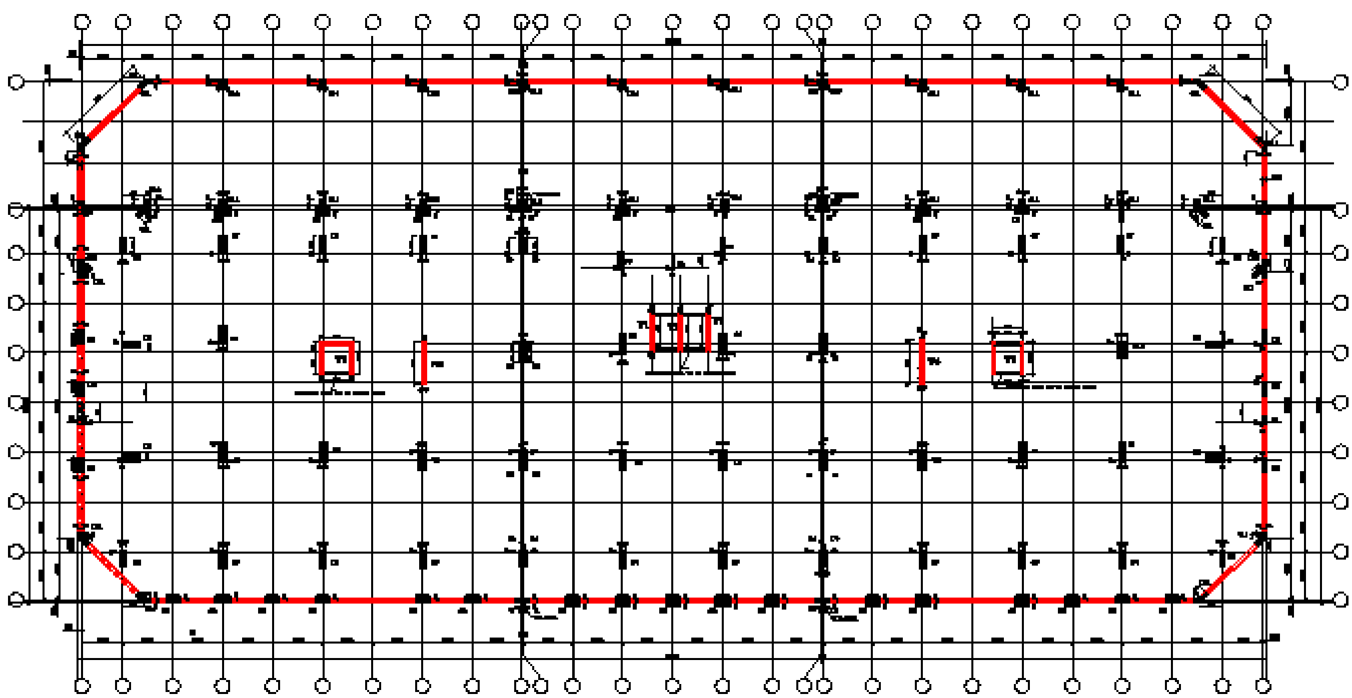

Figure 3: Building Frame showing Shear Walls

Design 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:

Table-1

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

Table-2

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

Existing Floor Framing Plan

There are two types of floor framing systems in the G.Muttrah Complex building; a two-way flat slab system with drop panels in the second and third floor, and a typical two-way reinforced concrete slab on concrete beams system. Figure 4 shows a typical bay in the flat slab system. The spans range between 10 and 30 feet at a regular pattern. All the columns are placed in a rectangular grid and follow the same pattern throughout the entire building producing a more uniform design.

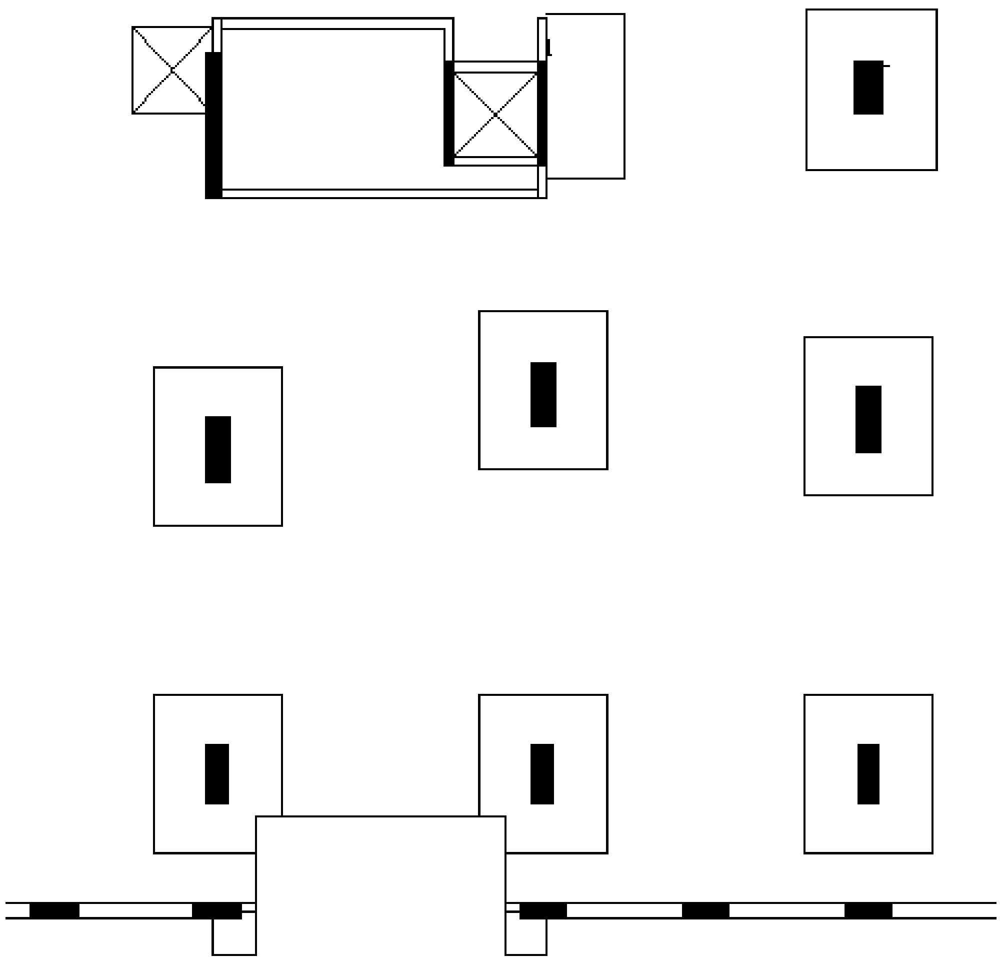

Figure 4: Typical bay in the floors with a flat plate system

Offices are used to occupy the second and third floor which requires many open spaces. The flat slab has a 10 inch slab thickness with a drop panel of 14 inches and reinforcement of #4's (see figure 5).

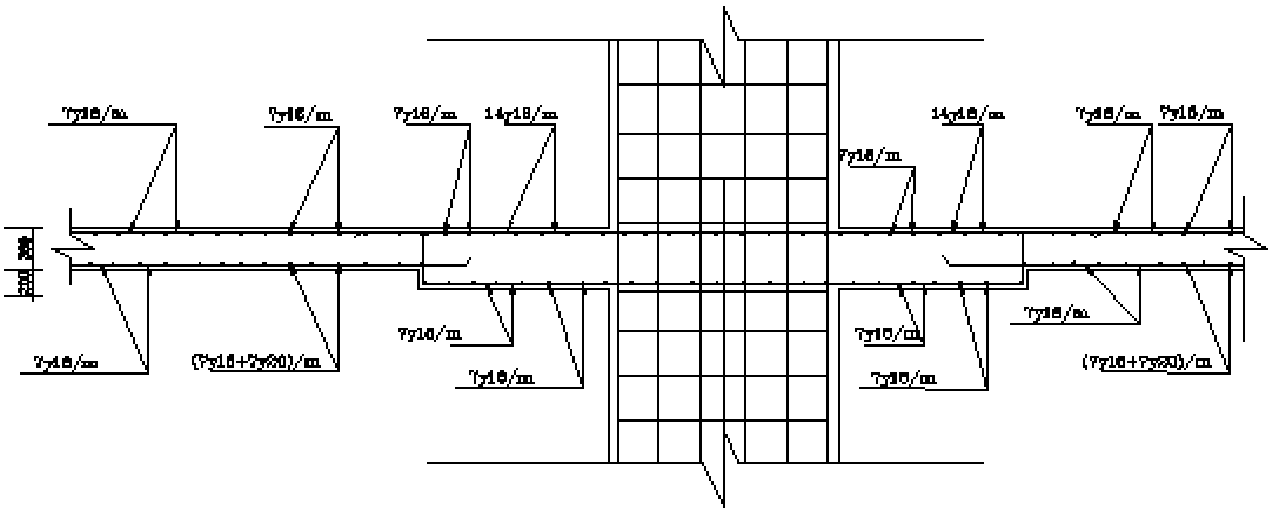

Figure 5: Flat plate slab system (details are in metric units and British rebar size)

