G.Muttrah Commercial & Residential Complex

Muscat, Sultanate of Oman

Final Report

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Structural Option

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G.Muttrah Commercial & Residential Complex

Building
G. Muttrah Commercial & Residential Complex

Location
Muscat, Sultanate of Oman

Occupants
Residential, Offices and Retail

Size
25900 SQM

Stories
8 Floors + Basement Parking

Owner
Salim Alshanfari
Jamal Alshanfari

Project Team

Architect: Engineering Studies Office

Engineers: Engineering Studies Office

CM: Al Rawahi International Company LLC

Geotechnical: Oman Drilling and Soil Technology Co. LLC

Architecture

G.Muttrah Commercial & Residential Complex is a mixed use building in a commercially developing region in the city of Muscat. Previously known as a tourist attraction, the Great Muttrah area has developed into one of the city's busiest commercial district. The G.Muttrah Commercial & Residential Complex is one of the many mixed used building in construction at moment. The building will include retail in the ground floor, offices in the second floor and 96 apartments in the rest of the 6 floors. A parking garage in the basement will serve 115 slots for the tenants due to the limited parking spaces in the area. Tenants on the 8th floor can be lucky enough to get a view of the ocean.

Construction

The project delivery method is a design-bid-build. The project cost is estimated at around $15.5M. Excavation is scheduled to start in September 2009 and the construction phase is expected to last 24 months.

MEP System

- Horizontal Fan Coil units ranging from 750 cfm to 4400 cfm.
- (2) 2.5m x 8.5m chillers connected to (3) chiller pumps.
- Wall or duct mounted split air units.
- 11KV line from main power feeds 11kv RMU's in (2) substations.
- (5) 100KVA, 11KVA/433V transformers connected to (15) distribution boards.

Structural

- Reinforced concrete building
- Two way flat plate concrete floor for the second and third floors, typical 140 to 200mm slab for the rest of the floors.
- Concrete columns range from 200mm x 400mm to 700mm x 1200mm.
- Perimeter & interior shear wall lateral system with a 1.2m deep mat foundation.

Samir Al-Azri
Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2009/ssa5024/index.html

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Executive Summary

The G.Muttrah Commercial & Residential Complex is an 8 story multi use building located in the city of Muscat in the Sultanate of Oman. Located on the coast, the 280,000 square foot reinforced concrete structure consists of two-way flat plate system on the first two floors and a typical two-way slab system on the rest of the building. The lateral system consists of 10 shear walls that are located in the core of the building. Considered a safe seismic zone, the sultanate of Oman also has low average wind speeds compared to the United States which results in relatively few shear walls for such a building.

As a senior thesis design project, changes were made to the structural system of the G.Muttrah complex. The building was relocated to the Houston, Texas, for a more dynamic design of the lateral system which included greater seismic and wind loads. Results from the design process indicated that 8 more shear walls, placed around the core of the building, were needed to sustain the new increased wind load.

In addition to the new loads due to the relocation of the building, the floor system was also changed. The flat plate on the first two floors and the two way slabs on beam on the rest of the floors were replaced with a two way post-tensioned flat plate system for the entire building. This new system decreases the thickness in the office floor from 14in to 8in. It also eliminated the beams in the residential floor while using fewer columns that spanned larger distances.

Furthermore, breadth topics were addressed as part of the thesis design. The first breadth topic is a study of the change in the construction schedule and cost of the new structural system where the analysis revealed that the new system saved about $90,000 per floor and 9 weeks per floor in construction time. The second breadth topic is a study of the architecture since more shear walls are added, some of the interior spaces are redesigned to accommodate the new lateral system.

Acknowledgments

The author would like to extend acknowledgments to the following individuals and firms:

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  - Prof. Robert Holland
  - Dr. Richard Behr
  - The entire AE faculty and staff

A special thank you to all my friends, fellow classmates and family back at home for all the encouragement, love and support. Finally, I would like to thank God for this opportunity and making this experience possible.

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. 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.

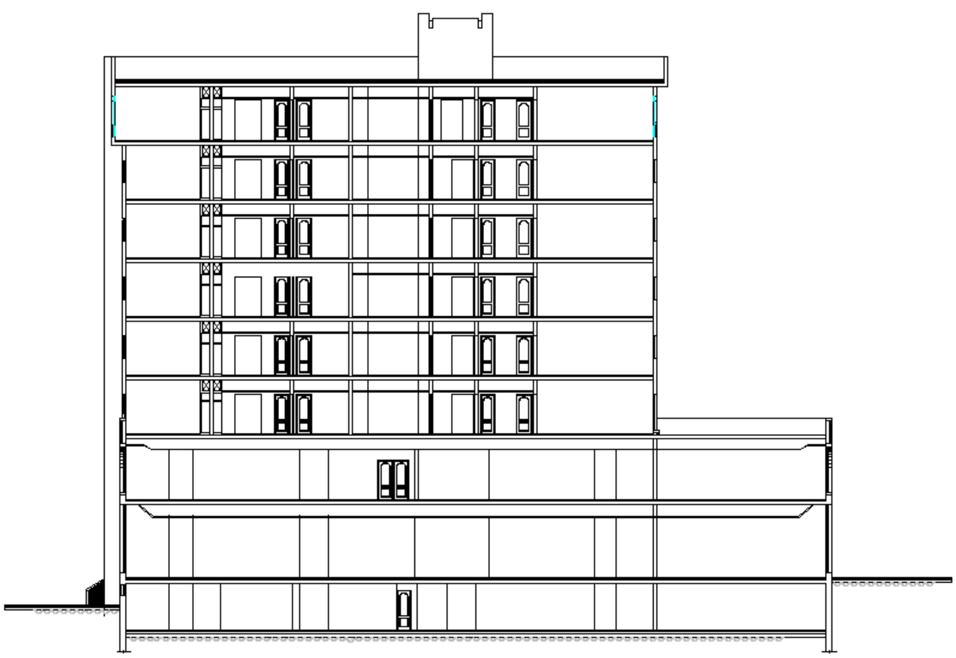


Figure 1: A section of the building showing levels and setbacks

Site and General Architecture

The site of the G. Muttrah Residential & Commercial Complex is located at the MBD East, greater Muttrah in the city of Muscat, Sultanate of Oman. Covering an area of about 28,500 square foot, the site mostly consists of silty sand soil without any vegetation. Adhering to the codes of the Municipality of Muscat, the building is only allowed 8 stories with a building maximum height of 100 ft. The car parking is also restricted by the site boundary which explains the car parking being located in the basement. Figure 3 shows the sites location relative to neighboring plots.

The majority of the façade consists of Omani marble wall cladding that is mechanically fixed and painted with sand mortar and colored grout to match different parts of the building. The marble in the corners of the building is painted to match the windows to create an appearance of a full glazed wall. Reinforced Glass concrete is also used as a façade in two strips running down the building which can be seen in the front elevation.

The roofing of the building consists of the typical concrete slab followed by 70mm of average inclination screed, 50mm thick heat insulation, 4mm thick water proofing, 20mm thick mortar and topped with 30mm thick cement tile. Notice that a thick heat insulator is provided due to the fact that the climate in Oman is very dry and temperature averages over 110 degrees during the summer.

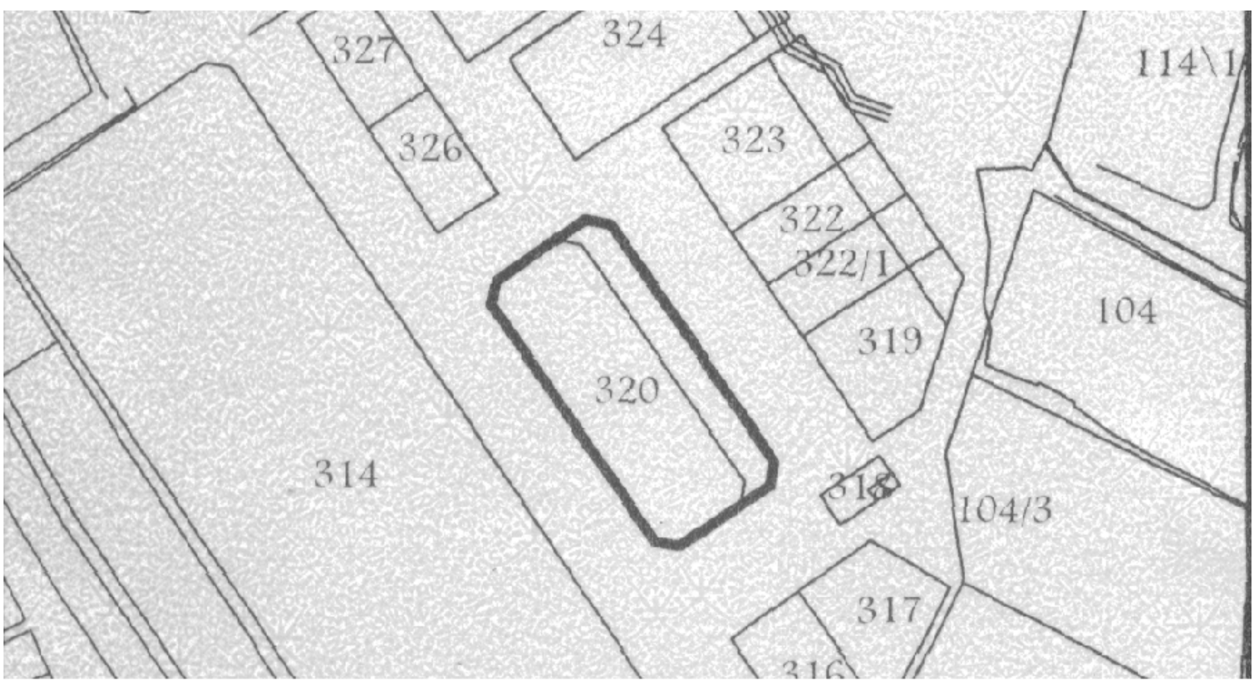


Figure 3: Plot No. 320 at MBD East, Greater Muttrah, Muscat

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 30ft. The material strength used is approximately 5,000 psi strength concrete and 60,000 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 a slab thickness of 8in. The slabs are supported by the usual rectangular beams that range from 6in x 20in to 32in x 20in.

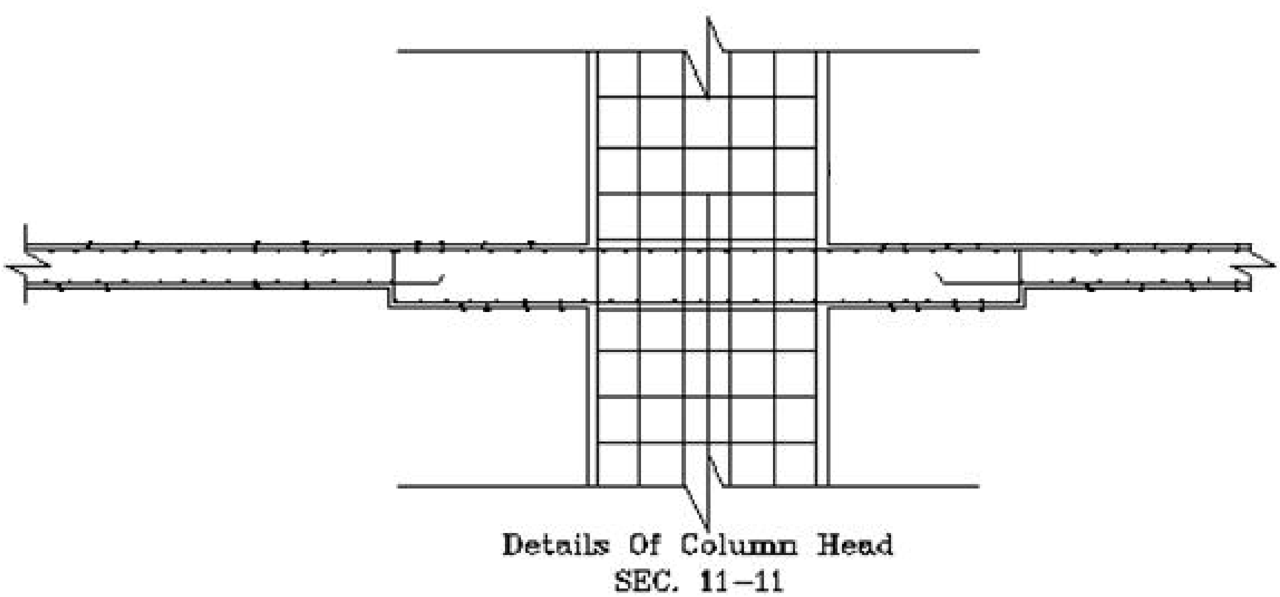


Figure 3: Flat plate slab and column on the second floor

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

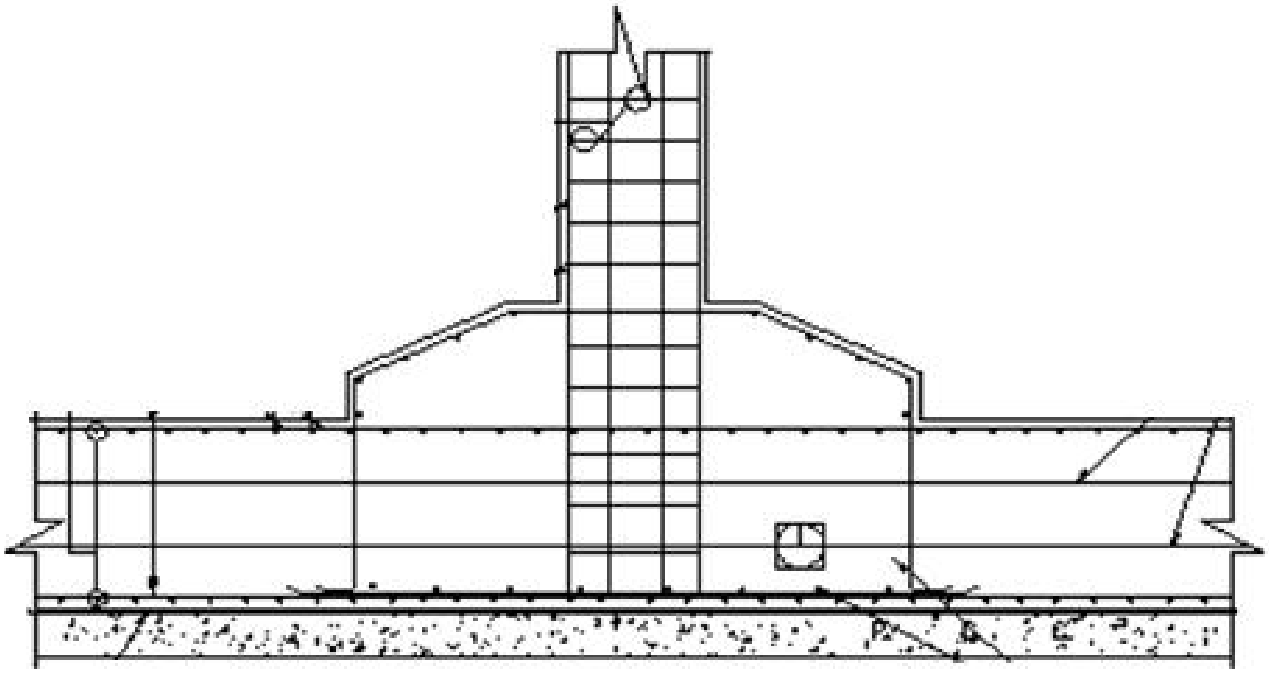


Figure 4: Typical column base at foundation level

Lateral System

Shear walls are used to resist the lateral force in the G.Muttrah complex. The shear walls are located in the core of the building and of 8in all the way to the roof. These walls run in the North-South direction which is expected since that is the weaker axis due to the wind direction and exposure to a larger surface area. There is only one shear which runs in the East-West direction.

Proposal

As a senior thesis design project, changes will be made to the structural system of the G.Muttrah complex. The building would be relocated to the United States for a more dynamic design of the lateral system which would include greater seismic and wind loads. Since the building is originally located in a unique environment, a city that most resembles Muscat had to be chosen in order to reduce the changes in the initial design condition while adding greater wind and seismic loads. The city chosen for the senior design thesis is Houston Texas.

In addition to the new loads due to the relocation of the building, the floor system will also be changed. The flat plate on the first two floors and the two way slabs on beam on the rest of the floors will be replaced with a two way post-tensioned flat plate system for the entire building. The new wind and seismic loads would change the lateral system, possibly increasing the number of shear walls while the new floor system would also affect the overall weight of the building. The new design would be conducted using US codes and standards.

Furthermore, breadth topics will be addressed as part of the thesis design. The first breadth topic would be a study of the change in the construction schedule and cost of the new structural system. The second breadth topic would be a study on the architecture of the building since more shear walls will possibly be added and also the lower weight of the building might require less or smaller columns.

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 in Original Design:

Cast-in-place Concrete

- Foundations: 5700 psi
- Formed Slabs: 5000 psi
- Columns & Walls: 5500 psi
- Reinforcement: 60000psi

Material Strength in New Design:

Cast-in-place Concrete

- Foundations: 5000 psi
- Formed Slabs: 5000 psi
- Columns & Walls: 5000 psi
- Reinforcement: 60000psi

Design Loads

Below is a list of the loads from ASCE 7-05 which will be used in this design of the new gravity and lateral system:

Live Loads:

Table-1

Occupancy	Load (psf)
Parking	40
Entry	100
Office	50
Retail	100
Residential	40
Corridor	100
Roof	20
Ramps (vehicle)	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

Load Combinations:

The load combinations examined for the new design are:

- 1.4D
- 1.2D+1.6L
- 1.2D+1.6L+0.8W
- 0.9D+1.6W
- 0.9D+1.0E

Loads and Load Cases

The loads used for the new design are adjusted to fit the design in Houston, Texas. The wind speed in Houston averages at around 120 mph compared to 75mph in the sultanate of Oman. In addition, there is a minimum design category A seismic requirements for the building in Houston according to ASCE 7-05. More calculations and details for lateral loads can be found in Appendix A. The following is a summary of the loads used for the design:

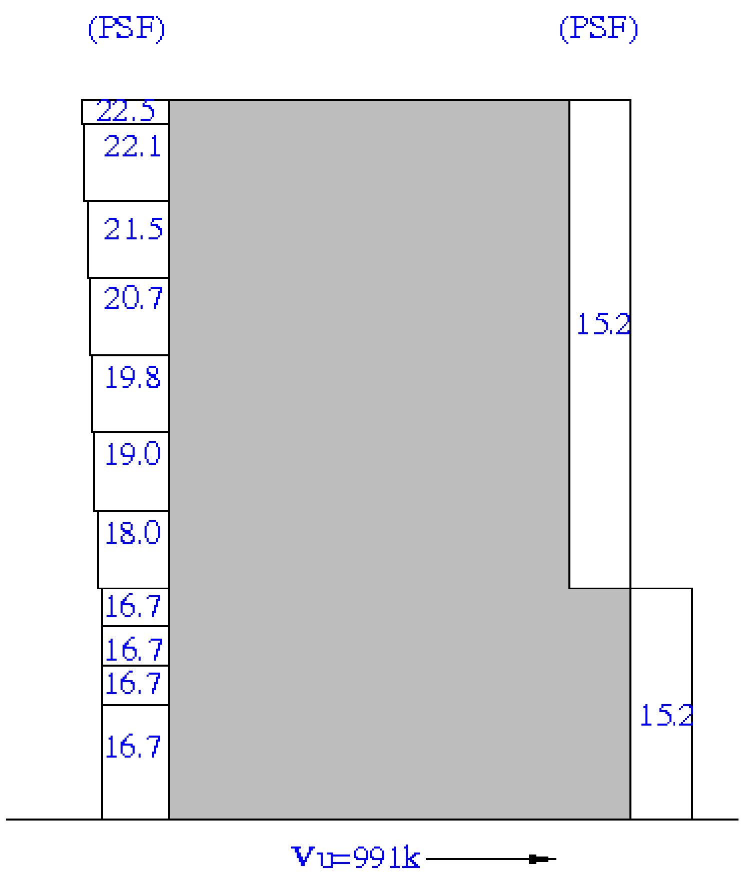


Figure 5: Wind Loads on North-South Face

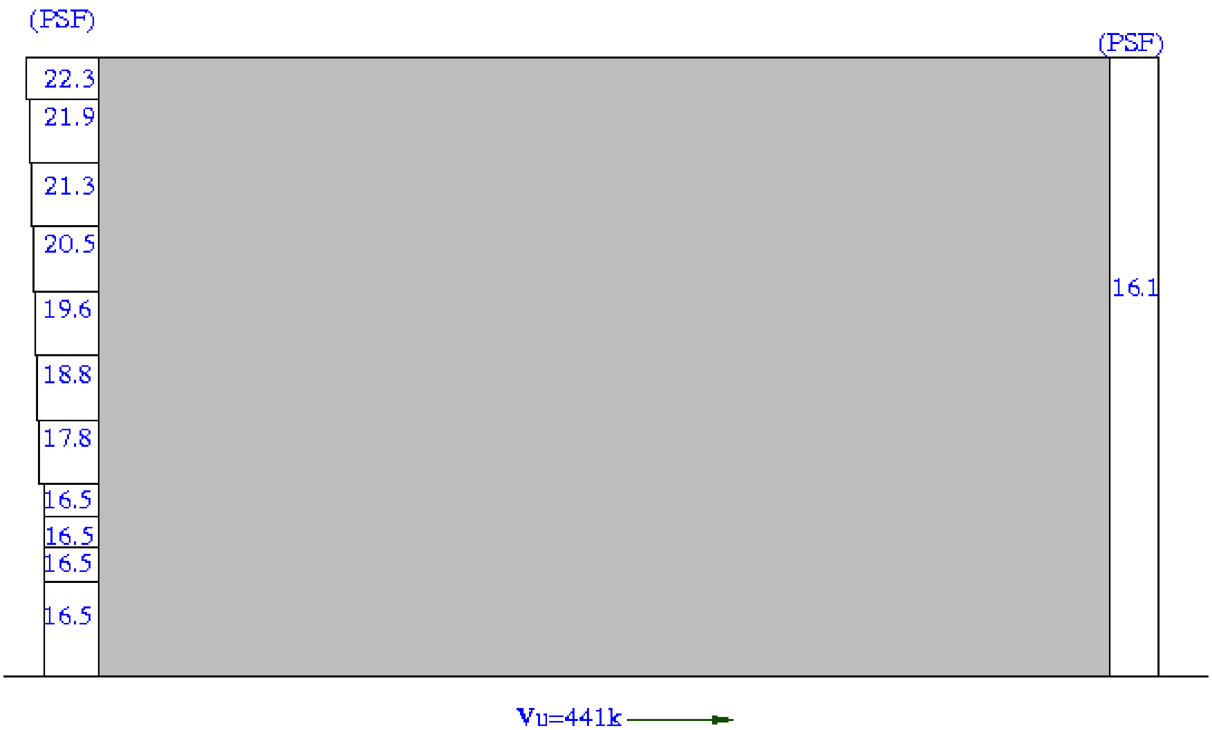


Figure 6: Wind Loads on East-West Face

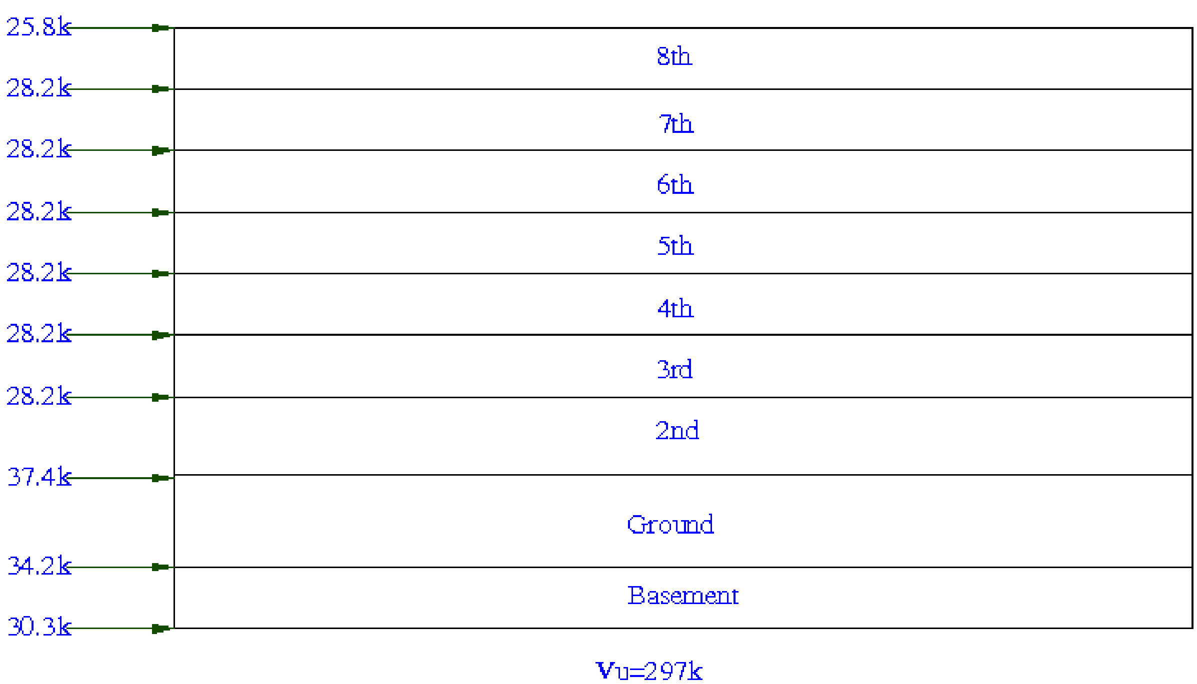


Figure 7: Seismic Loads on East-West face

Since the dominant load on the building is caused from the wind, the load combinations examined are from the ASCE 7-05 which resulted in a controlling load combination of **1.2 D + 1.6 L + 0.8W**. This load combination also satisfies many of the assumptions made for analysis such as low seismic force and high hurricane winds expected in Houston.