As previously mentioned, the rest of the floors have a typical two-way concrete slab on concrete beams system where typical bays are identical to the flate plate system in terms of span. The thicnkess, however, varies from one bay to another in any given floor. Different thickness ranges between 6 and 8 inches. Supporting these slabs are rectangular beams that are 60 inches deep. See figure 6.

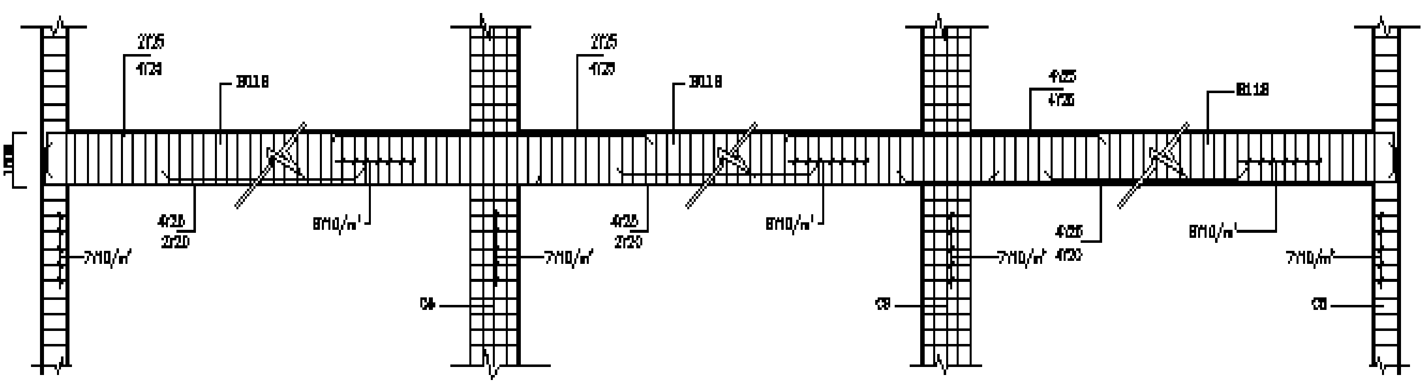

Figure 6: Section of frame on 4th to 8th Floor (details are in metric units and British

Alternative Floor Framing System 1:

The first alternative is to use a consistent flat slab or a two-way slab on beam for the entire building instead of changing systems on different floors. The same designed flat slab will be analyzed for the residential floors (4th to 8th) while the two-way slab on beams will be analyzed for the second and third floors.

Flat slab systems have many advantages that make them very common for residential buildings. The formwork for such a system is relatively easy to lay which results in a much faster and simpler construction process. False ceilings can also be eliminated since the underside of the slab can be used as a ceiling which in return reduces the floor to floor height. The overall reduced height of the building will help bring down the cladding cost and perhaps an extra floor can be added to maximize renting space. There is also more flexibility in designing the occupying space since the partitions are free to be moved anywhere around the space.

One of the disadvantages of this system is the low stiffness which could cause problems with deflection. The flat plate also has low shear capacity and can be critical when considering punching shear.

On the other hand, there is the two-way concrete slab on concrete beams system that has a better shear capacity while also adding stiffness to the frame. This system, however, increases member depth as seen in the original design. The 60 inch deep beams require a false ceiling and also provide difficulties in placing the mechanical systems. In addition, the varying slab thickness designed in this building will complicate the process of setting up the formwork. The weight of the building will also increase if the second and third floors are changed from the flat slab to two-way slab on beams. This could require for greater foundation strengths to carry the additional dead loads.

Since the building has more residential than office spaces, we can conclude that the flat slab floor framing system is a more efficient system for this building than the two-way slab on beams. The reduced floor to floor height combined with the other benefits makes the flat slab a practical and cost-effective way of constructing the G.Muttrah complex.

Alternative Floor Framing System 2:

The second alternative floor framing design is a post-tensioned two-way slab system. In order to simplify the design calculation while also being conservative, a 25ft by 30ft exterior bay was assumed for the entire floor and the analysis produced the following design in figure 7.

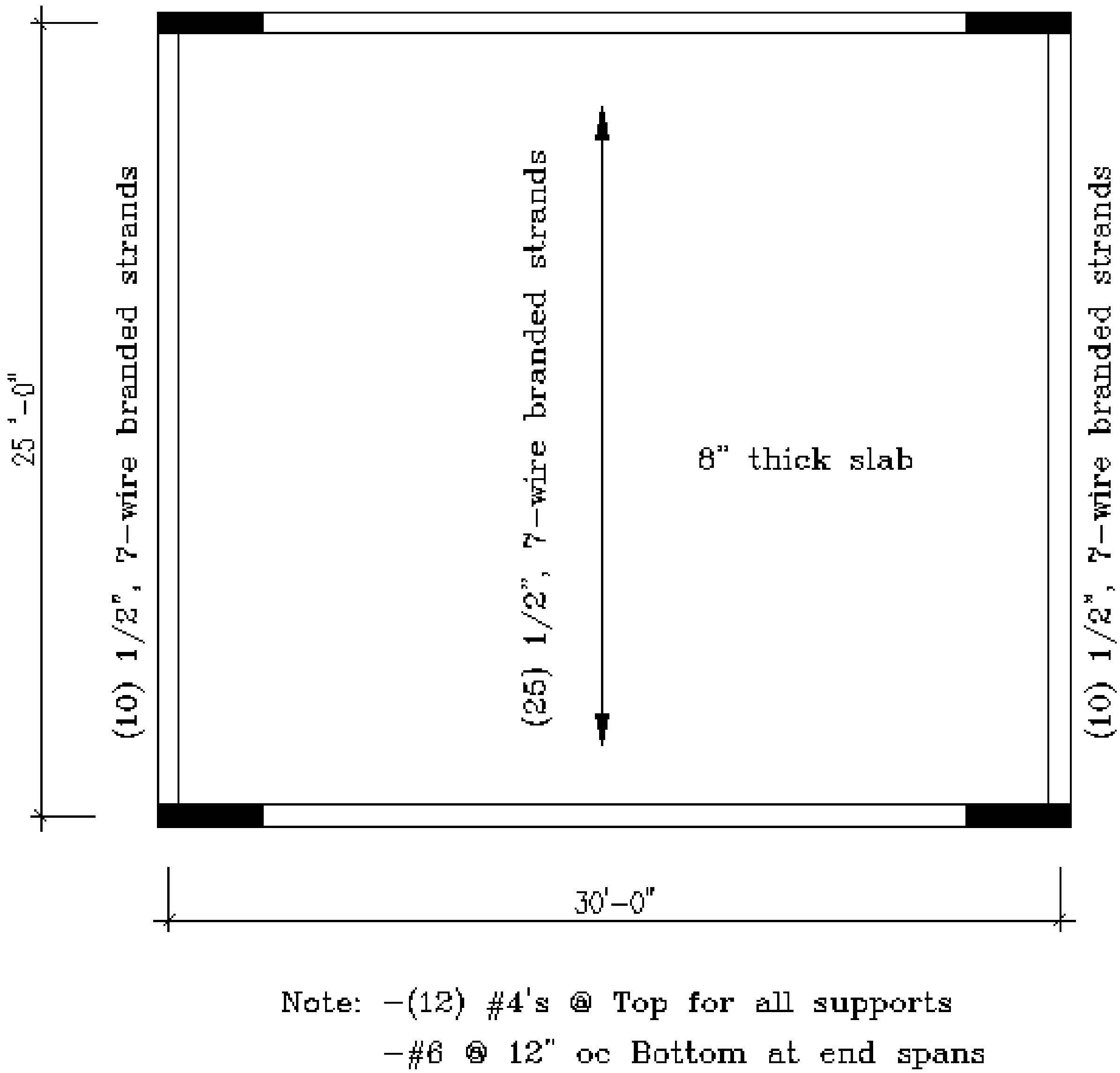

Figure 7: Typical bay designed as post-tensioned two-way slab

The system would contain unbonded tendons, ½" 7-wire strands giving a pre-stress force of about 665 kip. The thickness of the slab will be 8 inches and the reinforcement required is (12) #4's Top at the supports and #6's @ 12" o.c Bottom at end spans. The material used is 5,000 psi strength concrete and 60,000 psi strength steel. The live load was assumed to be 50 psf while the superimposed dead load used was 15 psf. The resulting system has a 2-hour fire rating.

This system will reduce the structural depth of the building while also providing the option of using longer spans. Deflection was not calculated in this report, but the slab thickness was designed using L/H > 35 which considers deflection of the members. Post-tensioned systems are also proficient at vibration control, crack control and water tightness. The formwork for this system is relatively easier to assemble compared to the normal two-way slab on beam system.

A disadvantage of this system is the added labor work while laying the tendons which requires expertise than might not be available at the moment in a region such as the sultanate of Oman. It would not be easy to convince the construction company to build an unfamiliar structure, or it might at least be more costly. Safety can also be an issue since the cables are at very high tension strength.

Further calculations and details on the design can be found in Appendix-A.

Alternative Floor Framing System 3:

The third alternative floor framing system is the precast hollow core concrete plank. A thickness of 8inches plus 2 inches topping was determined using tables provided by Nitterhouse Concrete Products. A typical bay of 30 ft maximum span and a strand pattern of 6-1/2” Ø were used. The safe load from the table was 88 psf which was compared to the designed load of 82 psf. See figure 8.

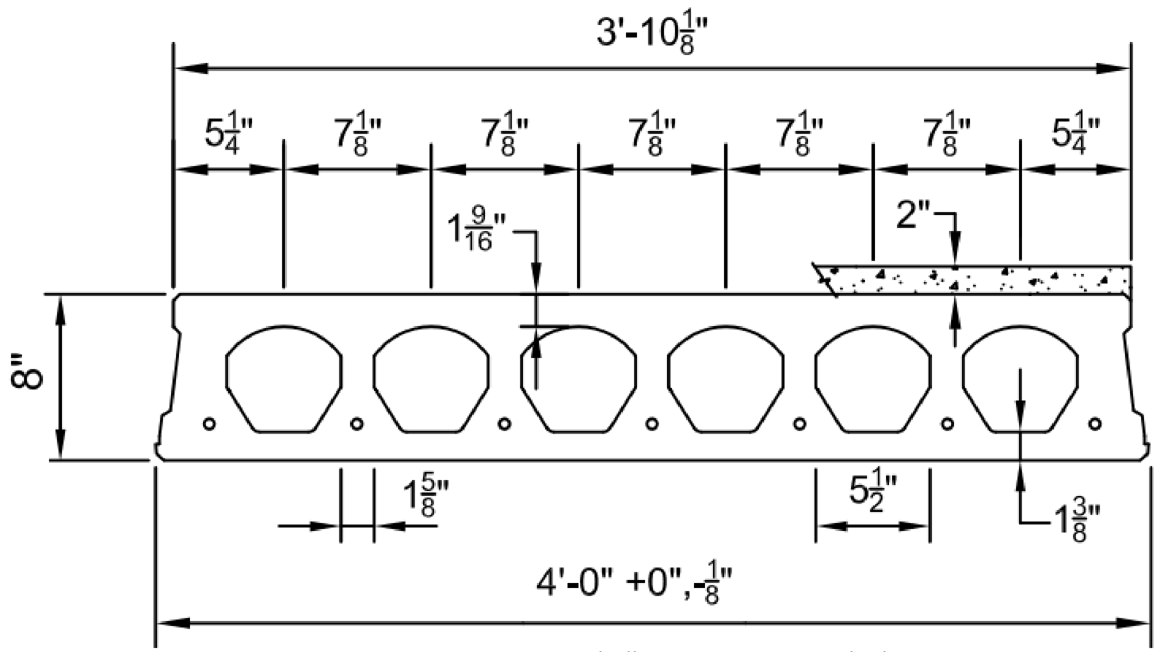

Figure 8: Precast hollow-core concrete plank

The beams carrying the planks are assumed to be W27X84 by inspection. No calculation was required since any beam with significant strength would be sufficient to carry the load. Such a system would be easy to erect while providing 2-hour fire rating. The thinner slabs with steel beams would provide a shorter depth of members and the bottom of the slab can also be used as a ceiling surface.

The columns would however have to be relocated to accommodate the planks that come in set sizes. The corners of the building would also be an issue since smaller custom sizes would have to be produced in order to construct such a system.

Comparison of Systems

Table-3

	Two-Way Slab on Beams	Flat Slab	Post-Tensioned Two-way	Hollow-core Planks
Cost	Medium	Low	Low	High
Weight	150 psf	150 psf	100	74
Depth	60 in (Existing)	10 in (Existing)	8 in	8 in
Fire-Proofing	2-HR	2-HR	2-HR	2-HR
Difficulty of construction	Medium	Easy	Hard	Easy
Effect on Column Grid	Min.	Min	Min.	Major
Viable Alternative	No	Yes	Yes	No

Note: Costs were estimated using RS means and may be different compared to costs in the Sultanate of Oman. Hence a low-medium-high category is used for comparison.

Conclusion

Following the comparison between the four floor framing system and analyzing their advantages and disadvantages, we can conclude that the post-tensioned two way system is the most practical alternative floor framing system. There are risks and costs associated to the construction process of such a system, but a qualified contractor could be assigned to carry out the construction.

The other framing systems had many advantages which were outweighed by their disadvantages. The hollow-core plank was lighter in weight and easier to construct but the rearrangement of column grid and the corners of the building would add to the cost. The post-tensioned systems decreased structural depth, low weight and minimum effect on the architecture and foundation of the building makes it the more feasible alternative which will be investigated as a possible proposal for the new thesis design of the building.

Appendix A: Calculations

Existing: Two-way Flat plate with drop panels

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

$\frac{L_1}{L_2} = \frac{25'}{25'} = 1$, $\alpha_f = 0$

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

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

MIDDLE STRIP MOMENTS

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

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

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

COLUMN STRIP:

used by engineer

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

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

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

$= 274\ \text{FT-K}$

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

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:

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

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

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

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

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

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

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

Existing: Two-way Concrete Slab on Beams

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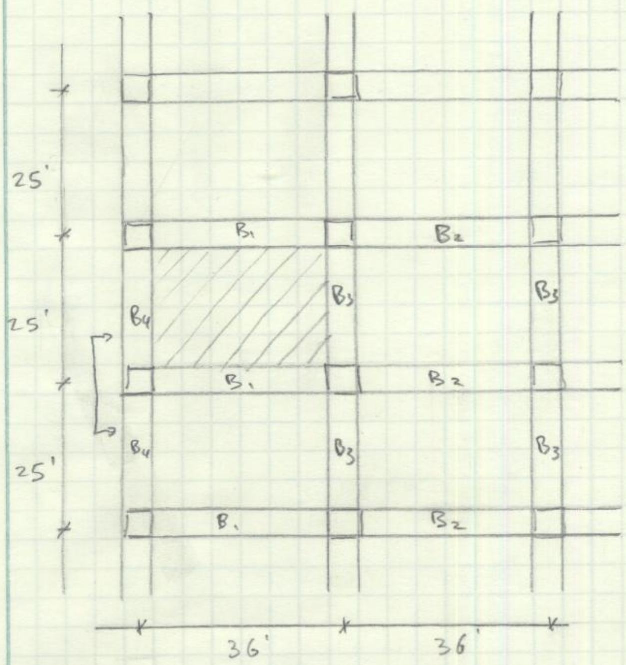
TWO-WAY REINFORCED CONCRETE SLAB ON BEAMS:



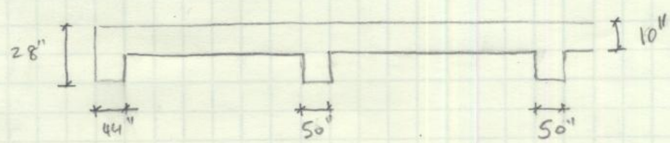
Int col. 12" × 50"

Ext. col. 14" × 44"

SLAB thickness = 10"

All beam width = 16"

NOTE: 36' is Longer than actual 30' span.
⇒ used to simplify estimation process
⇒ conservative



DIRECT DESIGN METHOD:

$$L_2 = 25' \quad , \quad L_n = 36' - \frac{44''}{2 \times 12} - \frac{50}{2 \times 12} = 32'$$

$$W_u = 1.2(140) + 1.6(40) = 232 \text{ PSF}$$

$$M_u = \frac{.232(25)(32)^2}{8} = 742 \text{ k-FT}$$

(2)

Ext SPAN

$M^-_{int} = 0.7(742) = 519 \text{ k-FT}$

$M^+ = 0.57(742) = 423 \text{ k-FT}$

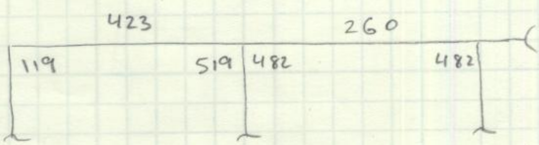
$M^-_{ext} = 0.16(742) = 119 \text{ k-FT}$

INT SPAN

$M^- = 0.65(742) = 482 \text{ k-FT}$

$M^+ = 0.35(742) = 260 \text{ k-FT}$

FRAME :



$$I_{b_1+b_2} = \frac{k\, b_w h^3}{12}$$

$$= \frac{1.5(16)(28)^3}{12}$$

$$= 43,904$$

$$I_{s_1} = \frac{25'(12")(10")^3}{12}$$

$$= 25,000 \text{ in}^4$$

$b_e \le b_w + 2h_w = 16 + 2(18)$

$= 52$

$\le b_w + 4t = 40"$ ⇒ governs

$b_e/b_w = \frac{40}{16} = 2.5$

$t/h = \frac{10}{28} = 0.357$

⇒ $k = 1.5$

$$\alpha_{b_1} = \alpha_{b_2} = \frac{I_{b_1+b_2}}{I_{s_1}} = \frac{43,904}{25,000} = 1.76$$

$$I_{b_3} = I_{b_1+b_2} = 43,904 \text{ in}^4$$

$$I_{s_2} = \frac{36'(12")(10^3)}{12} = 36000$$

$$\alpha_{b_3} = \frac{43,904}{36000} = 1.22$$

3

$$I_{b_4} = \frac{1.35(16)(28)^3}{12} = 39514$$

$$b_e \leq 16 + 18 = 34''$$
$$\leq 16 + 4(6) = 40''$$

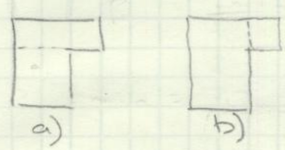
$$b_e/b_w = 2.13$$
$$t/h = 0.357$$
$$\Rightarrow K = 1.35$$

$$I_s = \frac{18'(12'')(10^3)}{12} = 18{,}000 \text{ in}^4$$

$$\alpha_{b_4} = \frac{39{,}514}{18{,}000} = 2.2$$

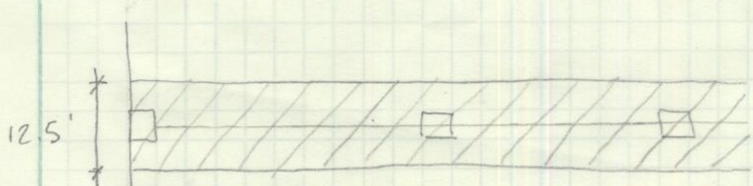
$$\alpha_m = \alpha_{avg.} = \frac{1.76 + 1.76 + 1.22 + 2.2}{4} = 1.74$$

TORSIONAL CONSTANT :



$$a)\ C = \left(1 - 0.63\tfrac{16}{18}\right)\left(\frac{16^3(18)}{3}\right) + \left(1 - 0.63\tfrac{10}{34}\right)\left(\frac{10^3(34)}{3}\right)$$
$$= 10813 + 9233 = 20046$$

$$C = \left(1 - 0.63\tfrac{16}{28}\right)\left(\frac{16^3(28)}{3}\right) + \left(1 - 0.63\tfrac{10}{18}\right)\left(\frac{10^3(18)}{3}\right)$$
$$= 24467 + 3900 = 28{,}367 \Rightarrow \text{governs.}$$



COLUMN STRIP

(4)

$$\beta_t = C_2 / 2 I_{s1} = \frac{28,367}{25,000} = 1.13$$

$$\text{Aspect Ratio} = \frac{25}{36} = .694$$

$\alpha_{b1} \, l_2/l_1 = 1.76(.694) = 1.22 > 1.0$ ∴ 85% column strip goes to beam.

Ext SPAN

l_2/l_1	0.5	.694	1.0
$\beta_t = 0$	100%	100%	100%
$\beta_t = 1.13$		92.8	
$\beta_t \geq 2.5$	90%	84.18%	75%

$M^-_{ext} = 119 \Rightarrow$ 92.8 to column strip = 110 K-FT
7.98 to middle strip = 9 K-FT

85% of 110 from C.S goes to beam = 93.5 K-FT
15% of 110 from C.S goes to Slab = 16.5 K-FT

l_2/l_1	0.5	.694	1.0
	90%	82.24%	70%

$M^+ = 423 \Rightarrow$ 82.24% to C.S = 348 K-FT
$\Rightarrow$ 17.76 to M.S = 75 K-FT

85% of 348 goes to beam = 296 K-FT
15% of 348 goes to Slab = 52 K-FT

$M^- = 519 \Rightarrow$ 82.24% to C.S = 427 K-FT
$\Rightarrow$ 17.76% to M.S = 92.2 K-FT

85% of 427 goes to beam = 363 K-FT
15% of 427 goes to Slab = 64.1 K-FT

5

REINFORCEMENT DESIGN

ITEM	DESCRIPTION	M^-_{ext}	M^+	M^-_{int}
1.	M_u (K-FT)	-16.5	52	-64.1
2.	Column strip (in) b	150"	150"	150"
3.	Effective depth	8.31"	8.31"	8.31"
4	Moment / ft $M_u(12")/b$	-1.26	4.16	-5.13
5.	$M_n = M_u/\phi$	-18.3	57.8	-71.2
6.	$R = \frac{M_n(12000)}{bd^2}$	21.2	67.0	82.5

$$\phi M_n = \phi \rho f_y bd^2(1-0.59\rho f_y/f'_c)$$
$$= 0.9(0.0035)(65)(150)(8.31)^2(1-0.59(0.0035)(\tfrac{65}{5.7}))$$
$$= 2070 \text{ in-K} = 172 \text{ K-FT} > 52 \text{ K-FT} \checkmark \text{ ok}$$

Deflection control

$$\beta_1 = \frac{32}{21} = 1.52$$

$$t_{min} = \frac{L_n(0.8 + f_y/200{,}000)}{36+9\beta}$$
$$= \frac{32(12)(0.8+\frac{60000}{200{,}000})}{36+9\beta}$$
$$= 8.5" < 10" \checkmark \text{ ok}$$

6

SHEAR:

$w_u = 1.2(140) + 1.6(40) = 232$ psf.

$V_u = (.232)(1')\left(12.5 - \frac{16"/12"}{2} - \frac{8.3"}{12"}\right)$

$= 2.58$ kips.