Different wind load cases from ASCE 7-05 were also studied. The following figure shows how the different cases were applied to the building for analysis:

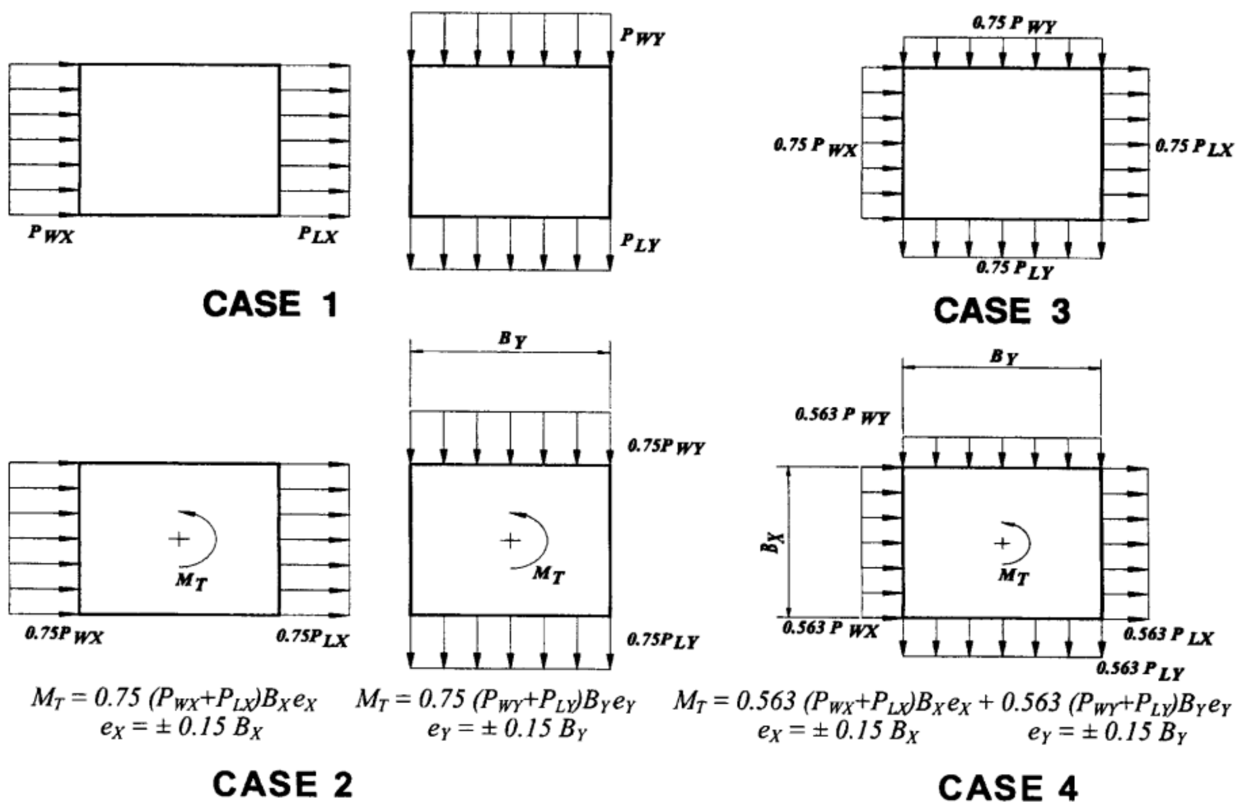


Figure 8: Wind Load Cases

Gravity System (Depth Topic I)

Post Tensioned Slab

A new floor system was designed in an attempt to create a more consistent flooring system throughout the entire building. This new design consists of a two way post-tensioned flat plat slab with no drop panels. The post-tension would help reduce the number of columns by allowing the slab to span larger distances. It would also decrease the thickness of the slab which would in turn increase the floor to ceiling height. The flat plat system is ideal for the residential building since it would eliminate the beams and provide a finished ceiling.

The slabs were designed using ADAPT-PT which uses the equivalent moment frame method. Hand calculations were also used to check the results obtained from ADAPT. The floor plan was divided into strips running in both the E-W and N-S direction. The following plans shows the strips generated with the typical strip designed highlighted:

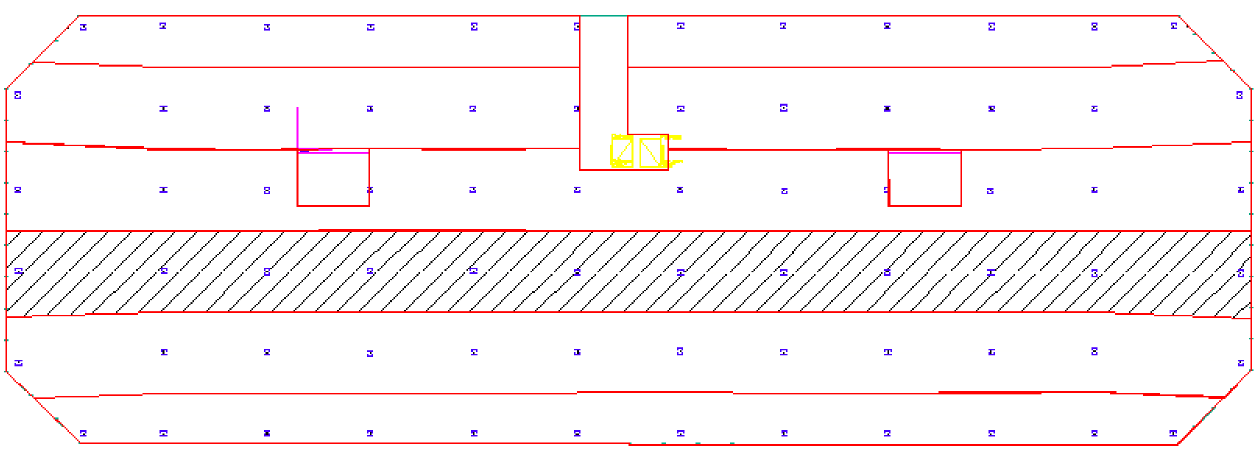


Figure 9: Post-Tension design Strips in E-W Direction
(Residential floor)

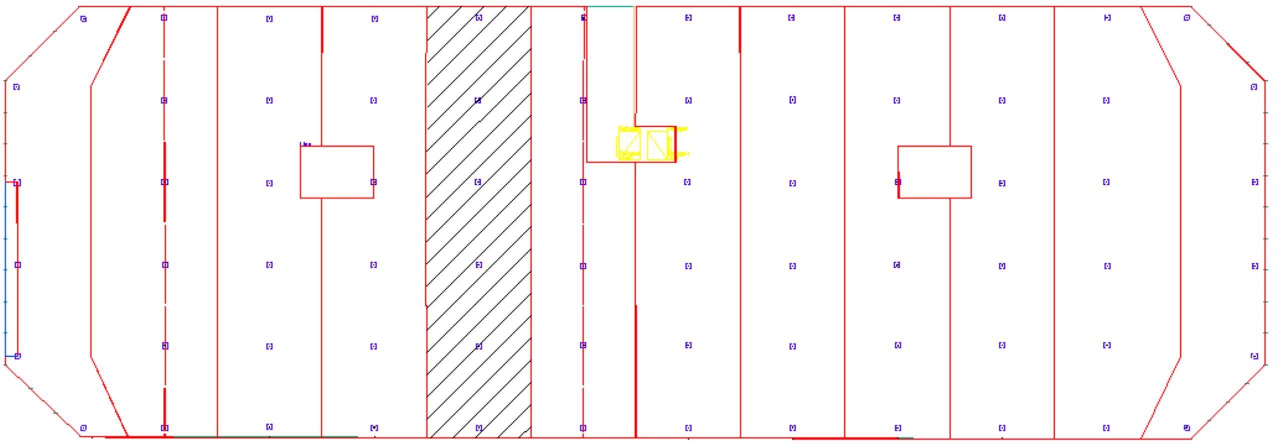


Figure 10: Post-Tension design Strips in N-S Direction
(Residential floor)

The following table summarizes the design parameters. Notice that the concrete strength used is kept at 5000 psi, in order to compare it with the original design (also designed at 5000 psi). The balanced dead load percentage was kept at fewer than 100% while the average precompression was bounded by a maximum value of 350 psi. The strand used is a 270ksi, 7-wire prestressing steel strand. Pattern loading was not considered since the LL/DL < ¾.

Parameter	Value	Parameter	Value
Concrete		Minimum Cover at BOTTOM	0.75 in
F'c for BEAMS/SLABS	5000.00 psi	Post-tensioning	
For COLUMNS/WALLS	5000.00 psi	SYSTEM	UNBONDED
Ec for BEAMS/SLABS	4031.00 ksi	Fpu	270.00 ksi
For COLUMNS/WALLS	4031.00 ksi	Fse	175.00 ksi
CREEP factor	2.00	Strand area	0.153 in 2
CONCRETE WEIGHT	NORMAL	Min CGS from TOP	1.00 in
UNIT WEIGHT	150.00 pcf	Min CGS from BOT for interior spans	1.00 in
Tension stress limits / (f'c)1/2		Min CGS from BOT for exterior spans	1.75 in
At Top	6.000	Min average precompression	125.00 psi
At Bottom	6.000	Max spacing / slab depth	8.00
Compression stress limits / f'c		Analysis and design options	
At all locations	0.450	Structural system - Equiv Frame	TWO-WAY
Reinforcement		Moments reduced to face of support	YES
Fy (Main bars)	60.00 ksi	Moment Redistribution	NO
Fy (Shear reinforcement)	60.00 ksi	DESIGN CODE SELECTED	ACI-318 (2005)
Minimum Cover at TOP	0.75 in		

Table 1:Post-Tension Design Parameters

E-W Direction strip:

Due to the shape of the building, there are two 30 ft exterior spans at each end of the strip while the rest of the spans are about 20ft. More detail regarding the column layout will be covered in the column design section of the report. The two long exterior spans resulted in an increase in stress compared to the interior spans. After several trials it was discovered that an 8in slab with 22 strands works for the flexure stresses and deflection. Punching shear was also checked when designing the columns. The following graphs illustrate the tendon profile and deflections produced:

Figure 11

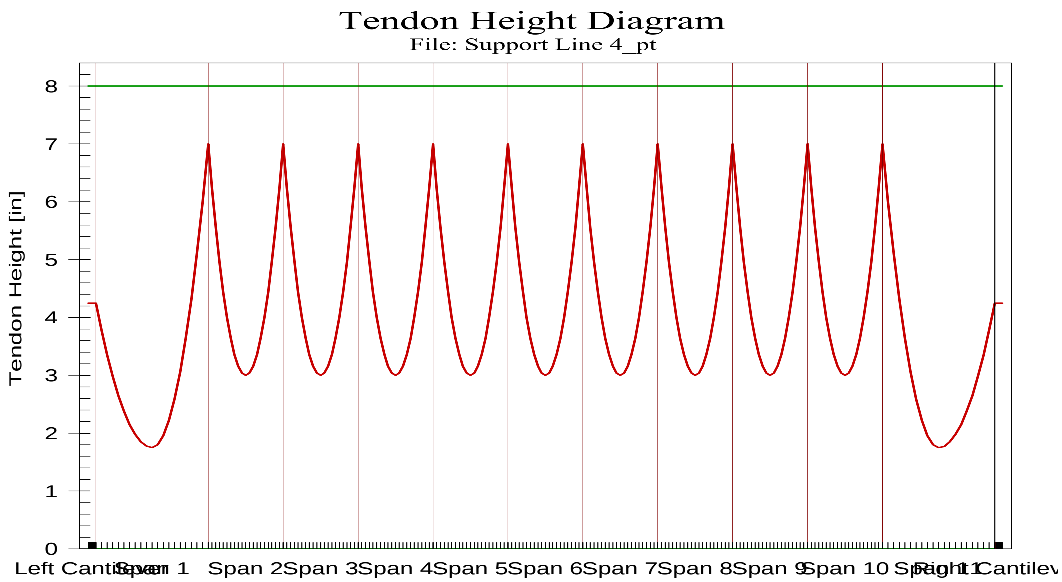
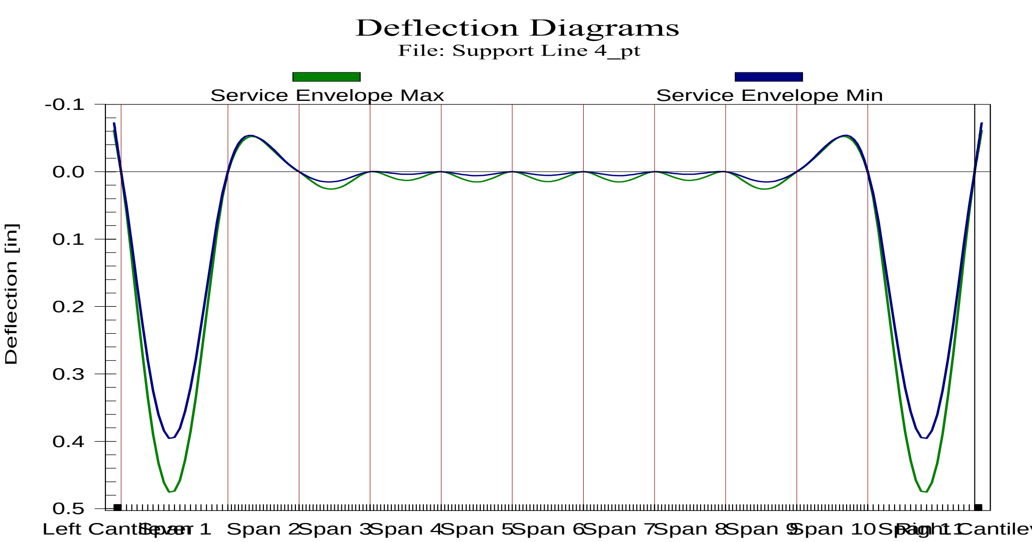


Figure 12



The deflection limit used in the design is L/360. This is due to the assumption that no deflection is induced by the dead load since it is mostly balanced by the tendons. Hence, the deflection would generally be caused by the live load which is limited at L/360. Notice that the largest deflection is under 0.5in which is acceptable based on our assumptions.

Hand calculations were used to check the results obtained from ADAPT-PT. The results from ADAPT-PT yielded larger forces and thus the design from ADAPT-PT is more conservative. The difference in results is due to the fact that the hand calculation is simplified and based on many assumptions. Below is the summary of the hand calculation. More details and calculations are provided in Appendix A:

a_{INT}	4		
a_{end}	3.875		
W_b(k/ft)	1.44		
P(k)	501.6774194	< 574.5 From Adapt	hence Adapt conservative
No. of tendons	18.8445		
P_{actual}(k)	505.818		
W_b(k/ft)	1.42821229		
P/A(psi)	329.3085938		

Table 2:Post-Tension parameters

Stage 1: Stresses after jacking

	Interior span	End span	Support stresses	
f_{top}(psi)	-223.8398438	121.8632813	-798.0585938	ok
f_{bot}(psi)	-434.7773438	-780.4804688	139.4414063	ok

Table 3:Post-Tension stresses after jacking

Stage 2: Stresses at service load

	Interior span	End span	Support stresses	
f_{top}(psi)	-305.8710938	-171.1054688	-475.7929688	ok
f_{bot}(psi)	-352.7460938	-487.5117188	-182.8242188	ok

Table 3:Post-Tension stresses at service load

N-S Direction Strip:

Since the N-S direction is the short span direction, the resulting stresses were much smaller as expected. The spans are uniform causing the tendon profile to be uniform as well. Furthermore, a resulting deflection of less than 0.03in was compared to the L/360 and checked out as acceptable. Only 6 strands were needed for the short spans. The following graph illustrates the tendon profile and deflection in the N-S direction:

Figure 13

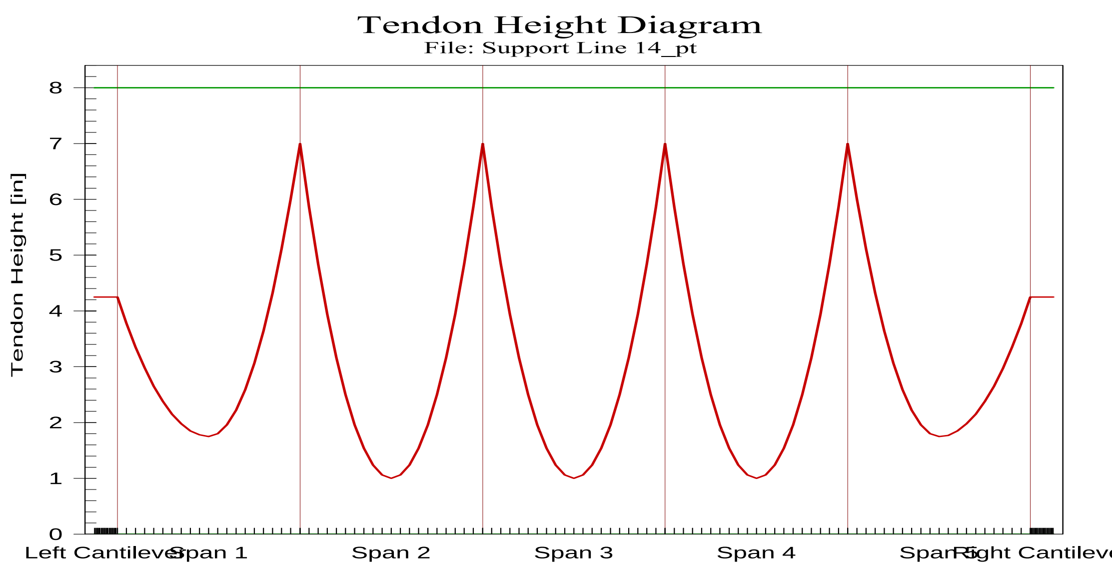
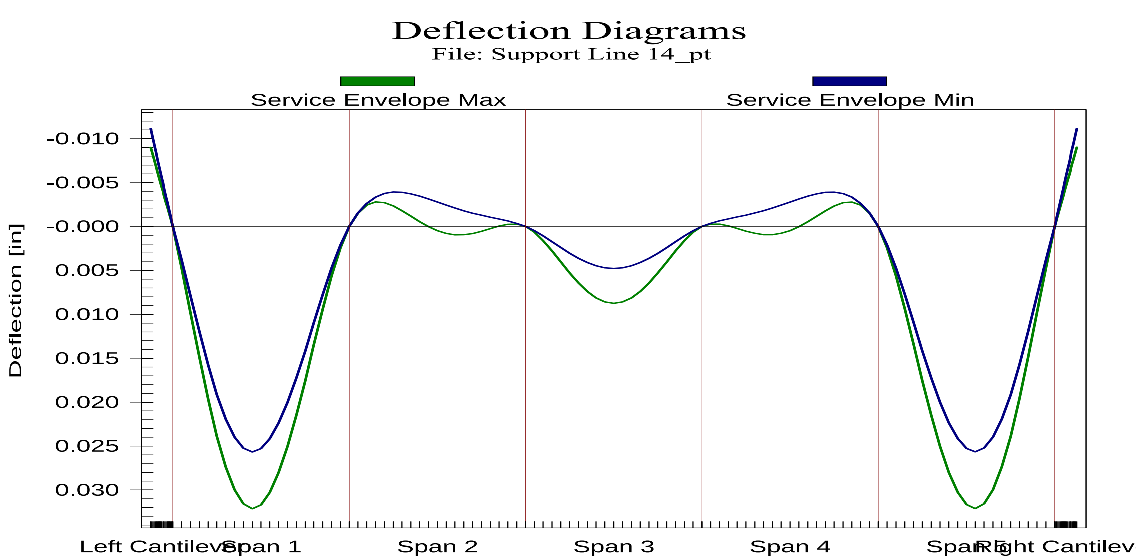


Figure 14



Slab Design Summary

The new post-tensioned flat plat design did not decrease the thickness of the slab as expected. On the other hand, it still served as a better flooring system since the beams were eliminated and fewer columns were used. The slab thickness, however, did decrease on the office floor from 14in flat plate with drop panels to 9in flat plate with no drop panels with the post tensioning. The elimination of the beams will decrease the weight of the building while significantly impacting the cost and schedule of the building which will be discussed later on in the report.

Keep in mind that the design discussed is a typical strip in the floor plan. Further study would be needed to determine the exact design of the other strips, especially the ones with an opening which is not included in the scope of this report. The following diagram summarizes the design of a typical interior bay in a residential floor:

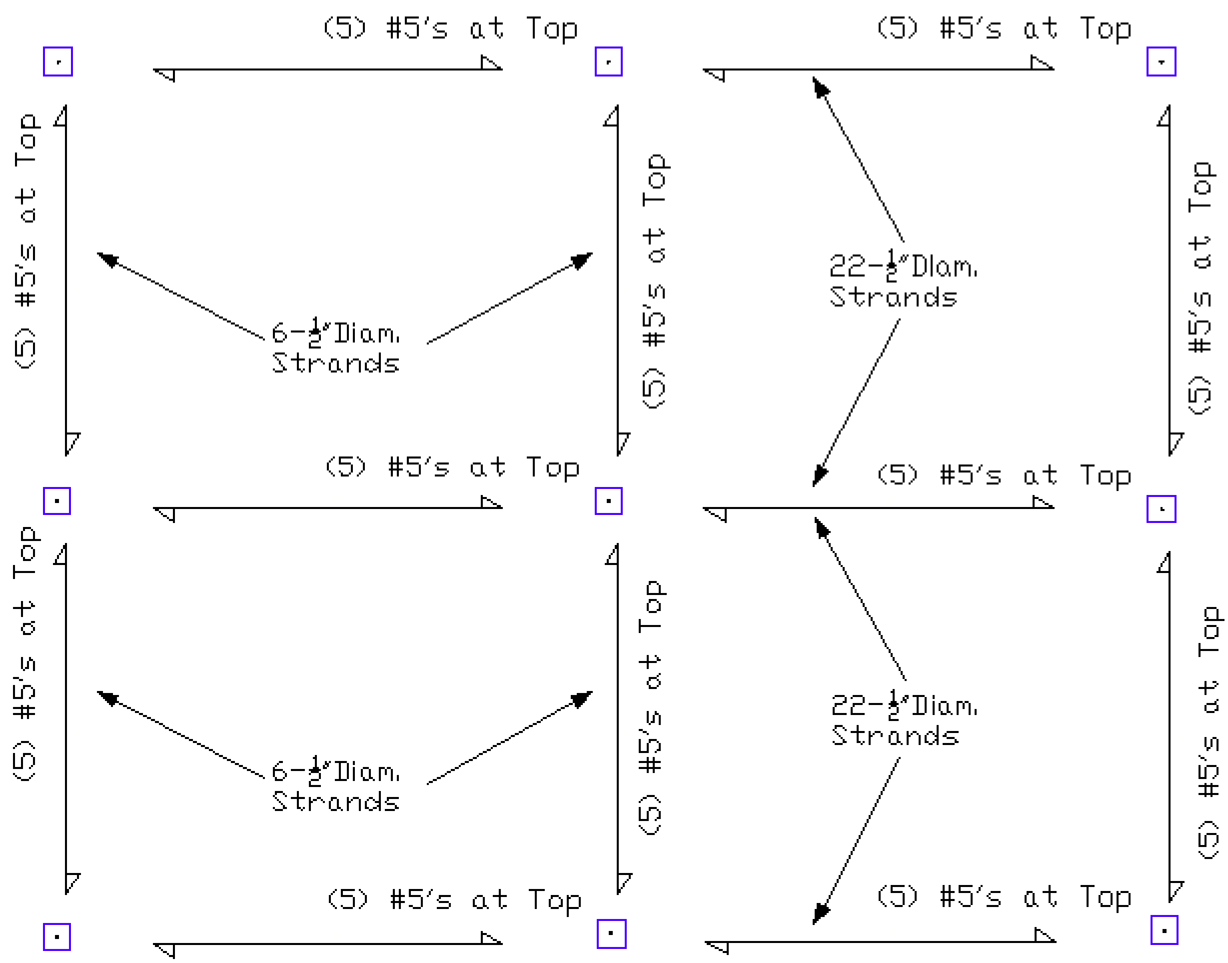


Figure 15: Design of a Typical Interior bay on the residential floor

Column Design:

The column grid for the existing building was designed to complement the two-way beams running in the residential area. The spans ranged between 10ft and 30ft where the columns at the exterior span were considerably too close to each other for a post-tensioned design. Many different column sizes were also used within the same floor which would increases the time of erecting the form work during the construction phase. The building had 14 different column sizes within a single floor. Figure 16 shows the existing column layout:

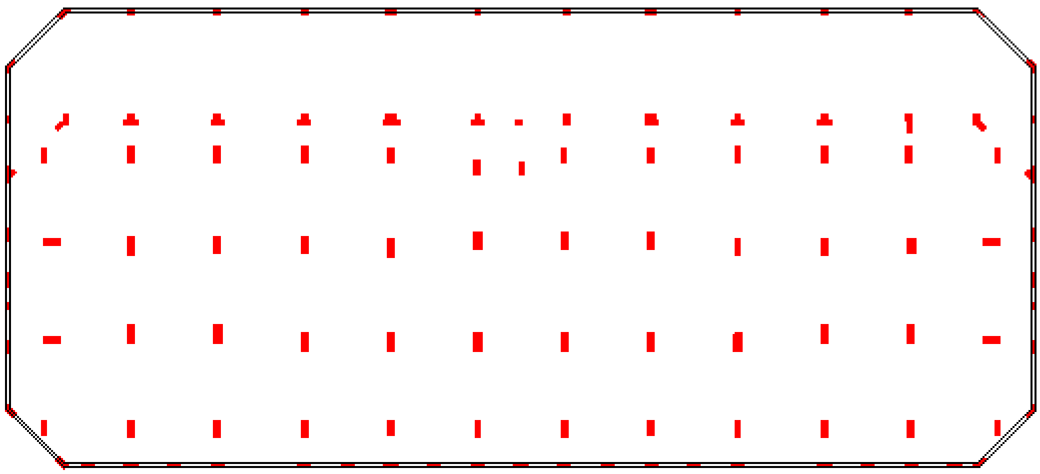


Figure 16: Column Layout of Original Design

A new Column Layout had to be designed in order to increase the spans to justify using a post-tensioned slab. A slab thickness of less than 8in could have been achieved if the same layout was used. However, an engineering decision was made to increase the spans and decrease the number of columns at the expense of the thinner slab. The reasoning behind this decision is that the form work for the slab is very basic and would not take more time to construct if you increase the slab thickness. The column form work on the other hand would cost more money and time to construct if more columns were designed. A more flexible space layout is also achieved when using fewer columns especially in the office spaces. Figure 17 shows the column layout for the new design:

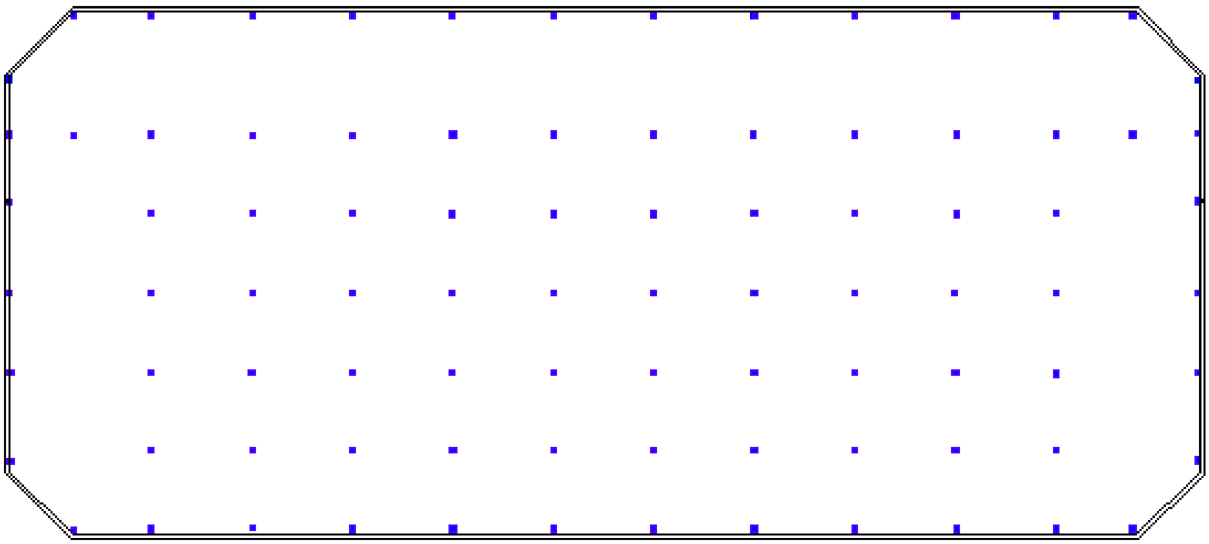


Figure 17: Column Layout of New Design

The new column layout was designed with larger more uniform spans in order to efficiently incorporate the post tensioning slab into the building. There are still larger spans at the exterior bays and that's due to the shape of the building which forces to use on of the following; two smaller spans, one large span, or the same span with a larger cantilever. The number of different column sizes was also reduced to two sizes only; exterior columns and interior columns sizes. Total number of columns was reduced from 112 to 88 columns.

A column takedown of the loads was generated by hand in order to design the columns. PCA column was then used to design the individual columns using the interaction diagrams. The size of the building column going up the different levels was also kept constant in order to better facilitate the construction process. Hence all columns were designed by the loads applied at the bottom level. The moments generated on the columns were minimal since the frame of the building was assumed as a non-sway frame and only the shear walls are used to resist the lateral loads. However, these minimal moments were still checked with the design. The following table summarizes the loads and sizes of the columns. Refer to Appendix A for more detailed calculations:

Column Type	Axial Load (k)	Size (in)
Interior Column	1052	20 x 20
Exterior Column (All floors)	1193	24 x 24
Exterior Column (North of 1st and 2nd Floor)	491	14 x 14

Table 4: Column Design Summary

Punching Shear:

The design of flat plate slabs was checked for punching shear to make sure the columns and slab were adequate to carry the loads. As expected, the exterior spans did not satisfy the shear check and hence a solution had to be determined. Possible solutions to the punching shear problem are; using drop panels, increasing column size, and using stud rails. Assuming the architect would not be very happy with the idea of drop panels in the residential floors, the column sizes where increased. In addition, to minimize the increase in column size shear studs were used. The software used to design the shear studs is called STDesign 3.1 provided by Decon. The following figure illustrates the design of the shear studs:

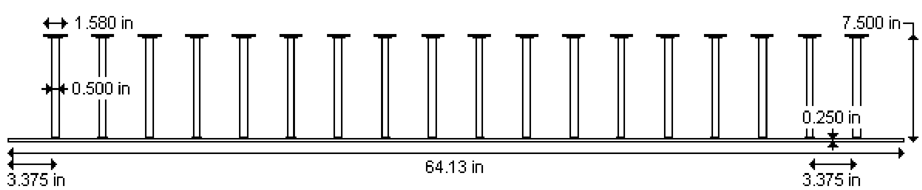
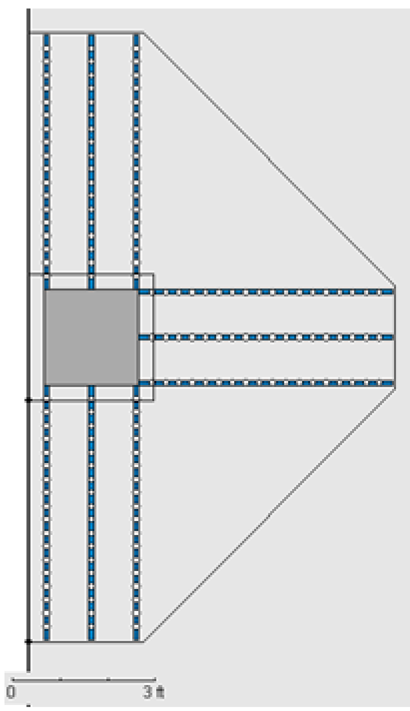


Figure 18: Elevation of Studrails



There are 9 studrails per column with 18 studs on each studrail. The studs are spaced at 3.38 in with an overall height of 7.5in.

Figure 19: Plan of Studrails

External Beams:

When designing the post tensioned slabs, the façade of the building was not taking into consideration. Hence a quick hand calculated estimate was used to design the external beams to carry the façade. These beams are not designed to carry any loads from the slab. Table 5 summarizes the beam design:

Façade Load (psf)	30	
W_u (PLF)	300	
M_u (K-FT)	20.19798	
Try b= 4/5d		
d^3	7.963109	
use h=11, d=8.5		
b	7	
d	8.5	
h	11	
bd^2	505.75	
W_{sw} (PLF)	80.20833	
W_u (PLF)	456.25	
M_u (K-FT)	23.27103	
$20M_u$	465.4206	< 506 o.k
A_s	0.684442	
A_s	0.21	
	0.2	
a	1.61	
c	2.02	
M_n (K-FT)	30.78	
ϕM_n (K-FT)	27.7	o.k

Table 5: External beam frame

Hence a 7in x11in beam with (4) # 4's is sufficient to carry the load of the façade.

Lateral System (Depth Topic II)

As mentioned earlier in the report, the lateral force resisting system in the original design consists of shear walls within the core of the building. Most of the shear walls are spread out about the East-West direction running along the North-South axis. The following plan highlights the location of the shear walls in the building.

Figure 20: Location of shear walls in the building
(shear walls highlighted in red)

Notice that the shear walls are located near the center of mass in the first two floors while being shifted away as you approach the residential floors. This relocation of center of rigidity causes a torsional moment on the building as discussed in previous technical reports. A new layout of shear design would have to reduce the distance between the center of rigidity and center of pressure from the loads.

As a result of relocating the building to Houston, Texas, the wind average speed increased from 75 mph to 120 mph. This change in wind speed doubled the story forces on the building. Refer to the loads section for a wind diagram and Appendix A for more calculations on the wind Loads.

The new system was designed using ETABS with the aid of ETABS, a three-dimensional structural building design and analysis software. In order to simplify the design and get a better understanding, the lateral system was designed independent of the remainder of the building. Only the shear walls and diaphragms were included in the model for analysis.

The new layout of shear walls shown below was designed to relocate the center of rigidity closer to the center of mass. All diaphragms where modeled as rigid members in order to make sure that all forces are transferred to the shear walls correctly while ensuring that only the shear walls are resisting the lateral loads.

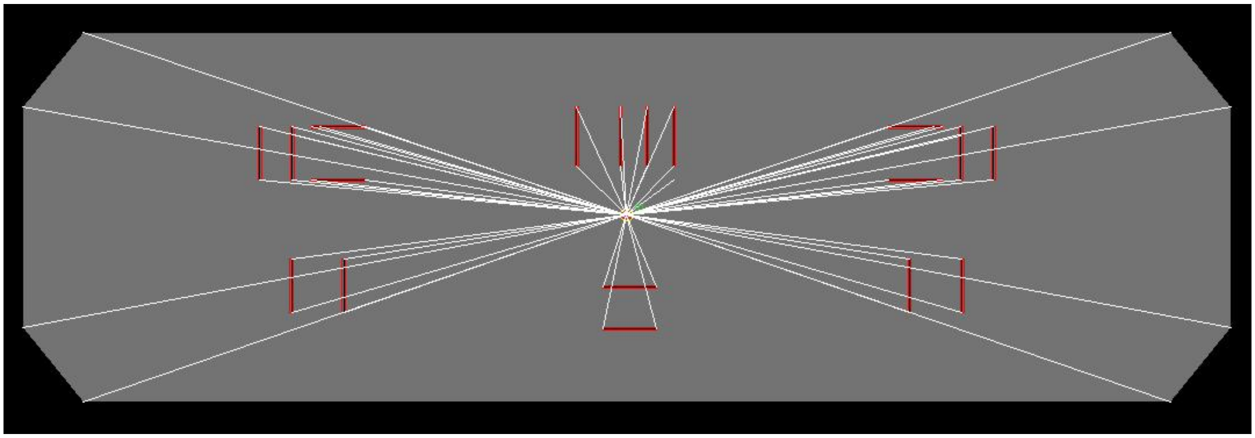


Figure 21: New shear wall layout

The new wall design includes (18) 140in shear walls with a thickness of 8in. Reinforcement consists of # 8's in the vertical direction and #4's in the horizontal direction. The walls checked out as adequate in both flexure strength and shear. Figure 21 shows the ETABS 3-D model with the new shear walls:

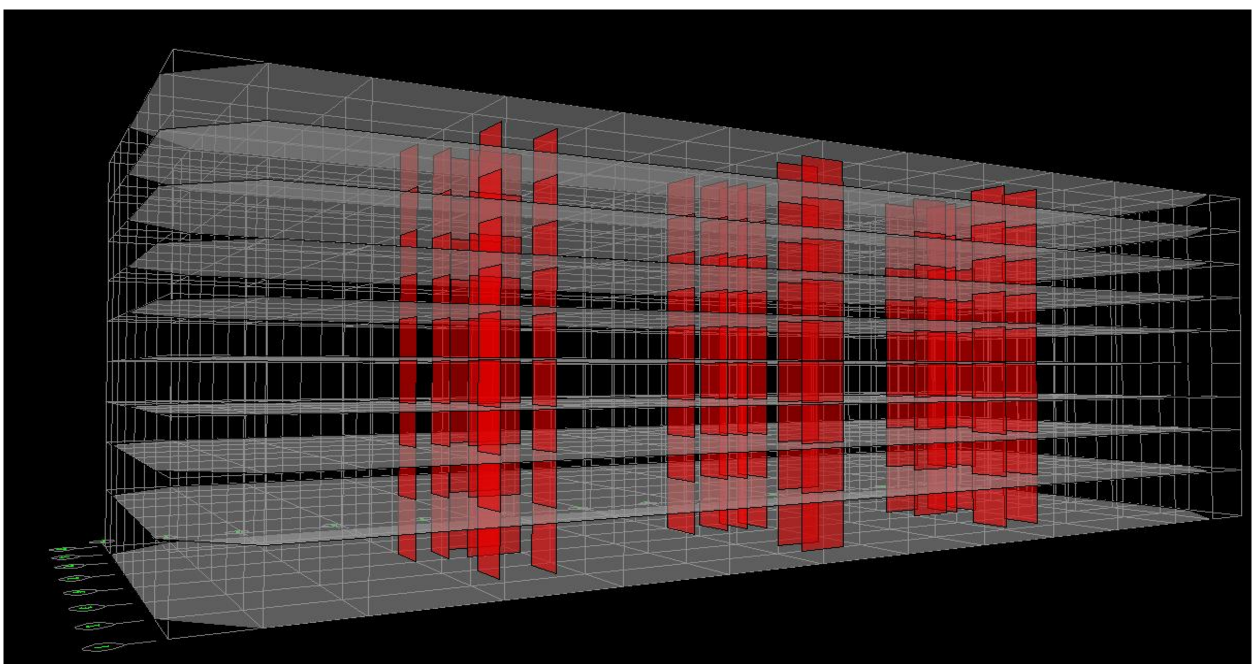


Figure 22: 3-D model from ETABS

The results obtained from ETABS were compared with hand calculations in order to check the designs capability. ETABS generated smaller rebar sizes for the vertical reinforcement hence the final design included the larger bar sizes from the hand calculations to ensure the design is conservative. Calculations are as follows:

Check Reinf.		
ρ_t	0.004167	>0.0025 o.k
Max Spacing	18	
ρ_l	0.004167	
h_w/l_w	8.22	> 3
Reinf. Ok		
Check Moment Strength		
$M_{(base)}$	4464	
$M_{u(base)}$	7142.4	
N_D	99.7	
N_u	89.73	
ω	0.05	
α	0.016023	
c	11.85032	
d	112	
ϕ	0.9	
A_{st}	4.666667	
T (kips)	256.2994	
M_n (Kip-ft)	1974.199	
ϕM_n (kip-ft)	1776.779	
No good		
Try # 8's for vertical Reinf.		
ρ_l	0.011	
A_{st}	18.43333	
T (kips)	1106	
M_n (Kip-ft)	7946.746	
ϕM_n (kip-ft)	7152.072	
Check Shear		
V_u	129.6	

l/2	5.8	
h/2	48	
Story height	14	
Critical Section	5.8	
$M_{u,critical\ section}$ (Kip-ft)	6390.72	
M_u/V_u	49.31111	
V_c	292.8787	
ϕVc	219.659	

Hence Shear Wall adequate in Flexure and shear

Check if Boundary Elements needed		
P_u (k)	100	
M_u	4464	
A_g	7.733333	
I_g	49152	
f_c (k/in^2)	0.093457	<0.2f$_{c'}$

Hence no boundary element needed

Table 6: Shear wall hand calculations

A hand calculation was used to determine the relative stiffness's of the shear walls. A load of 1000 kips was applied at the top of the wall and the following equation was used to calculate the deflection of the wall:

$$\Delta = \frac{Ph^3}{3EI} + \frac{2.78Ph}{AE} \quad \text{Eqn. 1}$$

The stiffness of the walls was then calculated by taking the reciprocal of the deflection. Keep in mind that this hand calculated method is only an approximation of the real stiffness. The thickness of the wall was assumed to be uniform throughout the entire height of the building for simplification. In addition, the calculated stiffness value was assumed to be the same for each floor. The difference in K values is small between floors and can be ignored. For the purpose of this report, the calculated values are close enough to reality for analysis. The following table summarizes the calculated stiffness factors in the N-S direction:

Table 7: wall stiffness calculations

WALL	FORCE	HEIGHT	WIDTH	THICNKESS	Δ FLEXURE	Δ SHEAR	Δ TOTAL	R_I	RELATIVE STIFFNESS
1	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
2	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
3	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
4	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
5	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
6	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
7	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
8	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
9	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
10	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
11	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
12	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083

Torsion, Deflection and Story Drifts

Due to the balanced layout of the shear walls, the torsional moment on the building decreased significantly compared to the original design. The center of mass and rigidity of the building were calculated using ETABS and the torsional moments were calculated manually. A maximum torsional moment was induced in the first floor since the floor plan is greater than the residential floors. The moment then drops in the residential floors since the new shear walls are designed around its core. A study of the effects of the walls on the architecture will be covered later in the report in a breadth topic analysis. For further details on the torsional moments see Appendix A.

The deflections caused by the different wind loads studied were compared to the L/400 requirement. At the roof level, the maximum wall deflection was 1.178in which passed the L/400 limit which is 2.88in. Story drifts caused by the wind loads were also compared to L/400 which is limited at 0.3in. The table on the right summarizes the story drifts due to wind.

	Disp-x	Drift-x
Roof	1.18	0.159
8	1.02	0.158
7	0.86	0.156
6	0.70	0.152
5	0.55	0.144
4	0.41	0.131
3	0.28	0.114
2	0.16	0.104
G	0.06	0.060

Table 8: Story Drifts caused by Wind Loads

Deflections resulting from seismic loads were compared to the allowable drift of 0.025h.