$\phi V_c = \phi 2 \sqrt{f_c'}\, bd = 0.75(2)(\sqrt{5700})(12")(8.3)/1000$

$= 11.3$ kips $> V_u = 2.58$ ✓ ok

Post-Tensioned Two Way Slab

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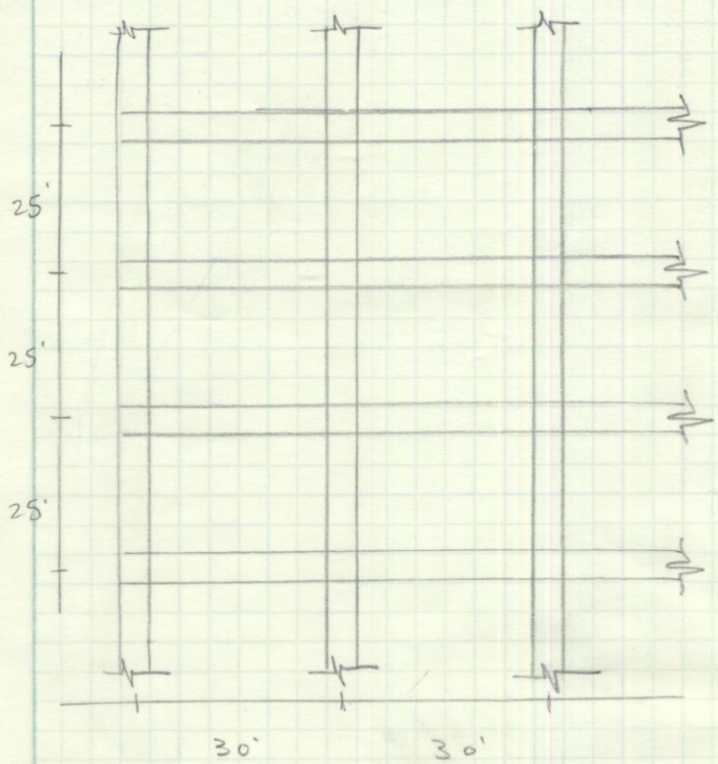


Int. Col 12" 50"

Ext. Col. 14" 44"

ASSUME 5 SPANS OF 30 FT MAX SPAN.

Loads:

Framing Dead Load = Selfweight.

Superimposed = 15 PSF

LIVE LOAD = 50 PSF

2-HOUR FIRE RATING

MATERIALS:

CONCRETE: NORMAL WEIGHT 150 PCF

f'_c : 5,000 psi

f'_{ci} : 3,000 psi

Rebar:

f_y = 60,000 psi

2

PT :

UNBONDED TENDONS

$\frac{1}{2}$" ϕ, 7-wire strands, $A = 0.153$ in^2

$f_{pu} = 270$ ksi

ESTIMATED PRESTRESS LOSSES = 15 ksi

$f_{se} = 0.7(270 \text{ ksi}) - 15 \text{ ksi} = 174$ ksi

$P_{eff} = A * f_{se} = (0.153)(174) = 26.6$ kips / tendon

ASSUME $L/H = 45$

Longest Span = 30 ft

$h = (30)(12)/45 = 8.0"$ preliminary slab thickness

$DL = (8 in)(150 PCF) = 100$ psf

SIDL = 15 psf

LL_o = 50 psf

TWO-WAY SLAB DESIGNED AS CLASS U (ACI 18.3.3)

$A = bh = (25')(12")(8in) = 2{,}400 \text{ in}^2$

$S = \frac{bh^2}{6} = \frac{(25')(12)(8 in)^2}{6} = 3{,}200 \text{ in}^3$

DESIGN PARAMETERS:

ALLOWABLE STRESSES: CLASS U

AT TIME OF JACKING (ACI 18.4.1)

$f_c' = 3{,}000$ psi

compression $= 0.6 f_c' = 0.6(3{,}000 \text{ psi}) = 1{,}800$ psi

Tension $= 3\sqrt{f_c'} = 3\sqrt{3{,}000} = 164$ psi

3

AT SERVICE LOADS (ACI 18.4.2(a) & 18.3.3)

$f'_c = 5,000$ psi

Compression $= 0.45 f'_c = 0.45(5,000 \text{ psi})$
$= 2,2500$ psi

Tension $= 6\sqrt{f'_c} = 6\sqrt{5,000} = 424$ PSI

AVERAGE PRECOMPRESSION LIMITS:

$P/A = 125$ psi min (ACI 18.12.4)

$= 300$ psi max.

TARGET load Balances:

75% of DL (selfweight) for slabs.

$0.75(W_{DL}) = 0.75(100) = 75$ PSF

COVER REQUIREMENTS (2-HR FIRE RATING, ASSUME CARBONATE AGGREGATE)

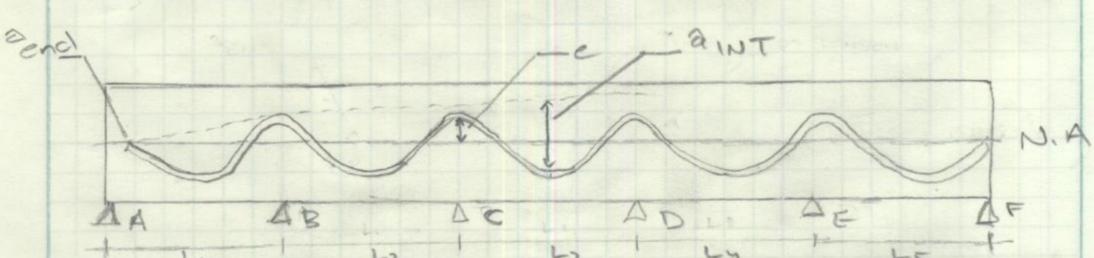
RESTRAINED SLABS = 3/4" bottom

UNRESTRAINED SLABS = 1 1/2" bottom

= 3/4" top

TENDON PROFILE:

PARABOLIC SHAPE:
FOR A LAYOUT WITH SPANS OF SIMILAR LENGTH, THE TENDONS WILL TYPICALLY BE LOCATED AT THE HIGHEST ALLOWABLE POINT AT THE INTERIOR COLUMNS, THE LOWEST POSSIBLE POINT AT THE MIDSPANS, AND THE NEUTRAL AXIS AT THE ANCHOR LOCATIONS. THIS PROVIDES THE MAXIMUM DRAPE FOR LOAD-BALANCING.



CONTINUOUS POST-TENSIONED BEAM

4

Tendon Ordinate	Tendon (CG) Location*
EXTERIOR SUPPORT - ANCHOR	4.0"
INTERIOR SUPPORT - TOP	7.0"
INTERIOR SPAN - BOTTOM	1.0"
END SPAN - BOTTOM	1.75"

((G) = center of gravity

* Measure from bottom of slab

$a_{INT} = 7.0'' - 1.0'' = 6.0''$

$a_{END} = (4.0'' + 7.0'')/2 - 1.75' = 3.75''$

$W_b = 0.75\,W_{DL} = 0.75\,(100\text{ PSF})(25\text{ FT}) = 1{,}875\text{ plf} = 1.875\text{ k/FT}$

$P = W_b L^2 / 8 a_{end} = (1.875)(30)^2 / [8(3.75/12)] = 675\text{ k}$

$\#\text{ of tendons} = \dfrac{675\text{ k}}{26.6\text{ k/TENDON}} = 25.38 \Rightarrow$ USE 25 TENDONS

$P_{actual} = (25)(26.6) = 665\text{ k}$

$W_b = \left(\dfrac{665}{675}\right)(1.875) = 1.85\text{ k/FT}$

$\dfrac{P_{ACTUAL}}{A} = \dfrac{(665\text{ k})(1000)}{2{,}400\text{ in}^2} = 277\text{ PSi}$ > 125 psi min ok
< 300 psi max ok.

$W_b = \dfrac{(665)(8)(6/12)}{30^2} = 2.96\text{ k/FT}$

$W_b / W_{DL} = 116\%$ => ADJUST INTERIOR DRAPE TO SUPPORT 95% of SELF DL.