At 8th floor: $0.02\ h_{sx} = 0.02(10' \times 12) = 2.4 > 0.26$ √ Okay

At 2nd floor: $0.02\ h_{sx} = 0.02(14' \times 12) = 3.36 > 0.16$ √ Okay

	Disp-x	Drift-x
Roof	1.87	0.260
8	1.61	0.260
7	1.35	0.260
6	1.09	0.250
5	0.84	0.220
4	0.62	0.210
3	0.41	0.170
2	0.24	0.160
G	0.08	0.080

Table 9: Story Drifts caused by Seismic Loads

Impact on Foundation

A soil report from an arbitrary site in Houston was obtained in order to examine if a new foundation design would be needed to withstand the loads from the new structural system. The recommended bearing capacity of a spread/pad footing in the site in Houston is around 5000 psi. However, the allowable bearing capacity for the same footing in the site in Muscat is 5221psi.

Since the weight of the building was significantly reduced by removing the beams and using fewer columns, it is safe to assume that existing foundation would withstand the loads from the new system. A more efficient foundation system should not be considered since there is a significant amount of overturning moment from the relatively slender shear walls that would require a mat foundation to resist the moments.

Depth Summary

The post-tensioning slab design did not reduce the thickness of the building, but greater spans were achieved while eliminating the beams. A finished ceiling is also an advantage since it would create a better space aesthetically for the residential floors. Therefore a two-way post-tensioned slab design would be recommended as an alternative flooring system to the G.Muttrah Commercial & Residential Complex.

A new column layout was proposed to complement the new post-tensioned system. Fewer columns were used while also using smaller size since the weight of the building decreased. This new layout would be greatly appreciated in both retail and office spaces.

In order to rebuild the G.Muttrah complex in Houston, Texas, 8 more shear walls would be needed in an arrangement that balances the center of rigidity of the building. The increased wind speed in a hurricane prone area would require these 8 additional shear walls to provide adequate strength and resistance.

Cost and Schedule Analysis (Breadth Topic I)

The cost and schedule of the new design was compared to the original design in order to evaluate the efficiency and cost-effectiveness of the new structural system. In order to do so, the existing design had to be estimated and scheduled as a building that would be built in the United States in order to have a fair and successful comparison.

Instead of performing a rough estimate for the entire building, a typical residential floor was estimated in detail. Since most of the building is residential, a comparison of a typical floor from each design would give a good indication of how the two systems compare.

Cost of Original Design:

Table 10: Cost of Original Design

Original Structure								
Code		Material Details	Quantity	Units	Material Cost	Labor Cost	Equipment Costs	Total Cost
03 11 10.60		**Forms In Place, Beams and Girders**			$0.00		$0.00	
	1500	ior Sprandel, job built plywood 24" wide,	15,932	SFCA	$32,182.64	4.13	$0.00	$7,547.26
03 11 10.60		**Forms In Place, Columns**			$0.00		$0.00	
	7000	36" x 36" columns, 1 use	13,394	SFCA	$24,109.20	4.27	$0.00	$2,016.00
03 11 13.35		**Forms In Place, Elevated Slabs**			$0.00		$0.00	
	7000	Depressed area forms to 12" high, 4 use	751	LF	$503.17	2.84	$0.00	$89,376.00
03 11 00.60		**Forms In Place Walls**			$2,379.78		$0.00	
	2450	2 use	2,034	SFCA	$0.00	3.83	$0.00	$48,658.50
03 21 10.60		**Uncoated Reinforcing Steel**			$840.00		$0.00	
	0100	Beams and Girders, #3 to #7	1.50	Ton	$380.80	630	$0.00	$9,874.80
	0200	Columns, #3 to #7	0.68	Ton	$100.30	670	$0.00	
	0400	Elevated Slabs, # 4 to #7	0.17	Ton	$26.50	350	$0.00	$158,126.56
	0700	Walls, #3 to #7	0.050	Ton	$0.00	335	$0.00	$40,068.00
03 31 00.70		**Concrete**			$83,655.00		$0.00	
	0400	5000 PSI Concrete	1,170	CY	$0.00		$0.00	$21,674.31
03 31 00.90		**Placing Concrete**			$0.00		$1,326.49	$8,620.91
	0200	Beams (Large Beams, Pumped)	163	CY	$0.00	17	$1,213.33	$11,738.34
	1000	36" Square Columns (Pumped)	231	CY	$0.00	10.95	$3,421.26	
	1500	Elevated Slabs 6"-10" (Pumped)	747	CY	$0.00	9.55	$190.39	$26,209.68
	5050	12" Walls (Pumped)	29	CY	$0.00	13.9	$0.00	
03 35 01.40		**Finishing Concrete**			$0.00		$0.00	$6,719.92
	0250	creed, bull float, macine trowel and finish	594	SF	$0.00	0.39	$0.00	
03 35 01.40		**Finishing Walls**			$0.00		$0.00	$10,880.10
	0010	Break Ties and patch voids	74	SF	$3.70	0.4	$0.00	
Subtotal:					$144,181.09	$147,482.09	$6,151.47	$297,814.65
5% Contingency								$14,890.73
Total:								$312,705.38

Cost of New Design:

Table 11: Cost of New Design

New Structure								
Code		Material Details	Quantity	Units	Material Cost	Labor Cost	Equipment Costs	Total Cost
03 11 10.60		**Forms In Place, Beams and Girders**						
	1500	ior Sprandel, job built plywood 24" wide,	1,996	SFCA	$4,031.92	8243.48	$0.00	$12,275.40
03 11 10.60		**Forms In Place, Columns**						$0.00
	7000	36" x 36" columns, 1 use	5,749	SFCA	$10,348.20	24548.2	$0.00	$34,896.43
03 11 13.35		**Forms In Place, Elevated Slabs**						
	7000	Depressed area forms to 12" high, 4 use	751	LF	$503.17	2132.84	$0.00	$2,636.01
03 11 00.60		**Forms In Place Walls**						
	2450	2 use	4,884	SFCA	$5,714.28	18705.7	$0.00	$24,420.00
03 21 10.60		**Uncoated Reinforcing Steel**					$0.00	
	0100	Beams and Girders, #3 to #7	0.39	Ton	$218.40	245.7	$0.00	$464.10
	0200	Columns, #3 to #7	0.38	Ton	$212.80	254.6	$0.00	$467.40
	0400	Elevated Slabs, # 4 to #7	0.17	Ton	$100.30	59.5	$0.00	$159.80
	0700	Walls, #3 to #7	0.090	Ton	$47.70	30.15	$0.00	$77.85
03 41 00.90		**Stressing Tendon**						
	1200	Ungrouted Strand, 50' Span, 100 kips	21K	Lb	$40,389.12	16828.8	$1,262.16	$58,480.08
03 31 00.70		**Concrete**						
	0400	5000 PSI Concrete	903	CY	$64,564.50		$0.00	$64,564.50
03 31 00.90		**Placing Concrete**						
	0200	Beams (Large Beams, Pumped)	16	CY	$0.00	265.54	$127.30	$392.84
	1000	36" Square Columns (Pumped)	93	CY	$0.00	1018.24	$488.20	$1,506.44
	1500	Elevated Slabs 6"-10" (Pumped)	747	CY	$0.00	7133.85	$3,421.26	$10,555.11
	5050	12" Walls (Pumped)	47	CY	$0.00	653.3	$312.55	$965.85
03 35 01.40		**Finishing Concrete**						
	0250	:reed, bull float, macine trowel and finish	594	SF	$0.00	231.66	$0.00	$231.66
03 35 01.40		**Finishing Walls**						
	0010	Break Ties and patch voids	121	SF	$6.05	48.4	$0.00	$54.45
Subtotal:					$126,136.44	$80,400.01	$5,611.47	$212,147.92
5% Contingency								$10,607.40
Total:								$222,755.32

By comparing the two floors, it is evident that building the new structure would save around $90,000 per floor. Such a huge saving is achieved by eliminating the interior beams in the residential floors and also reducing the number of columns. Hence using the new structural system would be more cost-effective.

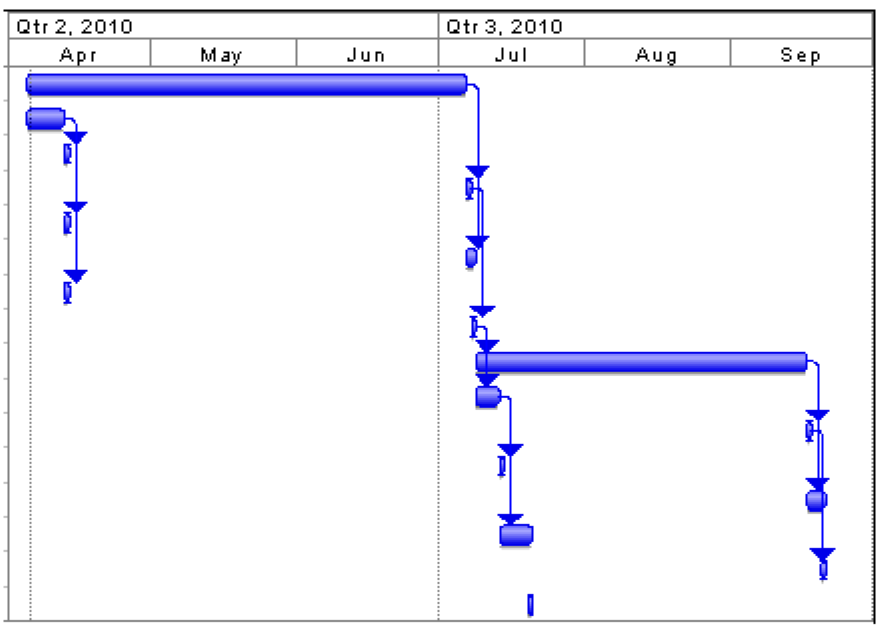
A similar comparison was made to examine the two schedules. Only a schedule for building a single residential floor was examined. Similar construction processes were used to avoid changing any variables. Only the structural systems will be compared.

Schedule of Existing Design:

Table 12: Schedule of Existing Design

ID		Task Name	Duration	Start	Finish	Predecessors
1		Form Columns	67 days	Mon 4/5/10	Tue 7/6/10	
2		Form Wall	6 days	Mon 4/5/10	Mon 4/12/10	
3		Place Wall Rebars	1 day	Tue 4/13/10	Tue 4/13/10	2
4		Place Column Rebars	1 day	Wed 7/7/10	Wed 7/7/10	1
5		Place Concrete for walls	1 day	Tue 4/13/10	Tue 4/13/10	2
6		Place Concrete for Columns	2 days	Wed 7/7/10	Thu 7/8/10	1
7		Finish Wall	1 day	Tue 4/13/10	Tue 4/13/10	2
8		Finish Columns	1 day	Thu 7/8/10	Thu 7/8/10	4
9		Form Beams	50 days	Fri 7/9/10	Thu 9/16/10	8
10		Form Slab	3 days	Fri 7/9/10	Tue 7/13/10	8
11		Place Beam Rebar	1 day	Fri 9/17/10	Fri 9/17/10	9
12		Place Slab Rebar	1 day	Wed 7/14/10	Wed 7/14/10	10
13		Place Beam Concrete	2 days	Fri 9/17/10	Mon 9/20/10	9
14		Place Slab Concrete	5 days	Wed 7/14/10	Tue 7/20/10	10
15		Finish Beam	1 day	Mon 9/20/10	Mon 9/20/10	11
16		Finish Slab	1 day	Tue 7/20/10	Tue 7/20/10	

Figure 23: Timeline of Existing Design

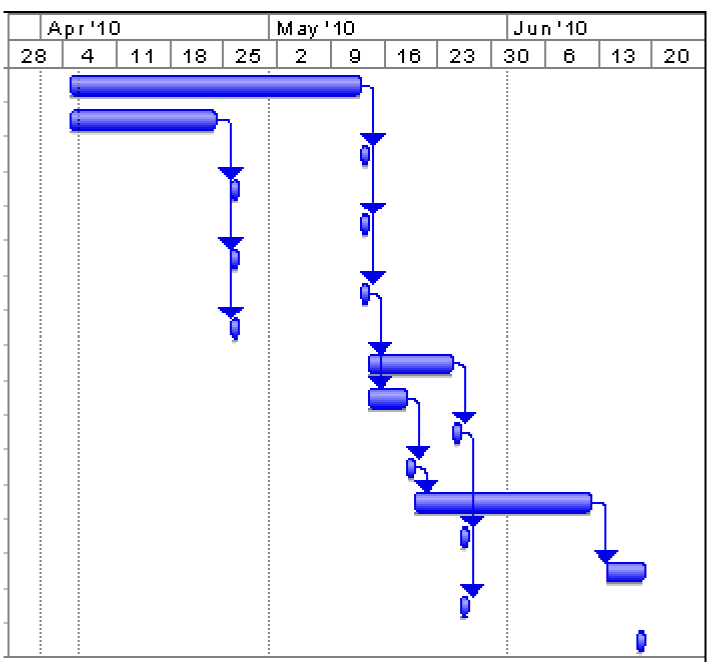


Schedule of New Design:

Table 13: Schedule of New Design

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
1		Form Column	28 days	Mon 4/5/10	Wed 5/12/10		
2		Form Wall	15 days	Mon 4/5/10	Fri 4/23/10		
3		Rebar Column	1 day	Thu 5/13/10	Thu 5/13/10	1	
4		Rebar Wall	1 day	Mon 4/26/10	Mon 4/26/10	2	
5		Place Column Concrete	1 day	Thu 5/13/10	Thu 5/13/10	1	
6		Place Wall Concrete	1 day	Mon 4/26/10	Mon 4/26/10	2	
7		Finish Column	1 day	Thu 5/13/10	Thu 5/13/10	1	
8		Finish Wall	1 day	Mon 4/26/10	Mon 4/26/10	2	
9		Form Beam	7 days	Fri 5/14/10	Mon 5/24/10	7	
10		Form Slab	3 days	Fri 5/14/10	Tue 5/18/10	7	
11		Rebar Beam	1 day	Tue 5/25/10	Tue 5/25/10	9	
12		Rebar Slab	1 day	Wed 5/19/10	Wed 5/19/10	10	
13		Place Tendons	17 days	Thu 5/20/10	Fri 6/11/10	12	
14		Place Concrete Beam	1 day	Wed 5/26/10	Wed 5/26/10	11	
15		Place Concrete Slab	5 days	Mon 6/14/10	Fri 6/18/10	13	
16		Finish Beam	1 day	Wed 5/26/10	Wed 5/26/10	11	
17		Finish Slab	1 day?	Fri 6/18/10	Fri 6/18/10		

Figure 23: Timeline of New Design



The schedule of the existing design states that the construction process would take about 17 weeks. On the other hand, the schedule for the new design indicates that it will only take about 8 weeks to construct the new structural system.

Analyzing the two schedules, we realize that the huge number of different columns and sizes in the existing design combined with the beams would cause a great increase in the schedule. This is due to the fact that the form work would take a longer time to construct. Remember that the original design had 112 columns with 14 different sizes on each floor compared to 88 columns and only 2 sizes for the new design.

In summary, using the new structural design would save about $90,000 per floor while also saving about 9 weeks per floor in construction time. Therefore, the new structural system would be a more efficient alternative in terms of savings in construction cost and construction time.

Table 14: Summary of cost and schedule analysis

	Existing Design	New Design
Construction Cost	$312,705	$222,755
Construction Time	17 weeks	8 weeks

Architectural Analysis (Breadth Topic II)

As mentioned earlier in the lateral system design, 8 more shear walls were added to the G.Muttrah complex in order to provide enough strength to resist the high wind loads residing in Houston, Texas. The old shear walls were located around the elevator core and stairwells. In order to minimize the effect of the architecture, 12 of the new shear walls were placed on the same location. However, the remaining 6 shear walls had to be located in areas away from the stairwell and elevators. The new shear walls were positioned on openings that are open to the sky to facilitate ventilation through the building. The following Figure shows the opening in the residential floors:

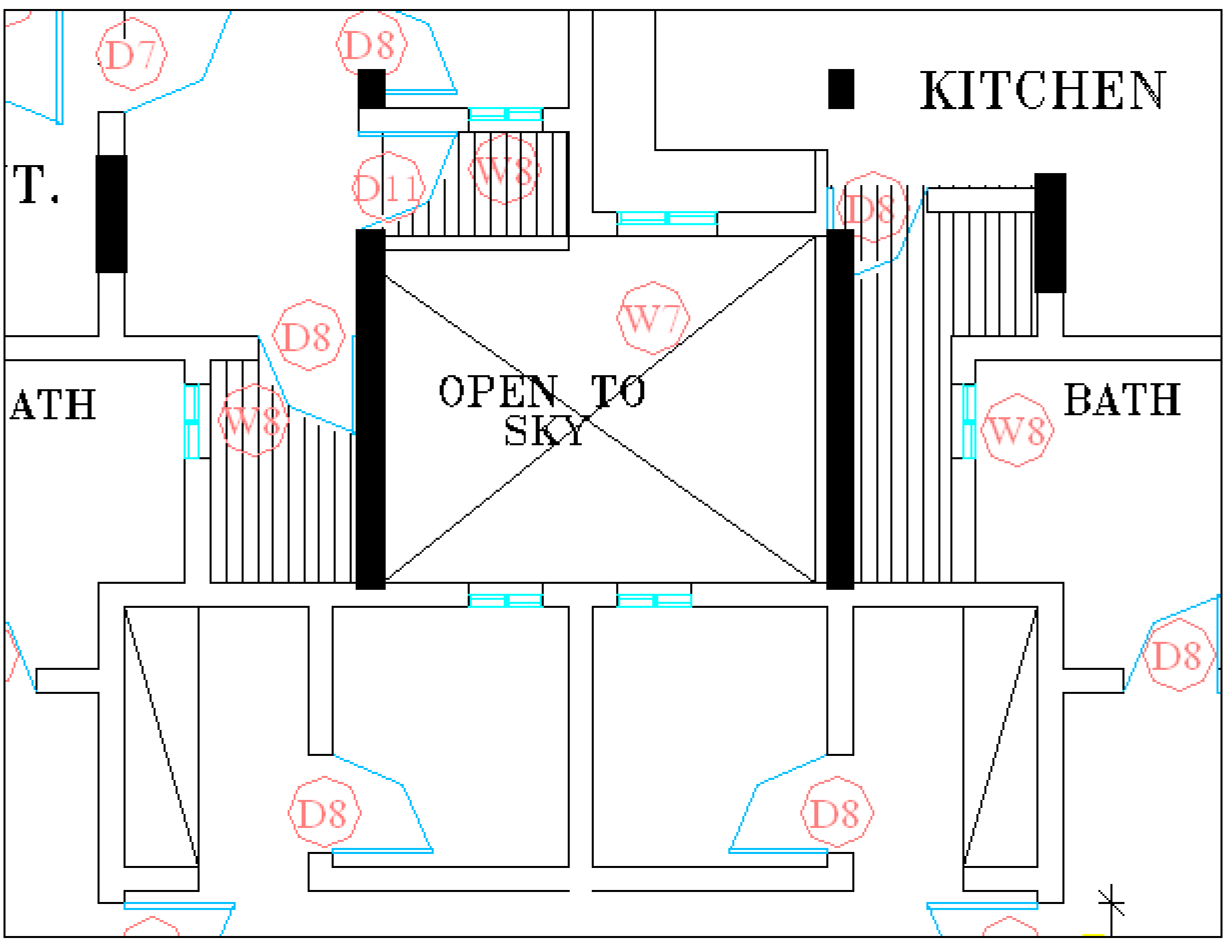


Figure 24: opening surrounded by shear walls

These openings however only start at the residential floors; hence the shear walls would affect the spaces in the retail and office floors. Since the office floor is an open flexible space, the shear walls do not cause any changes to the space layout. The retail space, on the other hand, was affected substantially with shear walls running down the hall ways. The following figure illustrates the new shear walls in the retail floor:

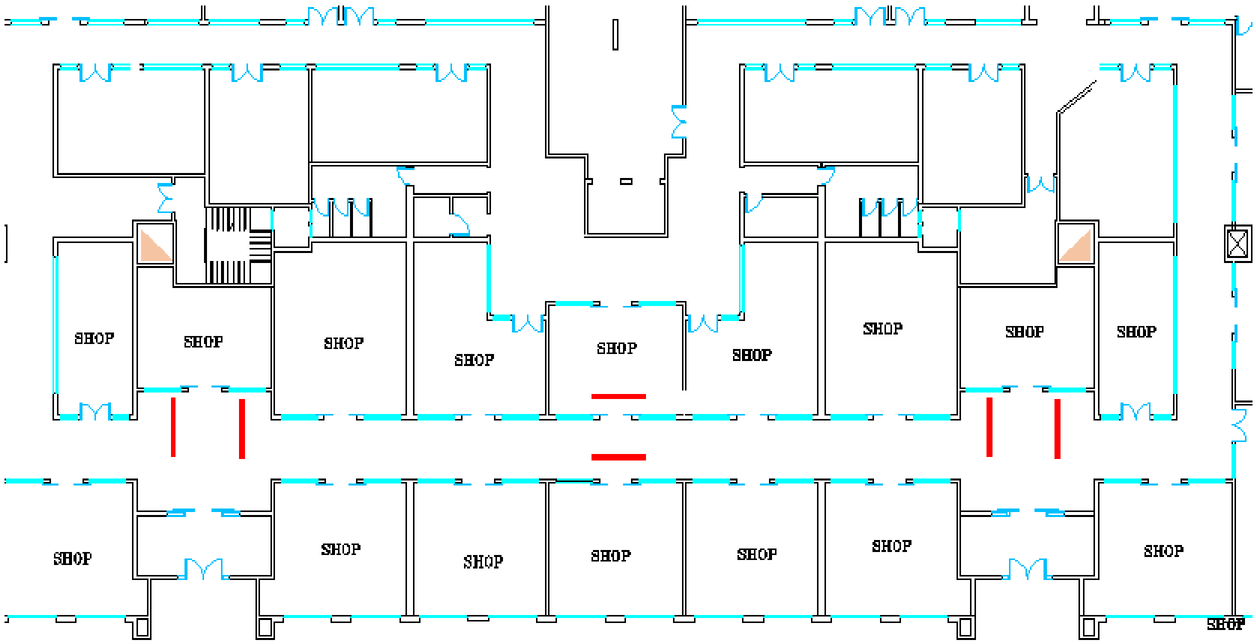


Figure 25: Spaces affected by shear walls

The spaces in the retail floor were redesigned in order to accommodate the new shear walls. Shops were designed around the shear walls and an additional hallway was added that connects the entrances to the stairwell. The columns did not have a major impact on the architecture since there were only about a foot away from the original column layout; hence an architecture study on the column effect was not needed.