5

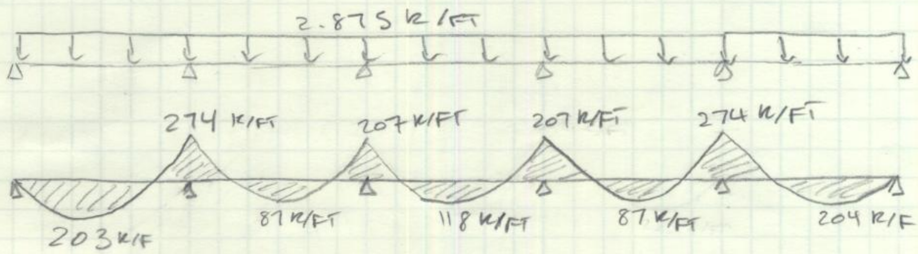
$$a = 12\left[\frac{2.375(30)^2}{665(8)}\right] = 4.82 \Rightarrow 5 \text{ in Drape}$$

$$w_b = \frac{665(8)\left(\frac{5}{12}\right)}{30^2} = 2.46 \Rightarrow 98\% \text{ of DL ok}$$

$\boxed{\text{EFFECTIVE PRESTRESS FORCE, } P_{eff} = 665 \text{ kips}}$

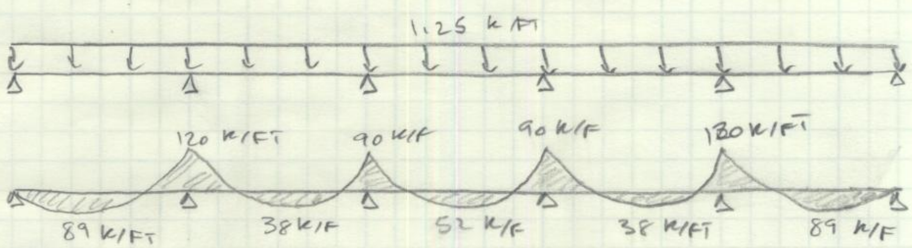
DEAD LOAD MOMENTS:

$w_{DL} = (115)(25)/1000 = 2.875$ plf



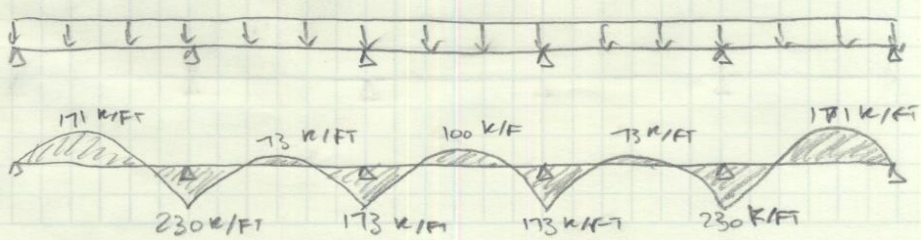
LIVE LOAD MOMENTS:

$w_{LL} = (50 \text{ psf})(25)/1000 = 1.25$ plf



TOTAL BALANCING MOMENTS, M_{BAL}

$w_b = -2.46$ k/ft



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STAGE 1: STRESSES IMMEDIATELY AFTER JACKING (DL + PT)

MIDSPAN STRESSES

$f_{top} = (-M_{DL} + M_{bal})/S - P/A$

$f_{bottom} = (+M_{DL} - M_{bal})/S - P/A$

Interior Span

$f_{top} = [(-118 + 100)(12)(1000)]/3{,}200 - 277$

$= -345$ psi compression $< 0.6 f'_{ci} = 1800$ psi ok

$f_{bot} = [(118 - 100)(12)(1000)]/3{,}200 - 277$

$= -210$ psi compression $< 0.6 f'_{ci} = 1800$ psi ok

END SPAN

$f_{top} = [(-203 + 171)(12)(1000)]/3{,}200 - 277$

$= -397$ psi compression $< 0.60 f'_{c} = 1800$ psi ok

$f_{bot} = [(203 - 171)(12)(1000)]/3{,}200 - 277$

$= -157$ psi compression $< 0.60 f'_{c} = 1800$ psi ok

SUPPORT STRESSES

$f_{top} = (+M_{DL} - M_{bal})/S - P/A$

$f_{bot} = (-M_{DL} + M_{bal})/S - P/A$

$f_{top} = [(274 - 230)(12)(1000)]/3{,}200 - 277$

$= -112$ psi compression $< 0.6 f'_{ci} = 1800$ psi ok

$f_{bot} = [(-274 + 230)(12)(1000)]/3{,}200 - 277$

$= -442$ psi compression $< 0.6 f'_{ci} = 1800$ psi ok

STAGE 2: STRESSES at Service load (DL+LL+PT)

MIDSPAN

$f_{top} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{bot} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

7

Interior Span

$f_{top} = [(-118-52+100)(12)(1000)]/3200 - 277$

$= -540$ psi compression $< 0.45f'_c = 2{,}250$ psi ok

$f_{bot} = [(118+52-100)(12)(1000)]/3200 - 277$

$= -14.5$ psi compression $< 0.45f'_c = 2{,}250$ psi ok

END SPAN

$f_{top} = [(-203-89+171)(12)(1000)]/3200 - 277$

$= -731$ psi compression $< 0.45f'_c = 2{,}250$ psi ok

$f_{bot} = [(203+89-171)(12)(1000)]/3200 - 277$

$= 176$ psi tension $< 6\sqrt{f'_c} = 424$ psi ok

SUPPORT STRESSES

$f_{top} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

$f_{bot} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{top} = [(274+120-230)(12)(1000)]/3200 - 277$

$= 338$ psi Tension $< 6\sqrt{f'_c} = 424$ psi ok

$f_{bot} = [(-274-120+230)(12)(1000)]/3200 - 277$

$= -892$ psi compression $< 0.45f'_c = 2{,}250$ psi ok

$\Rightarrow$ ALL STRESSES ARE WITHIN THE PERMISSIBLE CODE LIMITS.

ULTIMATE STRENGTH

$M_1 = P^*e$

$e = 0$ in at the exterior support
& 3.0 in at the interior support.

$M_1 = 665(3)/12 = 166$ FT-K

$M_{sec} = M_{bal} - M_1$

$= 230 - 166 = 64$ FT-K AT INTERIOR SUPPORTS.

8

$M_u = 1.2\,M_{DL} + 1.6\,M_{LL} + 1.0\,M_{sec}$

@ MIDSPAN: $M_u = 1.2(118) + 1.6(52) + 1.0(32)$
$= 257$ ft-k

@ SUPPORT: $M_u = 1.2(-274) + 1.6(-120) + 1.0(64)$
$= -457$ ft-k

DETERMINE MINIMUM BONDED REINF.

POSITIVE MOMENT REGION:

Interior span: ALL COMPRESSION ⇒ NO POSITIVE REINF REQUIRED

END SPAN: $f_t = 176$ psi $> 2\sqrt{f'_c} = 141$ psi

$Y = f_t / (f_t + f_c)\,h$
$= [(176)/(176 + 731)(8)$
$= 1.55$

$N_c = M_{DL+LL} / S * 0.5 * Y * L_2$
$= [(203 + 89)(12)/3,200]\,(0.5)(1.55)(25)(12)$
$= 255$ k

$A_{s,min} = N_c / 0.5 f_y$
$= 255 / [0.5(60 \text{ ksi})]$
$= 8.5 \text{ in}^2$

$A_{s,min} = (8.5)/25$
$= .34 \text{ in}^2/\text{FT}$

use #6 @ 12 in oc Bottom $= 0.44 \text{ in}^2/\text{ft}$

Negative moment region:

$A_{s,min} = 0.00075\,A_{cf}$

9

Interior Supports:

$A_{cf} = (8)[30](12)$

$= 2880 \text{ in}^2$

$A_{s,min} = 0.00075(2880)$

$= 2.16$

$\Rightarrow$ (11) #4's Top (2.20 in^2)

Exterior Supports: Same as interior

- MUST SPAN A MIN of 1/6 CLEAR SPAN ON EACH SIDE of SUPPORT
- AT LEAST 4 BARS REQUIRED IN EACH DIRECTION
- PLACE TOP BARS WITHIN 1.5h away From THE FACE OF THE SUPPORT ON EACH SIDE

$= 1.5(8)$

$= 12 \text{ in}$

max bar spacing IS 12"

$M_u = (A_s f_y + A_{ps} + f_{ps})(d - a/2)$

$A_{ps} = 0.153$ (Number of tendons)

$= 0.153(25)$

$= 3.83 \text{ in}^2$

$f_{ps} = f_{se} + 10{,}000 + (f_c' bd)/(300 A_{ps})$

$= 174{,}000 + 10{,}000 + [(5{,}000)(25(12))d]/[(300)(3.83)]$

$= 184{,}000 + 1305d$

$a = (A_s f_y + A_{ps} f_{ps})/(0.85 f_c' b)$

10

AT SUPPORTS

$d = 8 - 3/4 - 1/4 = 7"$

$f_{ps} = 184{,}000 + 1305(7) = 193{,}135$

$a = [(2.20)(60) + (3.83)(193)] / [(0.85)(5)(25 \times 12)]$

$= .68$

$\phi M_n = 0.9[(2.20)(60) + (3.83)(195)][7 - .68/2]/12$

$= 439 < 457$ ft-k $\Rightarrow$ by inspection $A_s = 2.4$ ok

$\phi M_n = 490$ ft-k

$\boxed{\text{(12) \#4's TOP at ALL SUPPORTS}}$

$d = 8 - 1\frac{1}{2} - \frac{1}{4} = 6\frac{1}{4}"$

$f_{ps} = 184{,}000 + 1305(6.25) = 192156$ psi

$a = [(8.5)(60) + (3.83)(192)] / [0.85(5)(25 \times 12)]$

$= .98$

$\phi M_n = 0.9[(8.5)(60) + 3.83(192)][6.25 - \frac{.98}{2}]/12$

$= 538$ Ft-k > 257 ok

$\boxed{\text{\#6 @ 12" oc BOTTOM at end SPANS}}$

Precast Hollow-Core Concrete Plank

SAMIR AL-AZRI | TECH II REPORT | 1

PRESTRESSED CONCRETE HOLLOW CORE PLANK

MAX SPAN = 30'

USE 8" x 4'-0" HOLLOW CORE PLANK

STRAND PATTERN ⟹ 6 – ½" Ø

SAFE LOAD = 88 PSF

$W_u = 1.2(15) + 1.6(40) = 82$ PSF < 88 PSF

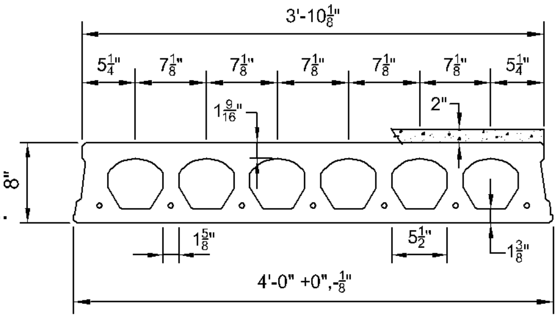
⟹ OK ✓

Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 301 in.^{2} Precast b_w = 13.13 in.
I_c = 3134 in.^{4} Precast S_{bcp} = 616 in.^{3}
Y_{bcp} = 5.09 in. Topping S_{tct} = 902 in.^{3}
Y_{tcp} = 2.91 in. Precast S_{tcp} = 1076 in.^{3}
Y_{tct} = 4.91 in. Precast Wt. = 245 PLF
Precast Wt. = 61.25 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 92.3 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 147.8 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	248	214	185	159	138	118	102	87	74	62	52	42						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	239	211	187	165	146	129	114	101	88	77	67	58	50	42
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	181	161	144	128	114	101	90	79	70	61