Below is a plan of the new floor plan with spaces rearranged to accommodate the shear walls:

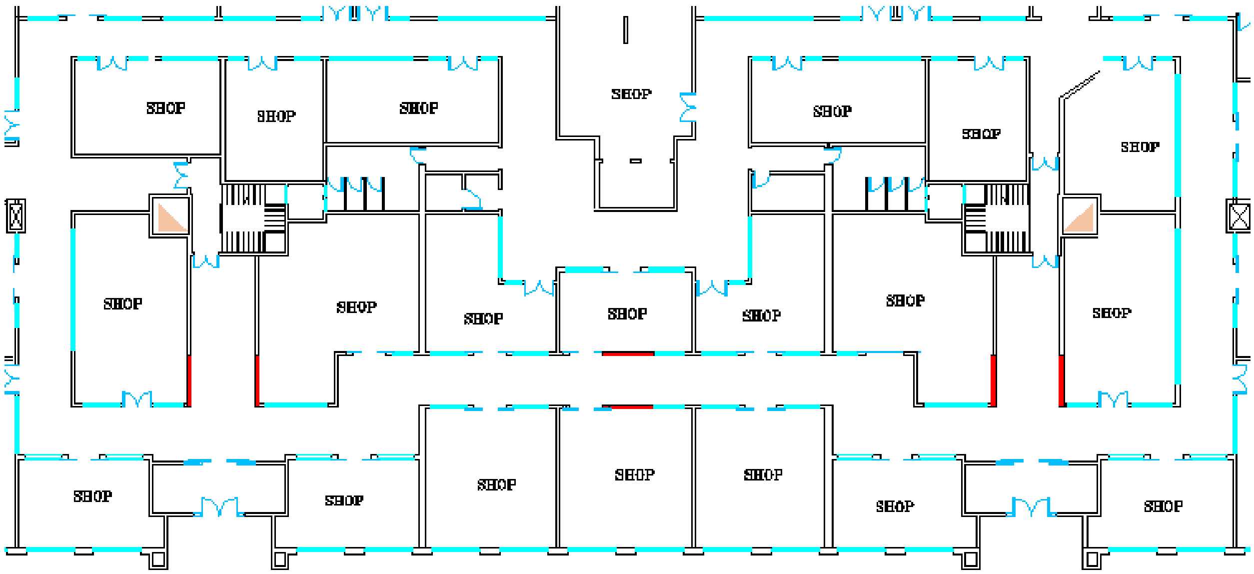


Figure 26: Spaces after redesign

In conclusion, it is understood that the new shear walls can be added into the G.Muttrah complex without having any major impacts on the architecture of the building. The only area being affected is the retail space which can easily be redesigned as discussed.

Final Conclusion

The new structural system of the G.Muttrah commercial & Residential Complex proved be a very efficient and cost effective design. Relocating the building to Houston, Texas, required that the building was constructed with more shear walls to withstand the large wind loads expected to apply on the building.

The two-way flat plate post-tensioning slab system provided a thinner slab for the office floor while eliminate created a more efficient space by removing the beams and increasing floor to ceiling height. A more flexible layout was provided for the office floor by using fewer columns for the entire building compared to the existing design.

The new structural system also proved to be very cost-effective by saving about $90,000 in construction cost per floor and reducing the schedule time by 9 weeks per floor. In conclusion, the two-way post tensioned flat plate is highly recommended as an alternative floor system, and that the building would require a better lateral system with more shear walls in order to be constructed safely in an area such as Houston, Texas.

Appendix A: Calculations

Wind Calculations:

Mean Velocity(mph)	120	
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	91.03372574	
Reduced Frequency N_1	4.146276309	
R_n	0.057112746	
R_h	0.18836731	for η=4.75
R_b	0.141215223	for η=6.54
R_L	0.019918303	for η=49.7
Resonant Response	0.010120572	(N-S)
Resonant Response		
Gust Effect Factor	0.832284941	(N-S)
Gust Effect Factor		

North-South

Height=	96 ft
B=	300
L=	132

Location	Height (Ft)	K_z	q_z	P_z (psf)	P_z (Kips)	Overturning Moment, M_o (ft-kips)
Winward	0-15	0.7	21.93	16.72	75.25	1128.82
	20	0.7	21.93	16.72	25.08	501.70
	25	0.7	21.93	16.72	25.08	627.12
	30	0.7	21.93	16.72	25.08	752.55
	40	0.76	23.81	17.97	53.92	2156.60
	50	0.81	25.38	19.01	57.04	2851.80
	60	0.85	26.63	19.84	59.53	3571.96
	70	0.89	27.89	20.68	62.03	4342.06
	80	0.93	29.14	21.51	64.53	5162.09
	90	0.96	30.08	22.13	66.40	5975.88
	96	0.98	30.71	22.55	40.59	3896.47
Leeward	ALL	0.98	30.71	-15.21	-27.38	-1314.10

East-West

Height=	96 ft
B=	132
L=	300

Location	Height (Ft)	K_z	q_z	P_z (psf)	P_z (Kips)	Overturning Moment, M_o (ft-kips)
Winward	0-15	0.7	21.934	16.548	32.765	491.4713
	20	0.7	21.934	16.548	10.922	218.4317
	25	0.7	21.934	16.548	10.922	273.0396
	30	0.7	21.934	16.548	10.922	327.6476
	40	0.76	23.814	17.781	23.471	938.8462
	50	0.81	25.381	18.809	24.828	1241.3905
	60	0.85	26.634	19.631	25.913	1554.7880
	70	0.89	27.888	20.453	26.998	1889.8920
	80	0.93	29.141	21.276	28.084	2246.7024
	90	0.96	30.081	21.892	28.898	2600.7995
	96	0.98	30.708	22.303	17.664	1695.7690
Leeward	ALL	0.98	30.708	-16.100	-12.751	-612.0576

Story Forces

		Force (k)		Story Shear (K)	
Story	Height	N-S	E-W	N-S	E-W
1	14	134	56.1	991	441.1
2	26	115	51.7	857	385
3	36	98	44.1	742	333.3
4	46	101	45.5	644	289.2
5	56	104	46.7	543	243.7
6	66	106	47.8	439	197
7	76	109	48.9	333	149.2
8	86	111	49.8	224	100.3
Roof	96	113	50.5	113	50.5

Seismic Calculations:

Building Weight

	Weight of slab (k)	Weight of Columns (K)	Weight of Façade (K)	Typical Weight (K)	No. of Bays in Building	Total Weight (K)
Interior Bay(Residential & Retail)	32	4.2	0.0	36.2	292.0	10560.7
Exterior Bay(Residential & Retail)	32	6.0	4.8	42.8	228.0	9758.4
Interior Bay(Office)	36	4.2	0.0	40.2	52.0	2088.7
Exterior Bay(Office)	36	5.0	4.8	45.8	36.0	1650.3
Interior Bay (Roof)	32	3.0	0.0	35.0	40.0	1400.0
Exterior Bay (Roof)	32	2.5	2.4	36.9	32.0	1181.5
Interior Bay (Garage)	32	2.1	0.0	34.1	52.0	1772.3
Exterior Bay (Garage)	32	3.0	0.0	35.0	36.0	1260.0
					Total	29671.8

Story Shear

Story	Typical Ext Bay(K)	Typical Int Bay (K)	Total weight(K)	Shear (K)
B	35.0	34.1	3032.3	30.3
1	42.8	36.2	3421.5	34.2
2	45.8	40.2	3739.0	37.4
3	42.8	36.2	2816.3	28.2
4	42.8	36.2	2816.3	28.2
5	42.8	36.2	2816.3	28.2
6	42.8	36.2	2816.3	28.2
7	42.8	36.2	2816.3	28.2
8	42.8	36.2	2816.3	28.2
R	36.9	35.0	2581.5	25.8
		Total	29671.8	296.7

Site Class: D

Ss: 0.088

S1: 0.036

Design Category: A

Post-Tension Slab Hand Calculations:

Loads	
DL	Self Weight
SDL	15 PSF
Live Load	40 PSF
2 HR Rating	
PT:	
Unbonded	
12" Ψ, 7- wire strand	
A(in^2)	0.153
f_{pu}(ksi)	270
Prestress loss(ksi)	15
f_{se}(ksi)	174
P_{eff}	26.622
h(in)	8

DL(psf)	100
LL/DL < 3/4	ignore pattern loading
A(in_2)	1536
S(in_2)	2048
Allowable Stresses	
at jacking	
compression(psi)	1800
Tension(psi)	164.3167673
At service	
compression(psi)	2250
Tension(psi)	530.3300859

Average Precompression Limits		
P/A (psi)	125	min
	325	max
Load Balance (90%)	90	
Cover(in)	0.75	
Tendon Ordinate	Tendon (CG) Location	
Ext. Sup. Anchor	4.25	
Int. sup. Top	7	
Int.Span. Bot.	3	
End. Span. Bot.	1.75	

a_{INT}	4
a_{end}	3.875
W_b(k/ft)	1.44
P(k)	501.6774194
No. of tendons	18.8445
P_{actual}(k)	505.818
W_b(k/ft)	1.42821229
P/A(psi)	329.3085938

Interior Span		
P(k)	214.2318434	ok
W_b(k/ft)	3.37212	

	Interior span	End span	Support Stresses
M_{DL}(K/ft)	46	162	180
M_{LL}(k/ft)	14	50	55
M_{BAL}(k/ft)	64	239	260
Stage 1: Stresses after jacking			

	Interior span	End span	Support stresses	
f_{top}(psi)	-223.8398438	121.8632813	-798.0585938	ok
f_{bot}(psi)	-434.7773438	-780.4804688	139.4414063	ok

Stage 2: Stresses at service load

	Interior span	End span	Support stresses	
f_{top}(psi)	-305.8710938	-171.1054688	-475.7929688	ok
f_{bot}(psi)	-352.7460938	-487.5117188	-182.8242188	ok

Ultimate Strength		
	Exterior Support	Interior span
e(in)	0.25	3

M_1(ft-k)	126.4545	
M_{sec}(ft-k)	-19.4545	Int support

	Midspan	Support
M_u(ft-k)	13.6	-14.6

Minimum Bonder Reinf.

Positive moment region
All stresses at service load are in compression, no positive reinf. Required
Negative moment region

A_{cf}	1920	
$A_{s(min)}$(in^2)	1.44	int support
8 #4's	1.6	

A_{cf}	1536
$A_{s(min)}$(in^2)	1.152
6 # 4's	1.2

Maximum bar spacing 12"
Top bars 12" from the face of support

A_{ps}	2.907
d(in)	7
f_{ps}(psi)	191700
a(in)	0.800578309

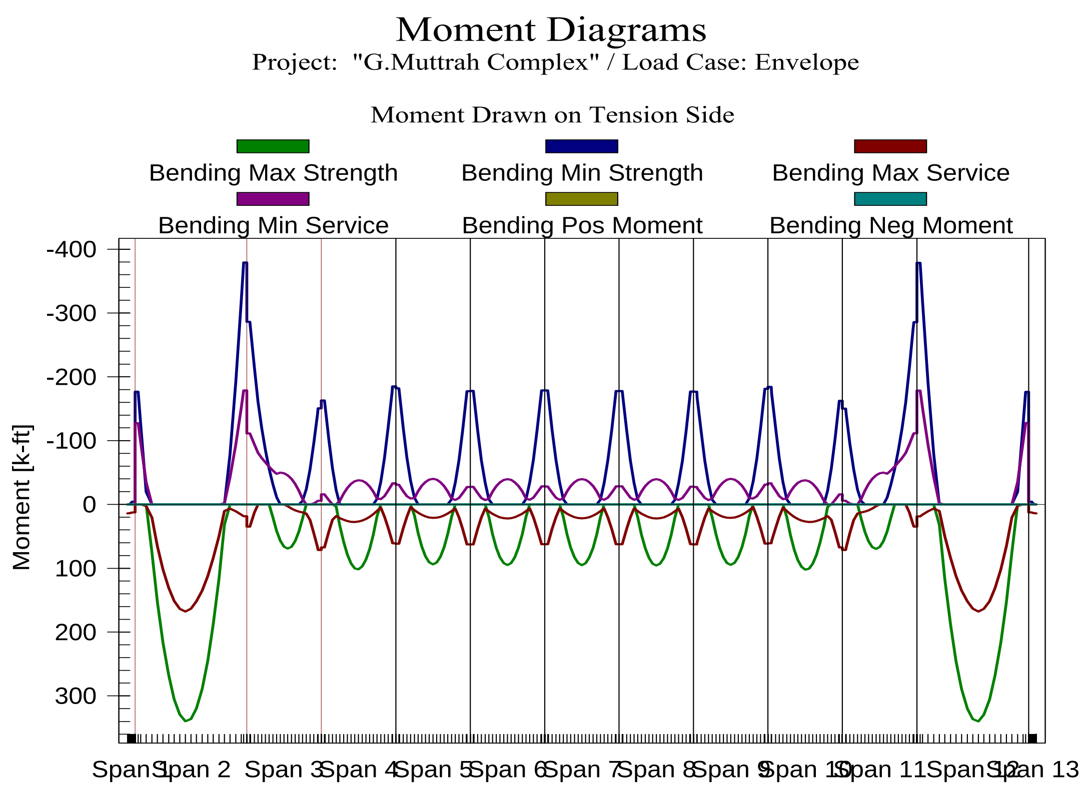
ϕM_n(ft-k)	323.3554233

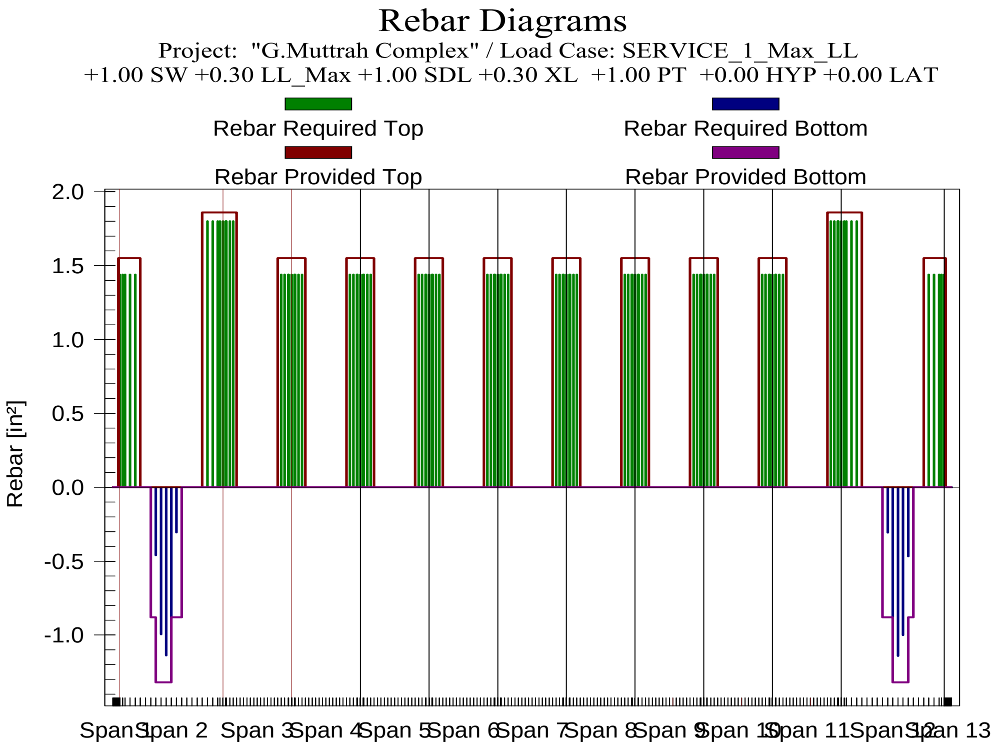
8 # 4's for Top supports

ADAPT-PT Results:

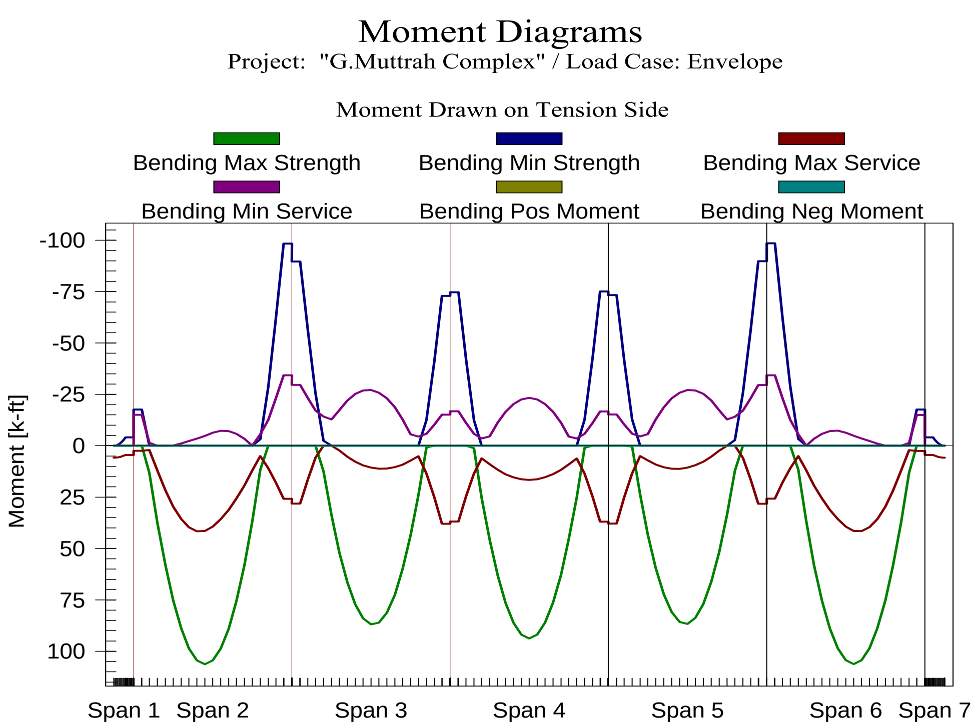
Residential Floor:

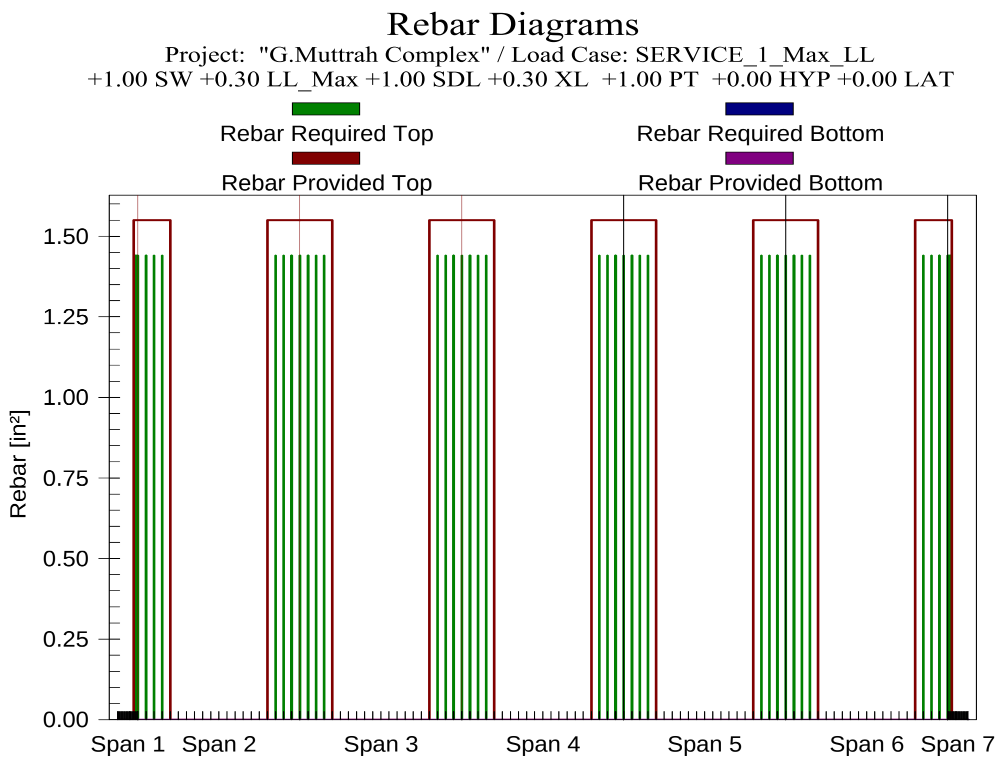
E-W direction:





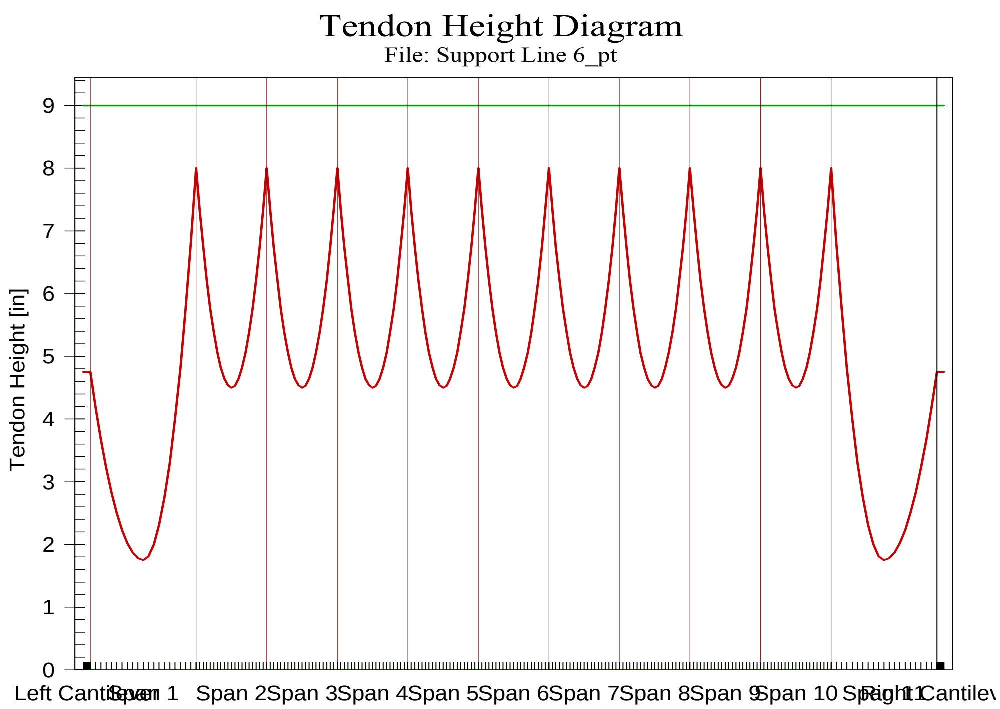
N-S direction:

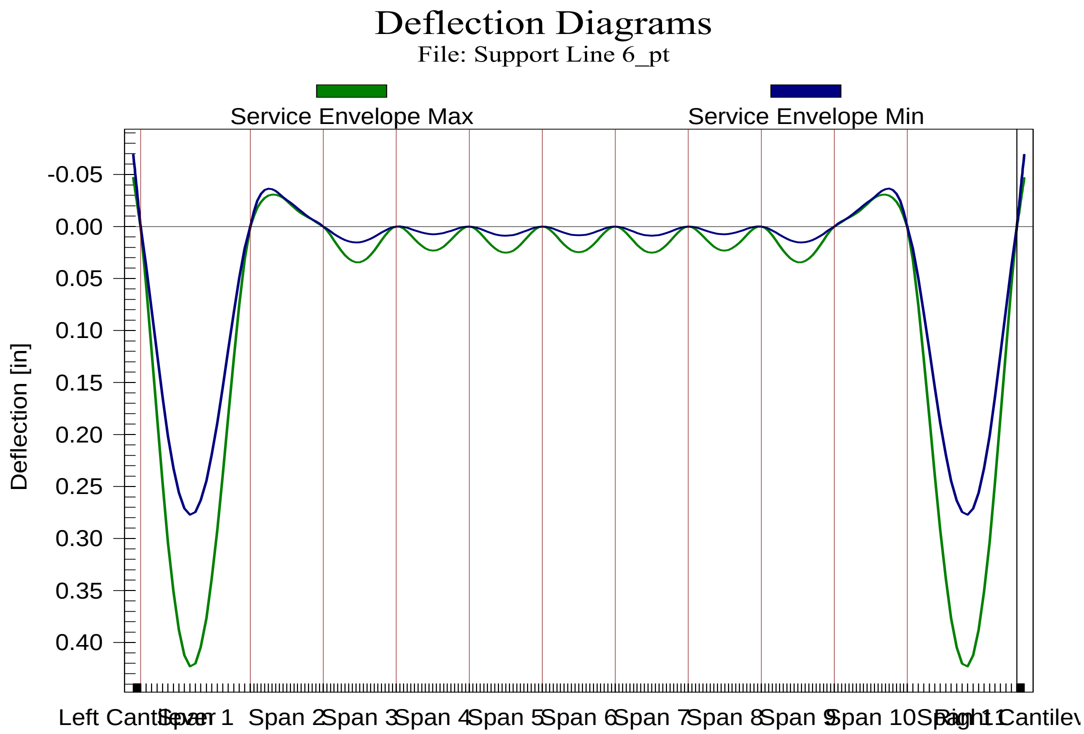


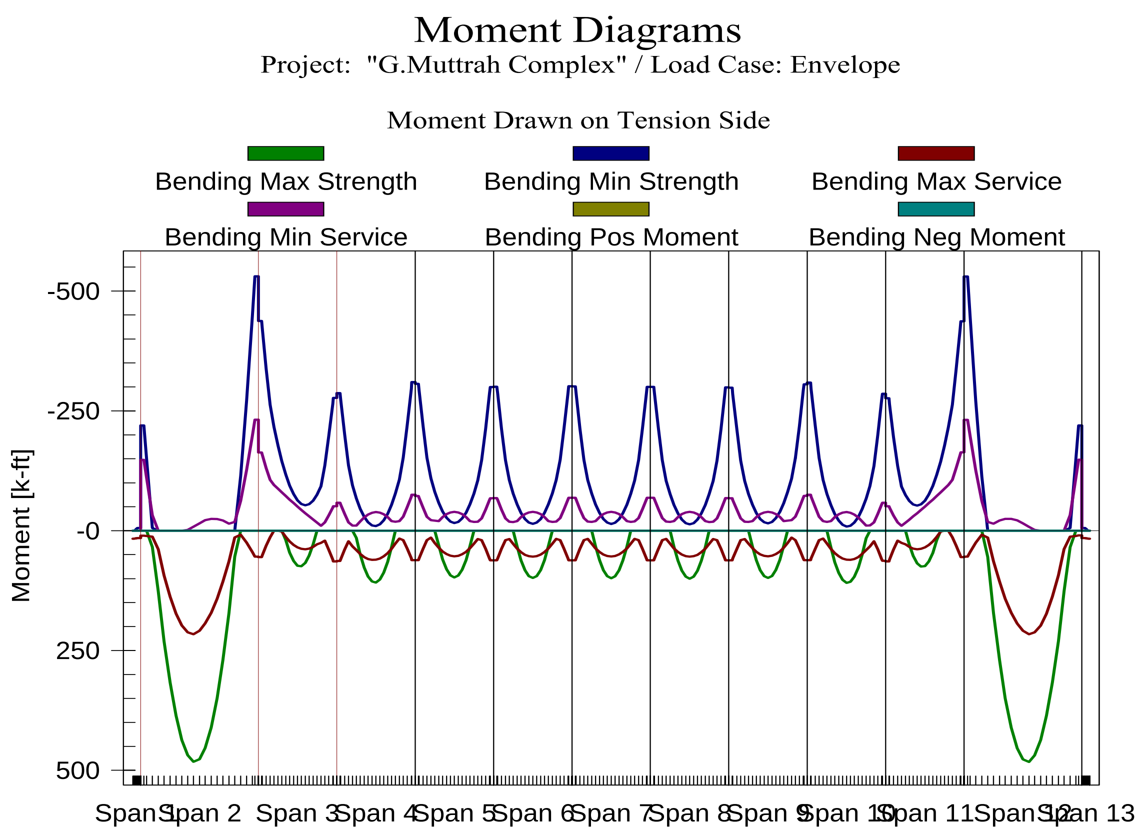


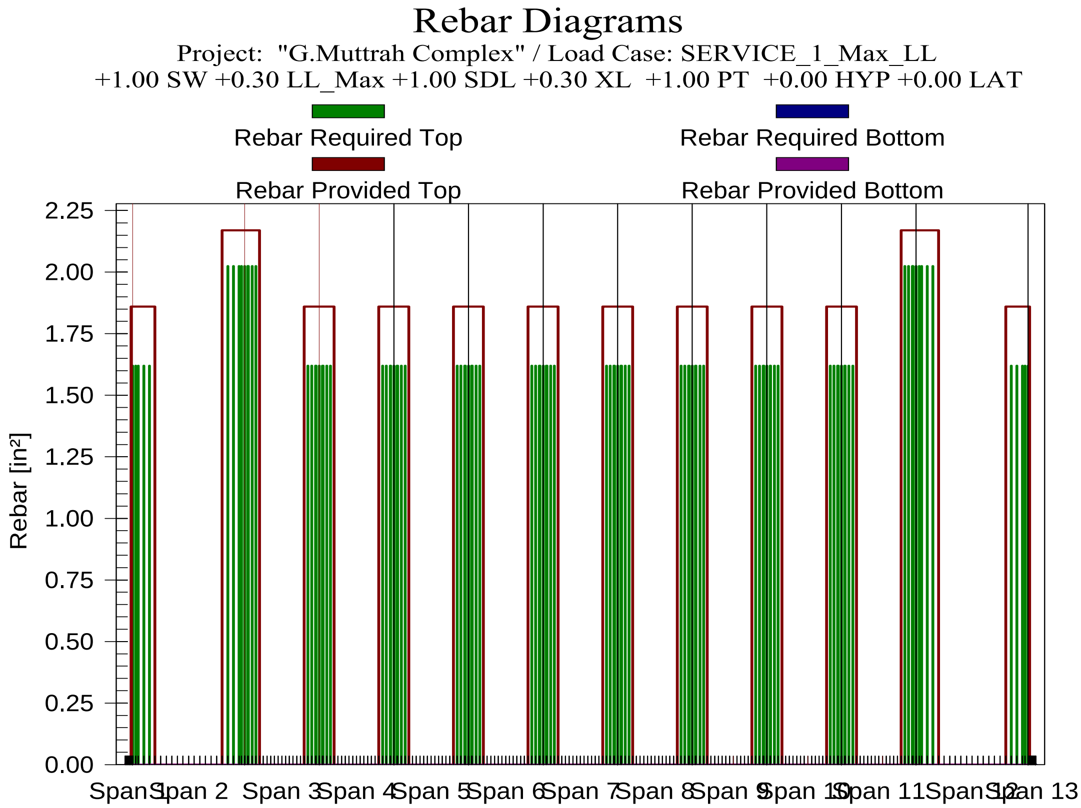
Office Floor:

E-W Direction:

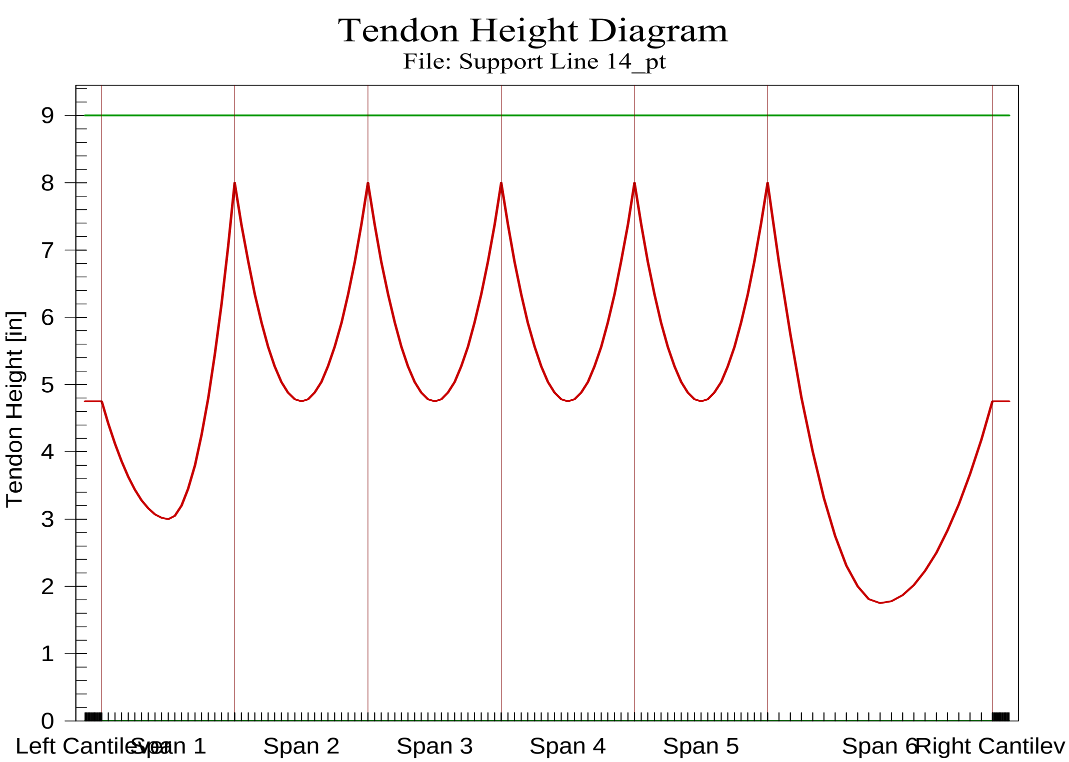


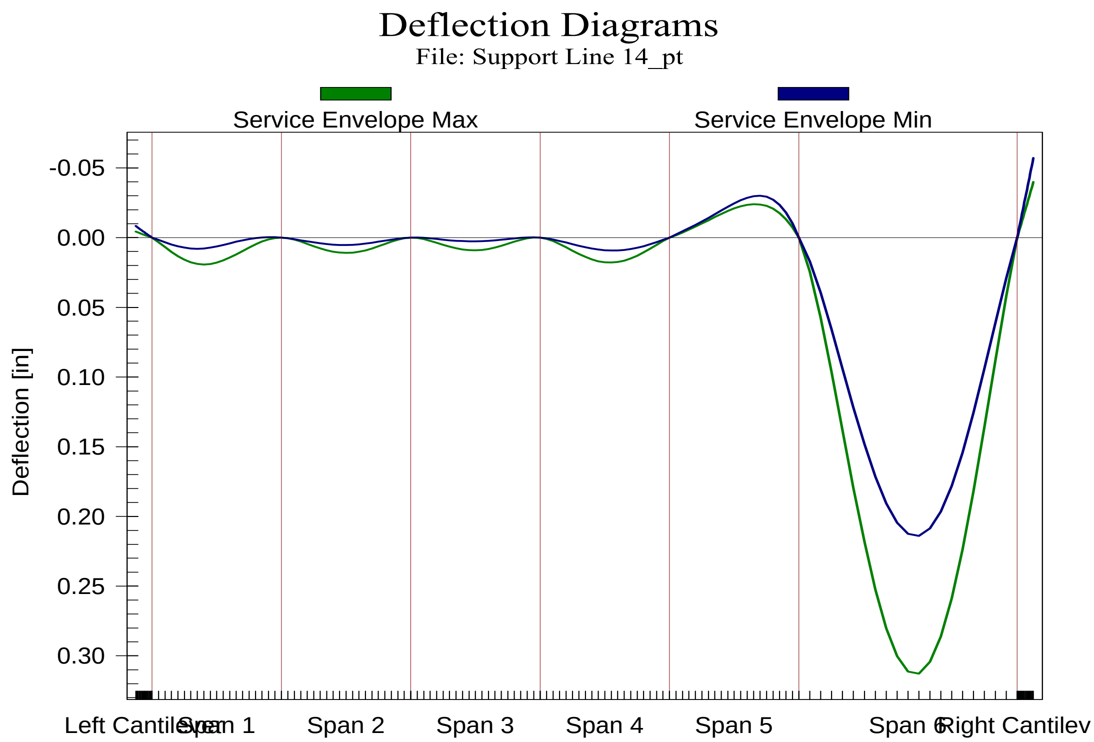

Deflection Diagrams
File: Support Line 6_pt
Service Envelope Max
Service Envelope Min
Deflection [in]
-0.05
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
Span 2
Span 3
Span 4
Span 5
Span 6
Span 7
Span 8
Span 10

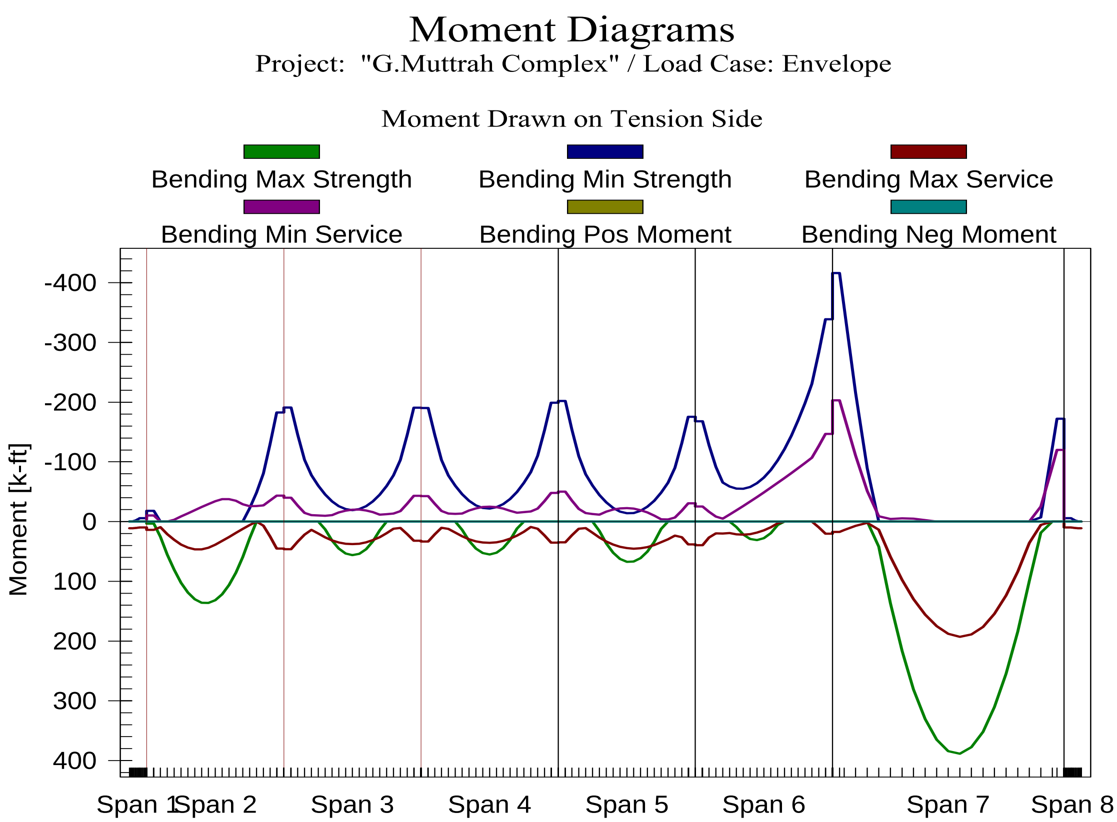

Moment Diagrams
Project: "G.Muttrah Complex" / Load Case: Envelope
Moment Drawn on Tension Side
Bending Max Strength
Bending Min Strength
Bending Max Service
Bending Min Service
Bending Pos Moment
Bending Neg Moment
Moment [k-ft]
-500
-250
-0
250
500
Span 2
Span 3
Span 11
Span 13

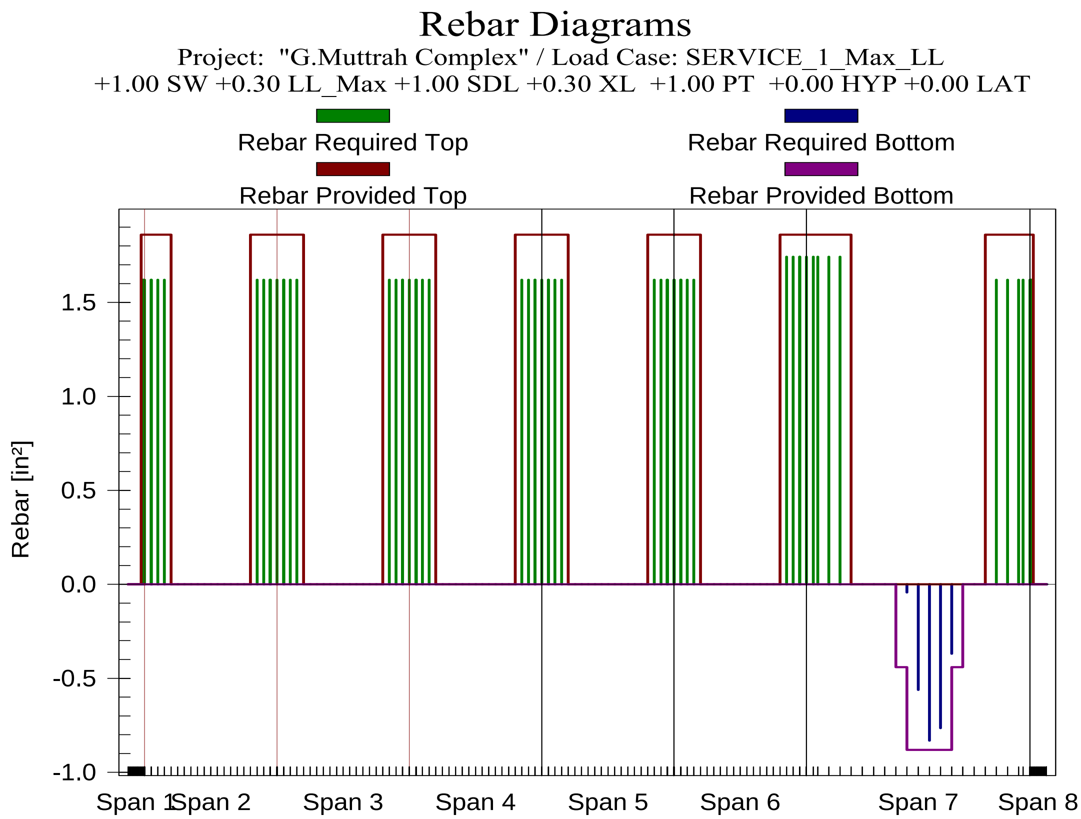


N-S Direction:



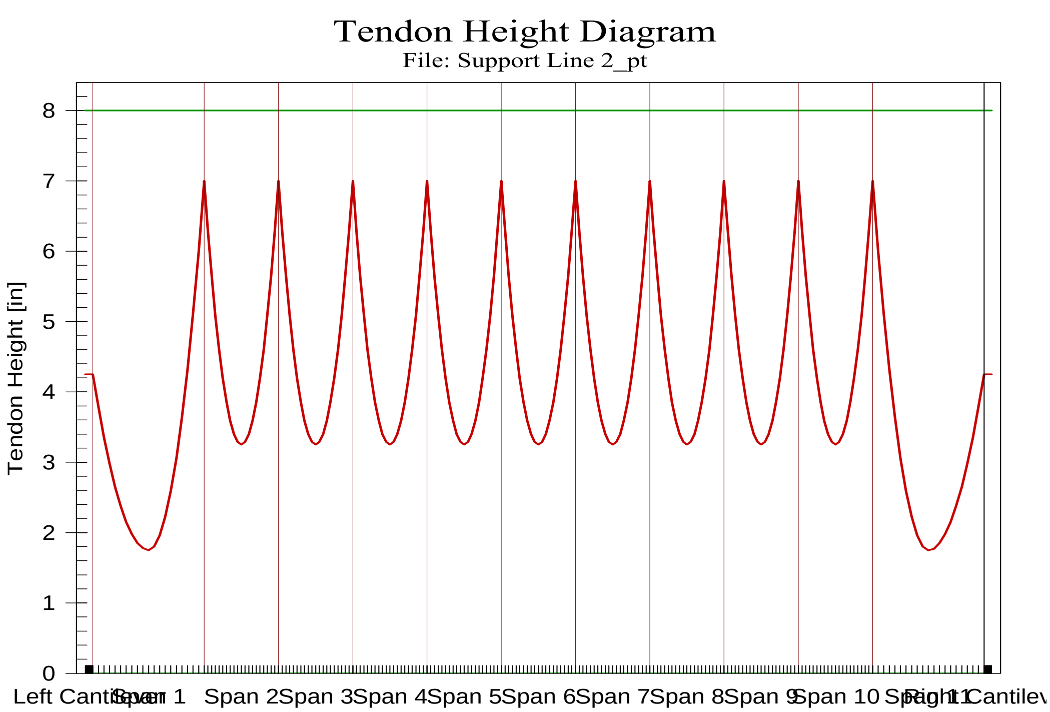

Deflection Diagrams
File: Support Line 14_pt
Service Envelope Max
Service Envelope Min
Deflection [in]
-0.05
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Left Cantilever
Span 1
Span 2
Span 3
Span 4
Span 5
Span 6
Right Cantilev

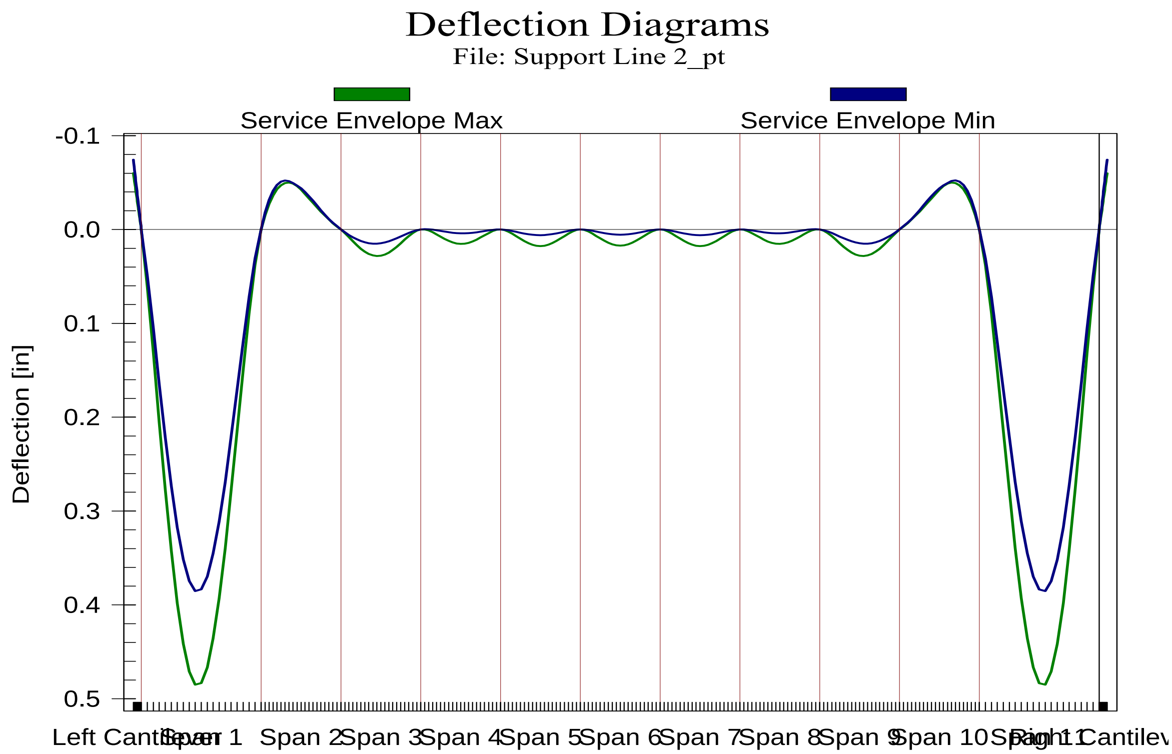

Moment Diagrams
Project: "G.Muttrah Complex" / Load Case: Envelope
Moment Drawn on Tension Side
Bending Max Strength
Bending Min Strength
Bending Max Service
Bending Min Service
Bending Pos Moment
Bending Neg Moment
Moment [k-ft]
-400
-300
-200
-100
0
100
200
300
400
Span 1
Span 2
Span 3
Span 4
Span 5
Span 6
Span 7
Span 8

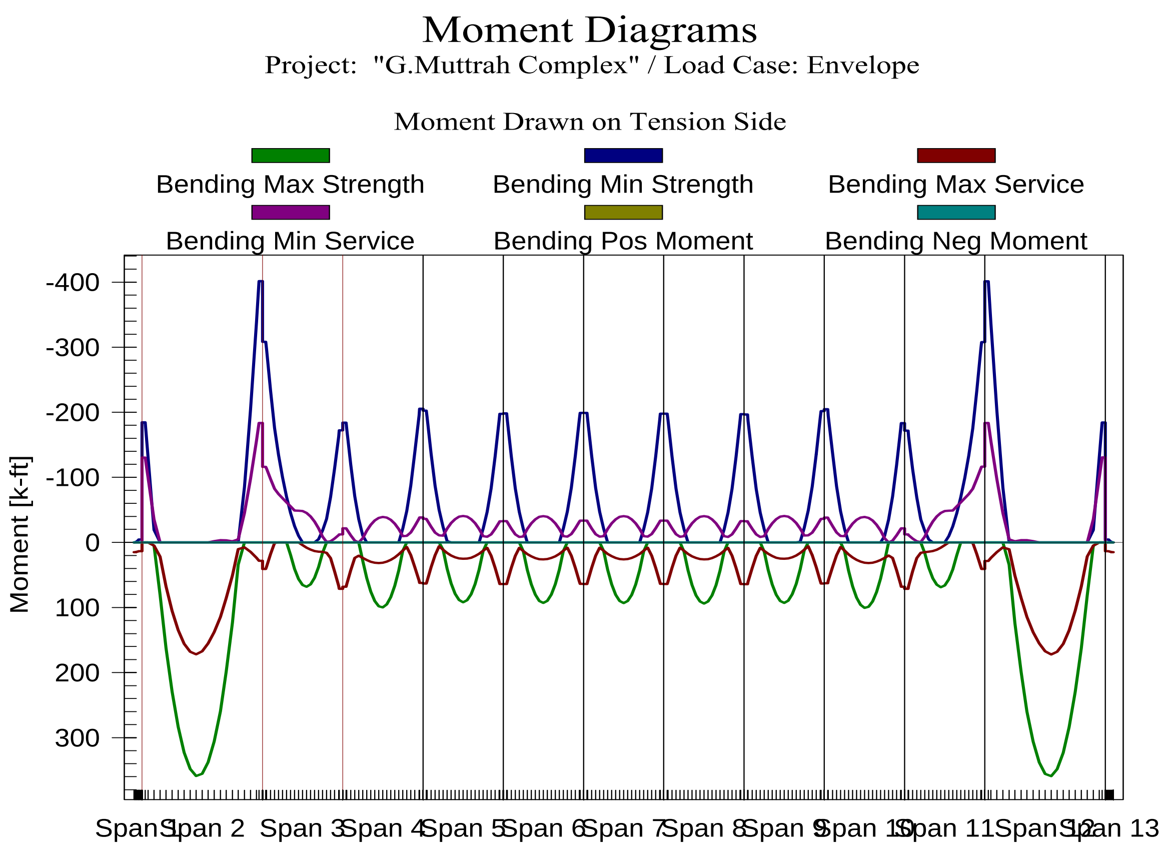


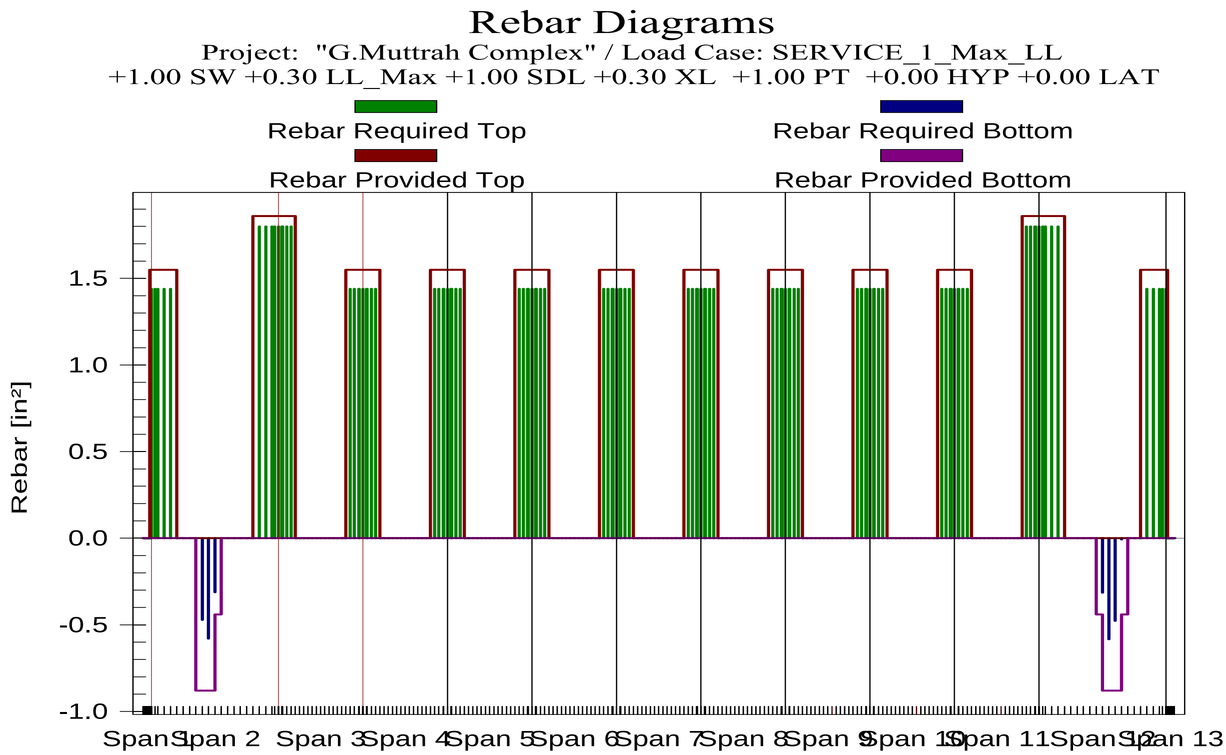
Retail Floor:

E-W Direction:

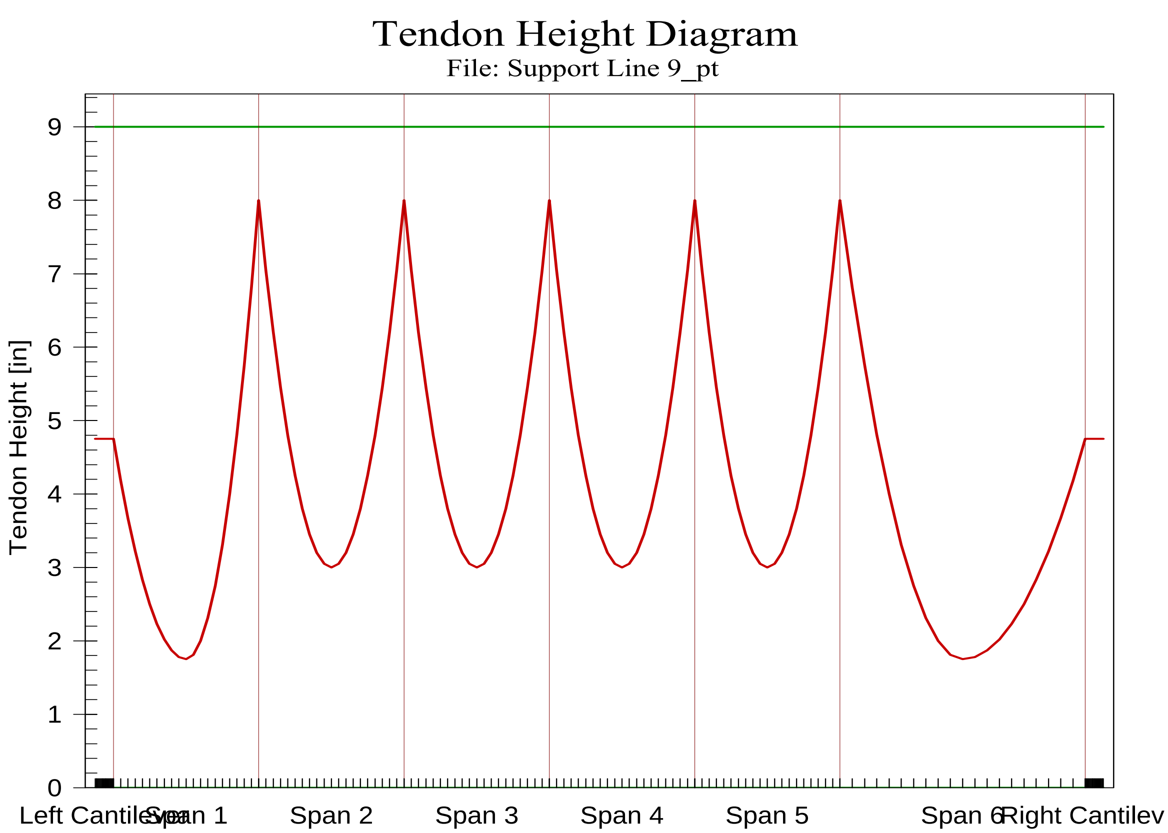


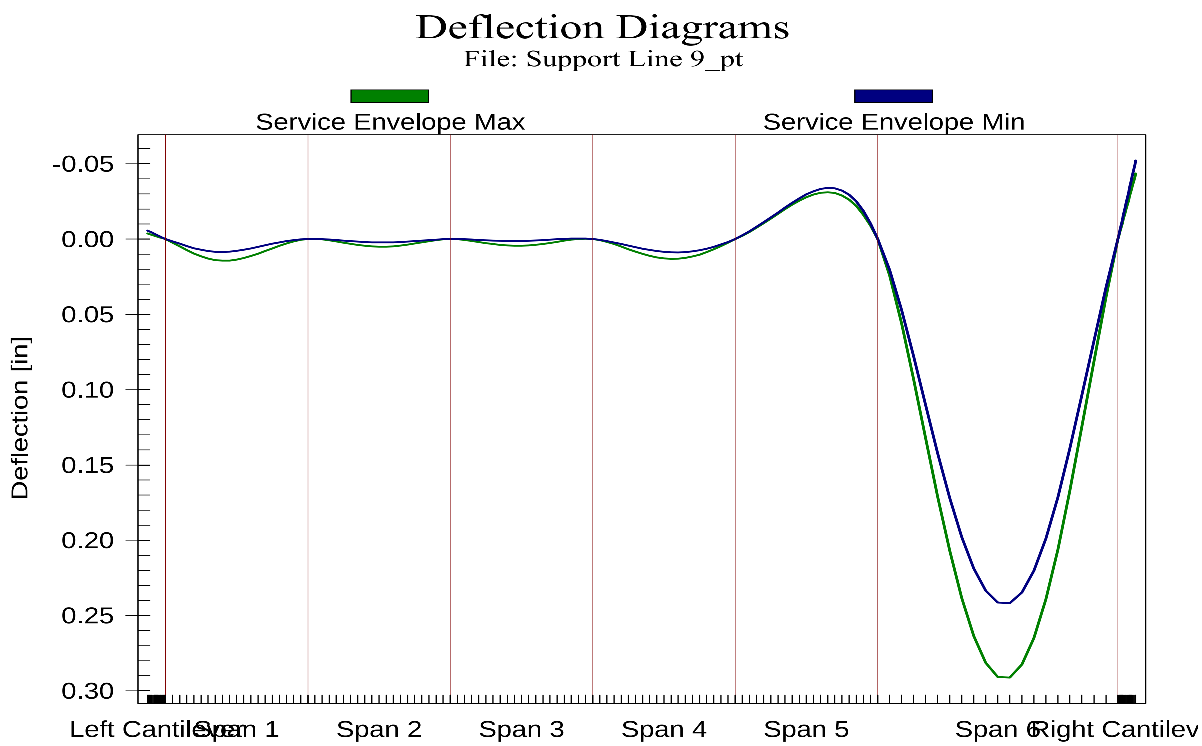

Deflection Diagrams
File: Support Line 2_pt
Service Envelope Max
Service Envelope Min
Deflection [in]
-0.1
0.0
0.1
0.2
0.3
0.4
0.5
Left Cantilever
Span 1
Span 2
Span 3
Span 4
Span 5
Span 6
Span 7
Span 8
Span 9
Span 10
Span 11
Right Cantilev

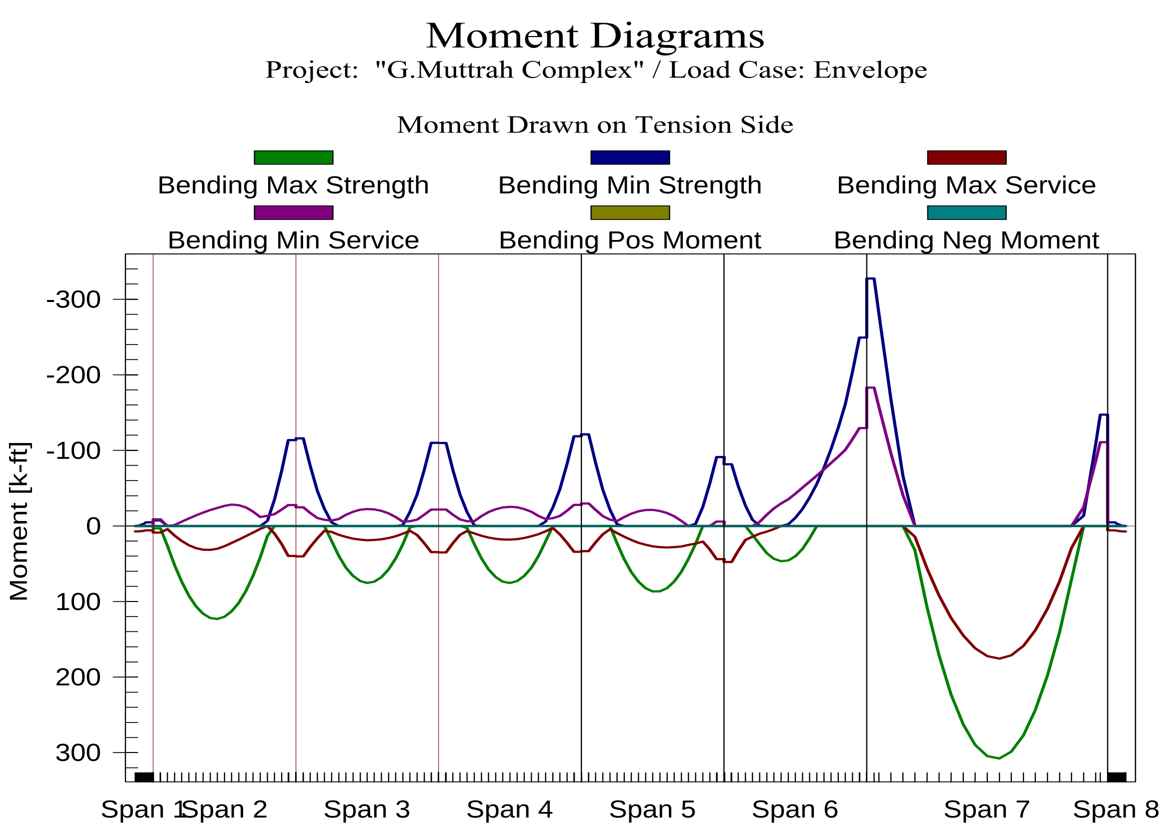

Moment Diagrams
Project: "G.Muttrah Complex" / Load Case: Envelope
Moment Drawn on Tension Side
Bending Max Strength
Bending Min Strength
Bending Max Service
Bending Min Service
Bending Pos Moment
Bending Neg Moment
Moment [k-ft]
-400
-300
-200
-100
0
100
200
300
Span 1
Span 2
Span 3
Span 4
Span 5
Span 6
Span 7
Span 8
Span 9
Span 10
Span 11
Span 12
Span 13

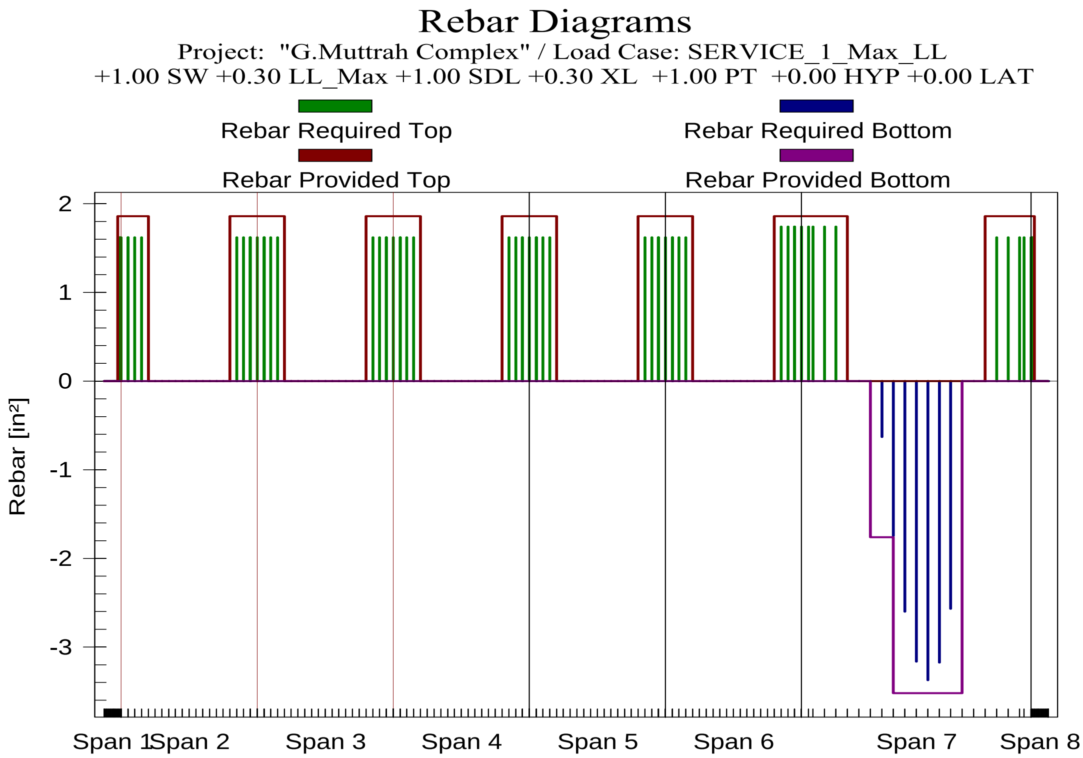


N-S Direction:



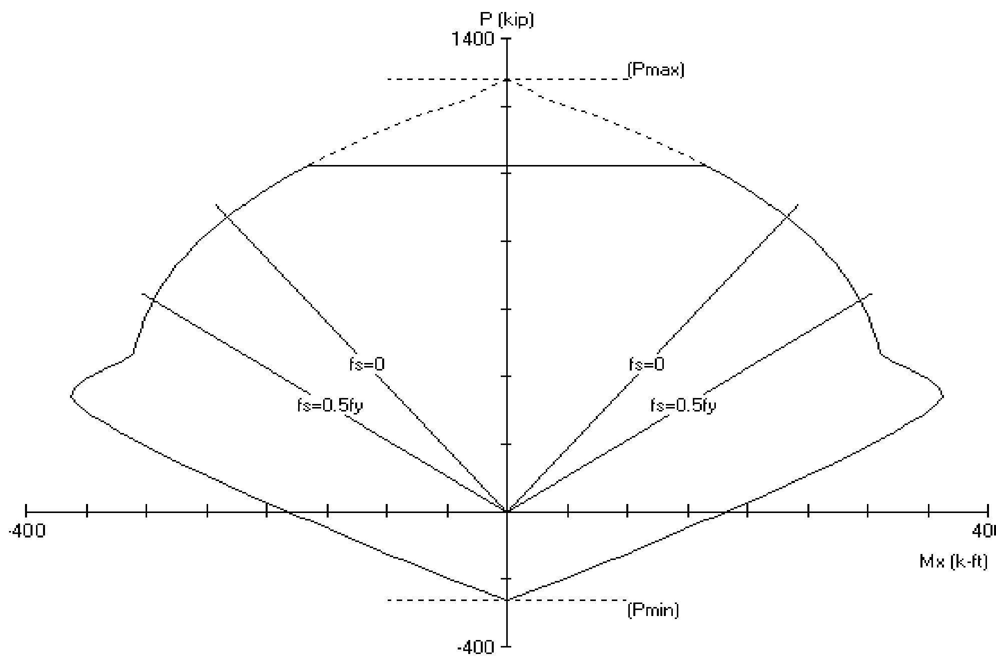

Deflection Diagrams
File: Support Line 9_pt
Service Envelope Max
Service Envelope Min
Deflection [in]
-0.05
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Left Cantilever
Span 1
Span 2
Span 3
Span 4
Span 5
Span 6
Right Cantilev