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

8SF2.0T

Appendix B: Plans

Figure B-1: Site Plan

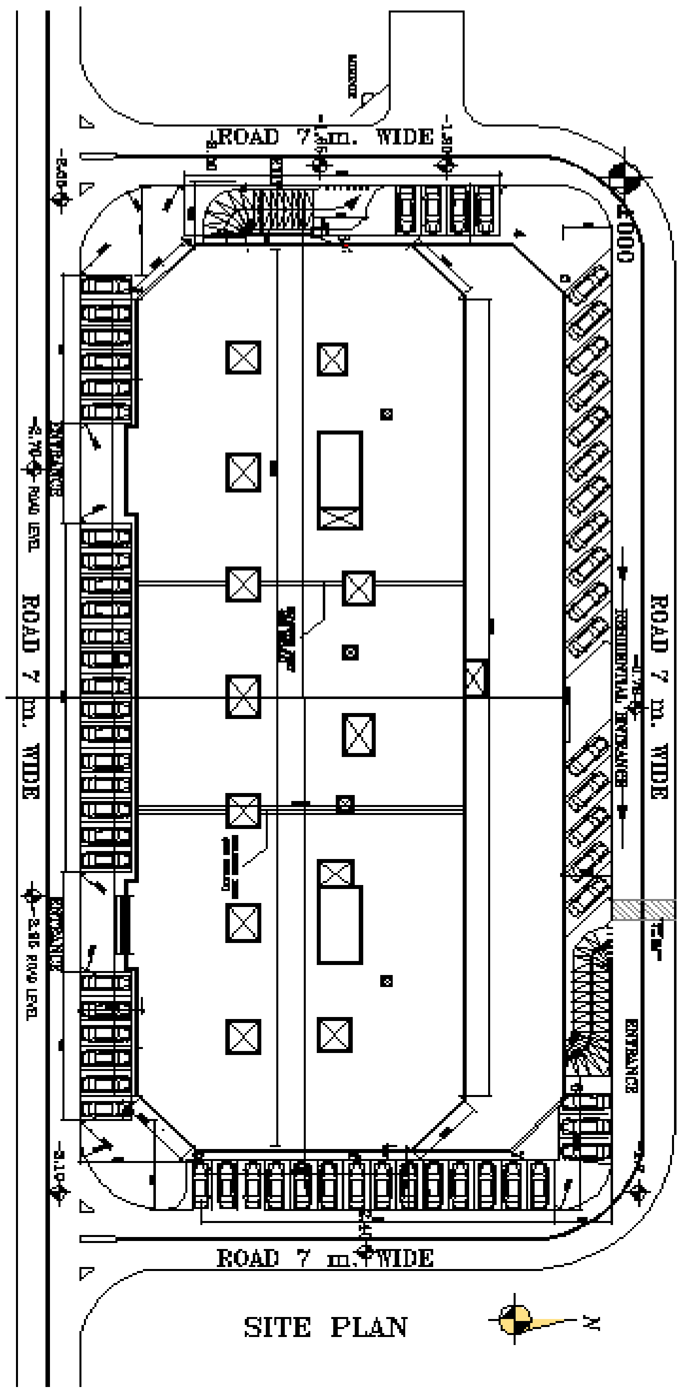


Figure B-2: Ground Floor Plan

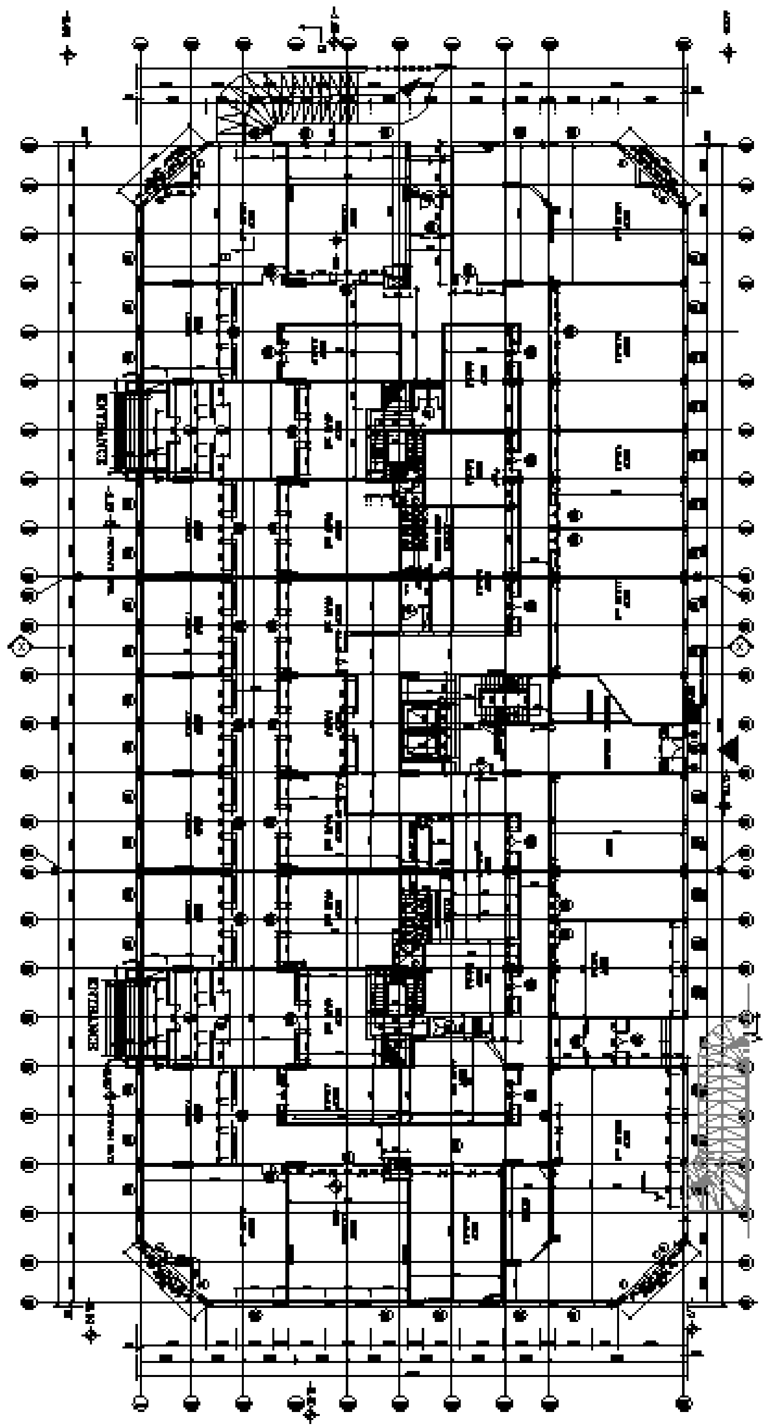


Figure B-3: Building Section (facing west)

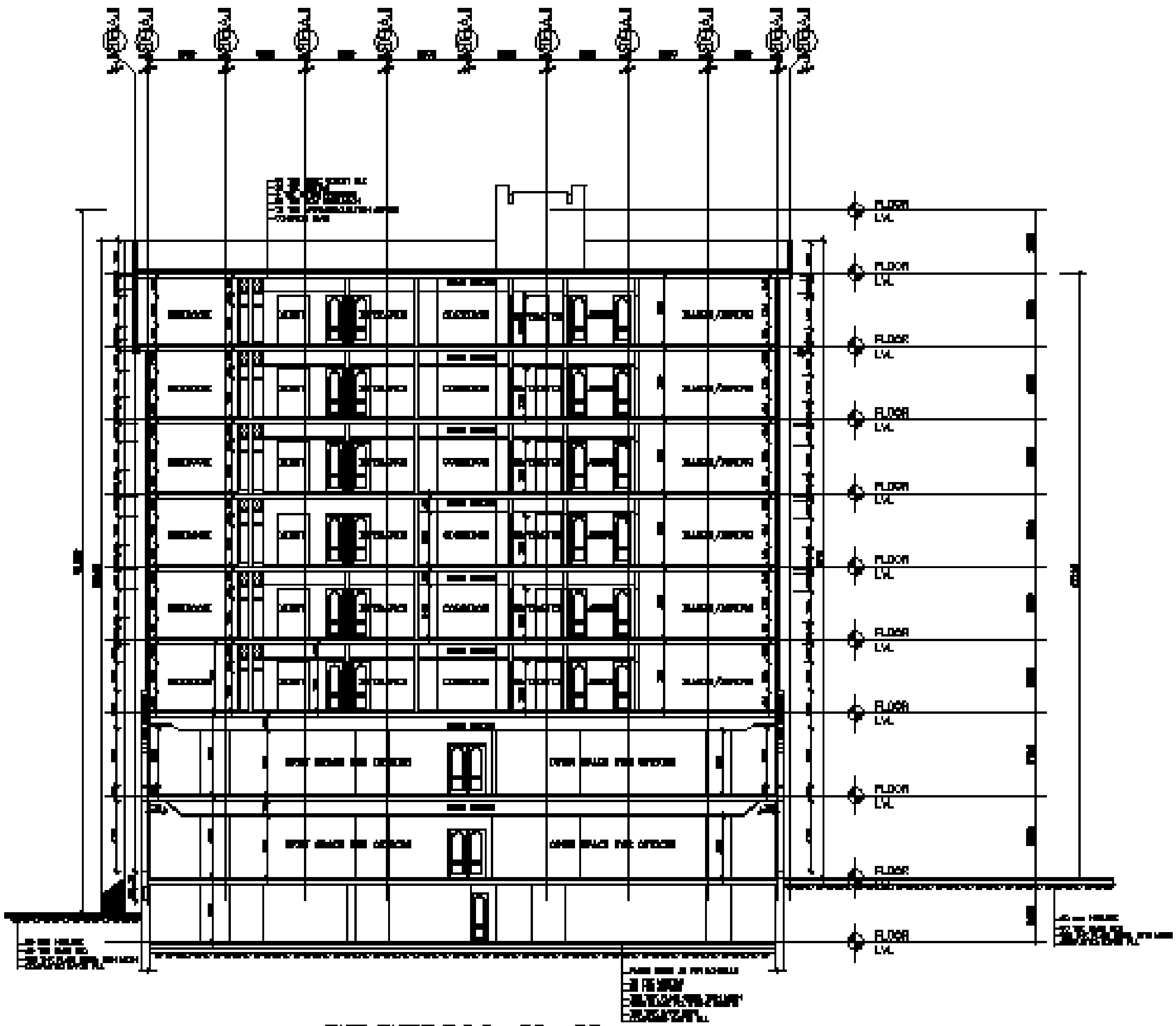
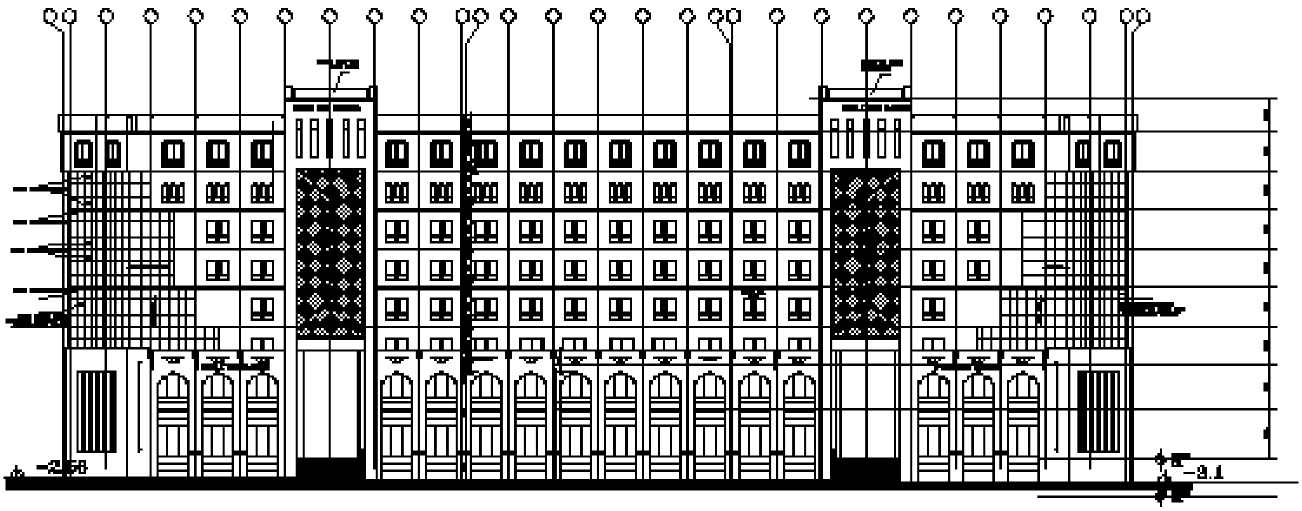


Figure B-4: South Elevation



FRONT ELEVATION