Moment Diagrams
Project: "G.Muttrah Complex" / Load Case: Envelope
Moment Drawn on Tension Side
Bending Max Strength
Bending Min Strength
Bending Max Service
Bending Min Service
Bending Pos Moment
Bending Neg Moment
Moment [k-ft]
-300
-200
-100
0
100
200
300
Span 1
Span 2
Span 3
Span 4
Span 5
Span 6
Span 7
Span 8



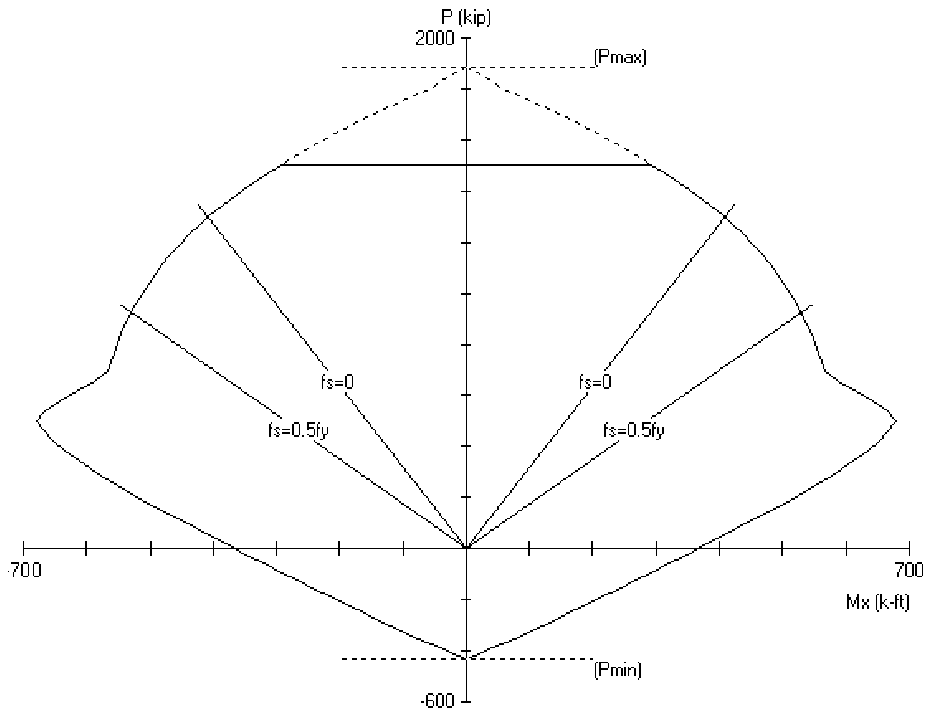
Column Design:

Interior column:



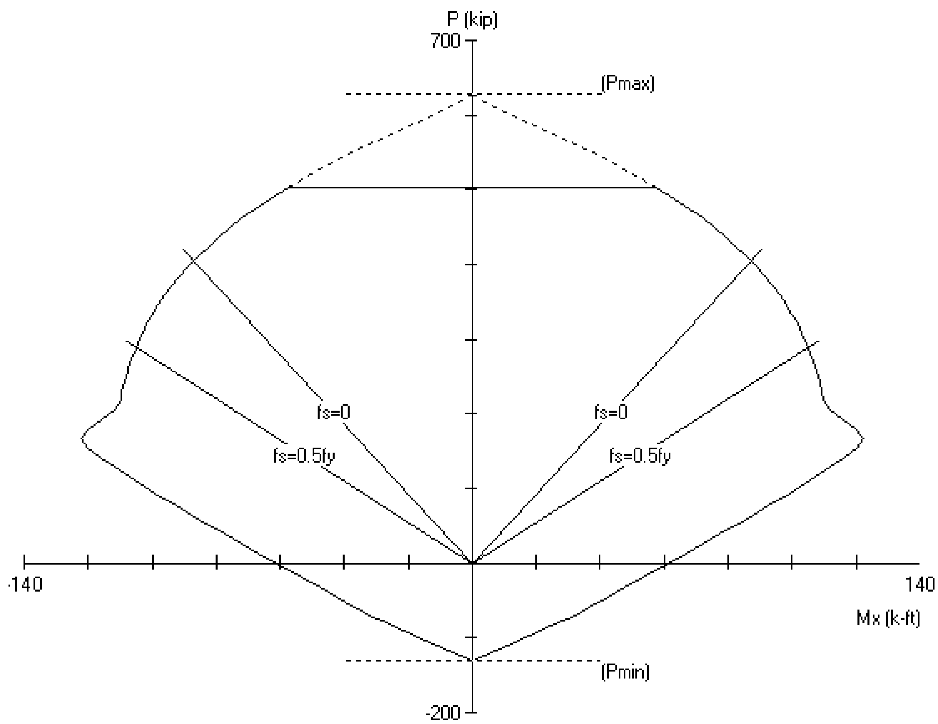
```
Section:
========
   Rectangular: Width = 20 in              Depth = 20 in

   Gross section area, Ag =  400 in^2
   Ix =  13333.3 in^4                      Iy =  13333.3 in^4
   Xo =  0 in                              Yo =  0 in

Reinforcement:
==============
   Rebar Database: ASTM A615
   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
   ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
   #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
   #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
   #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
   # 14      1.69        2.25   # 18      2.26        4.00

   Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
   phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

   Layout: Rectangular
   Pattern: All Sides Equal  (Cover to transverse reinforcement)
   Total steel area, As = 4.80 in^2 at 1.20%
   8 #7    Cover = 1.5 in

Axial Load and Corresponding Moment Capacities: (see user's manual for notation)
===============================================
   Load          fPn        fMnx     N.A. depth
    No.          kip         k-ft          in
   ---- ------------ ------------ -----------
      1         -0.0        182.8        2.49
                           -182.8        2.49
```

Exterior Columns (All Floors):



```
Section:
========
   Rectangular: Width = 24 in             Depth = 24 in

   Gross section area, Ag =  576 in^2
   Ix =  27648 in^4                       Iy =  27648 in^4
   Xo =  0 in                             Yo =  0 in

Reinforcement:
==============
   Rebar Database: ASTM A615
   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
   ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
   #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
   #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
   #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
   # 14      1.69        2.25   # 18      2.26        4.00

   Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
   phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

   Layout: Rectangular
   Pattern: All Sides Equal  (Cover to transverse reinforcement)
   Total steel area, As = 8.00 in^2 at 1.39%
   8 #9   Cover = 1.5 in

Axial Load and Corresponding Moment Capacities: (see user's manual for notation)
===============================================
   Load         fPn         fMnx     N.A. depth
    No.         kip         k-ft         in
   ---- ------------ ------------ -----------
      1          0.0        366.9        3.13
                           -366.9        3.13
```

Exterior column (North face of 1st and 2nd Floor):



```
Section:
========
   Rectangular: Width = 14 in              Depth = 14 in

   Gross section area, Ag =  196 in^2
   Ix =  3201.33 in^4                      Iy =  3201.33 in^4
   Xo =  0 in                              Yo =  0 in

Reinforcement:
==============
   Rebar Database: ASTM A615
   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
   ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
   #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
   #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
   #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
   # 14      1.69        2.25   # 18      2.26        4.00

   Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
   phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

   Layout: Rectangular
   Pattern: All Sides Equal  (Cover to transverse reinforcement)
   Total steel area, As = 2.40 in^2 at 1.22%
   4 #7   Cover = 1.5 in

Axial Load and Corresponding Moment Capacities: (see user's manual for notation)
===============================================
   Load          fPn        fMnx    N.A. depth
    No.          kip        k-ft            in
   ---- ------------ ------------ -----------
      1         -0.0         61.3        1.94
                            -61.3        1.94
```

Punching Shear Design:

Number of studrails per column: 9
Number of studs per studrail: 18
Stud diameter: 0.5 in

Typical stud spacing, S: 3.375 in
End stud spacing, S_0: 3.375 in
Overall height of studrail: 7.500 in

Inner Critical Section (d/2 outside of column face):

Common Properties

Area, A_c: 724.0 in^2

Critical Section Perimeter, b_0: 94.89 in

Natural Axis Properties

Centroid coordinate, e_x: 5.272 in
Centroid coordinate, e_y: 0.0 in
Section moment of inertia, I_x: 1.408×10^5 in^4
Section moment of inertia, I_y: 8.048×10^4 in^4
Section product of inertia, I_{xy}: 0.0 in^4

Principal Axis Properties

Centroid coordinate, e_1: 5.272 in
Centroid coordinate, e_2: 0.0 in
Section moment of inertia, I_1: 1.408×10^5 in^4
Section moment of inertia, I_2: 8.048×10^4 in^4
Principal axis rotation, (theta): 0.0 degrees
Moment fraction, γ_{v1}: 0.400
Moment fraction, γ_{v2}: 0.3735

Natural Axis Loads

V_u: 96.00 k
M_{ux}: 230.0 k-ft
M_{uy}: 0.0 k-ft

Principal Axis Loads

V_u: 96.00 k
M_{u1}: 230.0 k-ft
M_{u2}: 0.0 k-ft

Stresses

Maximum shear stress, v_u: 306.1 psi
at x = -15.82 in, y = -15.82 in

Shear resistance, ϕv_n (concrete only):
212.1 psi

Shear resistance, ϕv_n (with Studrails):
313.0 psi
Shear resistance, ϕv_n (upper limit):
318.2 psi

Outer Critical Section (d/2 outside of reinforced zone):

Common Properties
Area, A_c: 2009 in^2
Critical Section Perimeter, b_0: 263.4 in

Natural Axis Properties
Centroid coordinate, e_x: 37.80 in
Centroid coordinate, e_y: 0.0 in
Section moment of inertia, I_x: 5.798×10^6 in^4
Section moment of inertia, I_y: 1.532×10^6 in^4
Section product of inertia, I_{xy}: 0.0 in^4

Principal Axis Properties
Centroid coordinate, e_1: 37.80 in
Centroid coordinate, e_2: 0.0 in
Section moment of inertia, I_1: 5.798×10^6 in^4
Section moment of inertia, I_2: 1.532×10^6 in^4
Principal axis rotation, (theta): 0.0 degrees
Moment fraction, γ_{v1}: 0.4619
Moment fraction, γ_{v2}: 0.2974

Natural Axis Loads
V_u: 96.00 k
M_{ux}: 230.0 k-ft
M_{uy}: 0.0 k-ft

Principal Axis Loads
V_u: 96.00 k
M_{u1}: 230.0 k-ft
M_{u2}: 0.0 k-ft

Stresses
Maximum shear stress, v_u: 102.4 psi
at x = -15.82 in, y = -76.56 in
Shear resistance, ϕv_n: 106.1 psi

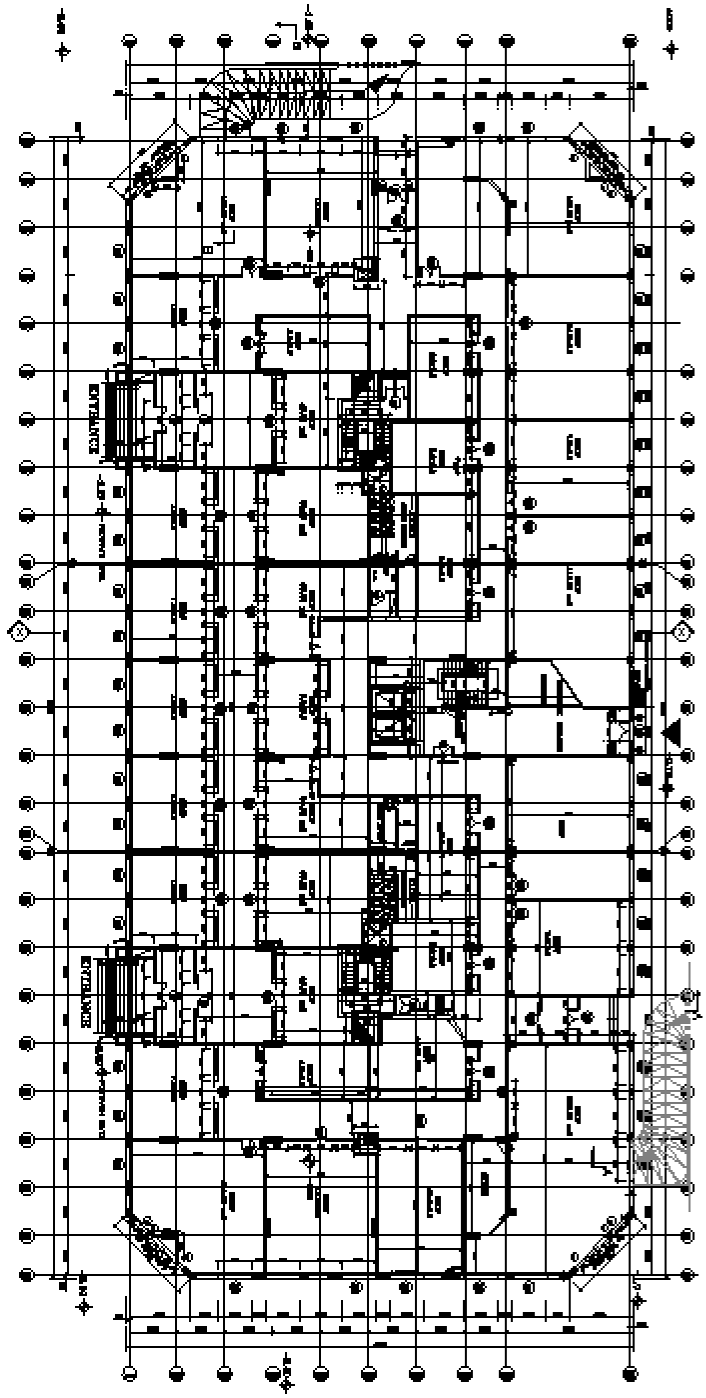
Design Comments:
For prestressed slabs, concrete strength above 4900 psi does not result in increased punching resistance.

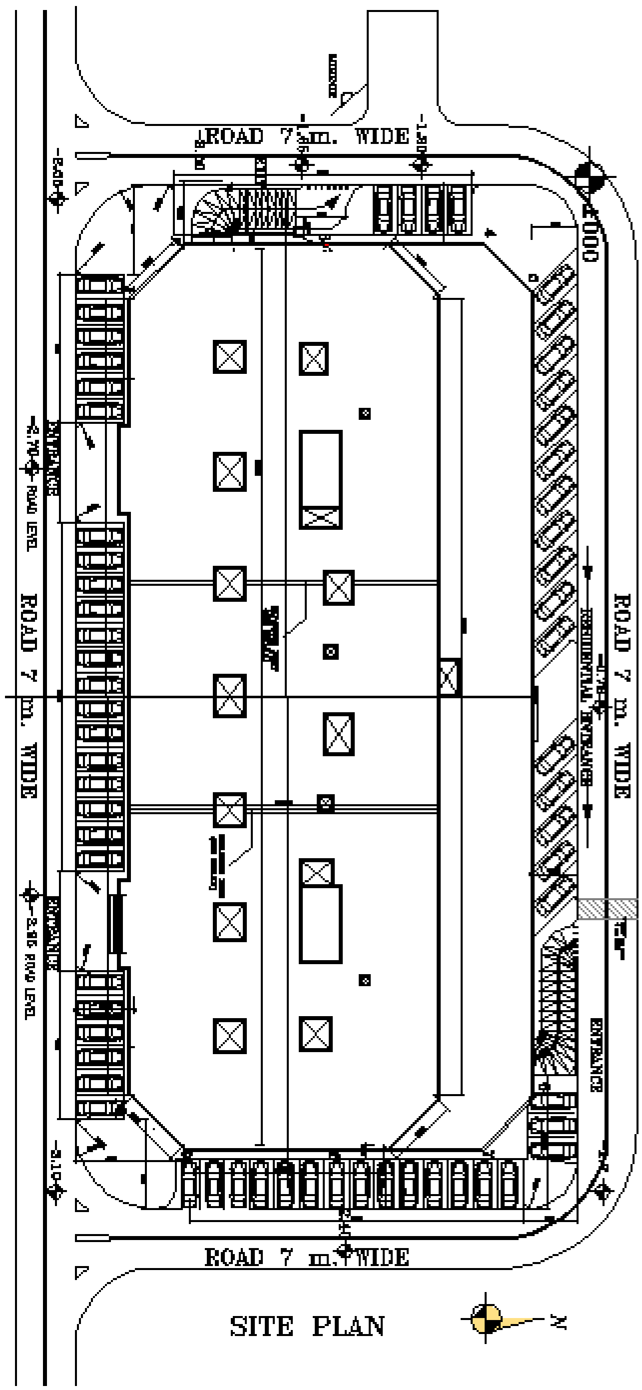
Lateral System Calculations:

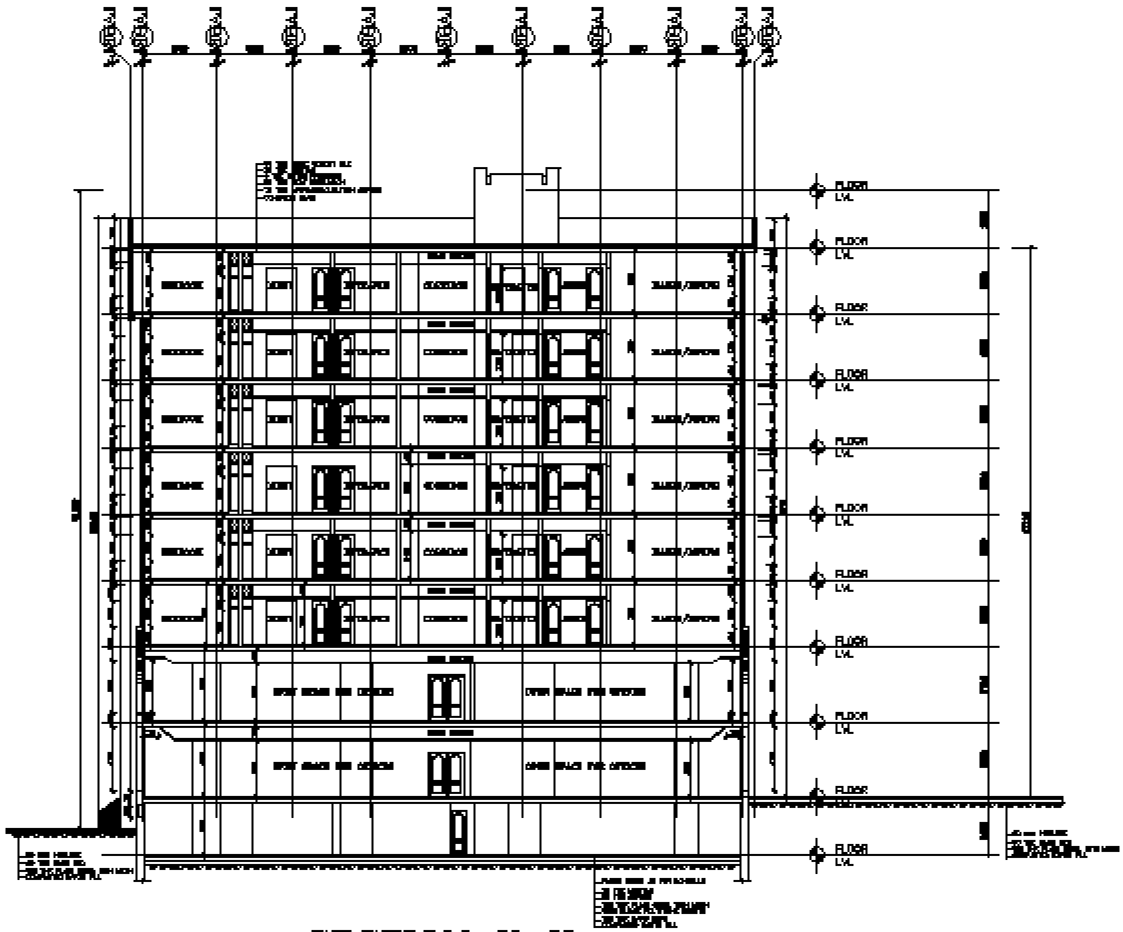
Story	COM X	COM Y	COR X	COR Y	Story Force	Torsional Moment
1	132.50	44.4	132.5	38.0	56.1	359.0
2	132.50	44.4	132.5	38.0	51.7	330.9
3	132.50	40.6	132.5	41.7	44.1	48.5
4	132.50	40.6	132.5	41.7	45.5	50.1
5	132.50	40.6	132.5	41.7	46.7	51.4
6	132.50	40.6	132.5	41.7	47.8	52.6
7	132.50	40.6	132.5	41.7	48.9	53.8
8	132.50	40.6	132.5	41.7	49.8	54.8
Roof	132.50	40.6	132.5	41.7	50.5	55.6

Story	I_{xx}	I_{yy}	I_p	Story Force	Shear in Wall
1	22700	493100	515800	134	11.12225652
2	22700	493100	515800	115	9.542406338
3	22700	493100	515800	98	8.160666399
4	22700	493100	515800	101	8.410475914
5	22700	493100	515800	104	8.660312642
6	22700	493100	515800	106	8.826829642
7	22700	493100	515800	109	9.076679975
8	22700	493100	515800	111	9.243224188
Roof	22700	493100	515800	113	9.409795612

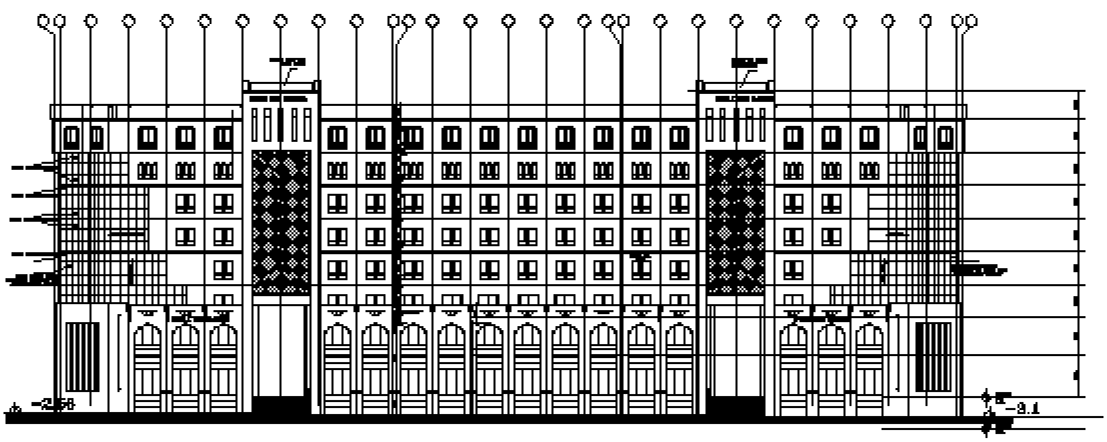
Appendix B: Plans and Details




ROAD 7 m. WIDE
ROAD 7 m. WIDE
ROAD 7 m. WIDE
ROAD 7 m. WIDE
ENTRANCE
ENTRANCE
ENTRANCE
RESIDENTIAL ENTRANCE
SITE PLAN
N



SECTION X–X



FRONT ELEVATION