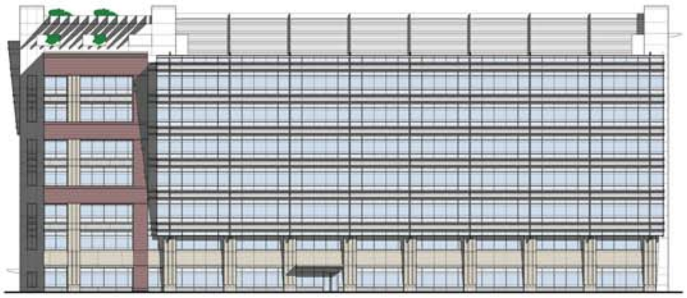


Parkridge Center - Phase VI

Reston, VA

<u>Structural</u>
Post-Tensioned Slab and Beam

<u>Breadth Topics</u>
Construction Management
Mechanical

Don Bockoven
Structural
Advisor: Dr. Boothby

Parkridge Center - Phase VI

Reston, VA

General Statistics:

Size: **226,000 sq.ft.**
Stories: **7**
Delivery Method: **Design-Bid-Build**
Codes:
- **BOCA 1996**
- **IBC 2000**
- **2000 IPC**

Zoning: **Fairfax County Zoning Ordnance, I-3**

Project Team:

Owner: **Walker & Company**
Architects: **Hickok Warner Cole Architects**
Structural: **Structural Design Group, LTD.**
MEP: **Girard Engineering**
Civil: **William H. Gordon Associates, Inc.**
Building Specifications: **Ronald M. Beard CCS, LLC**
Landscape Consultant: **Parker Rodriguez**

Architecture:

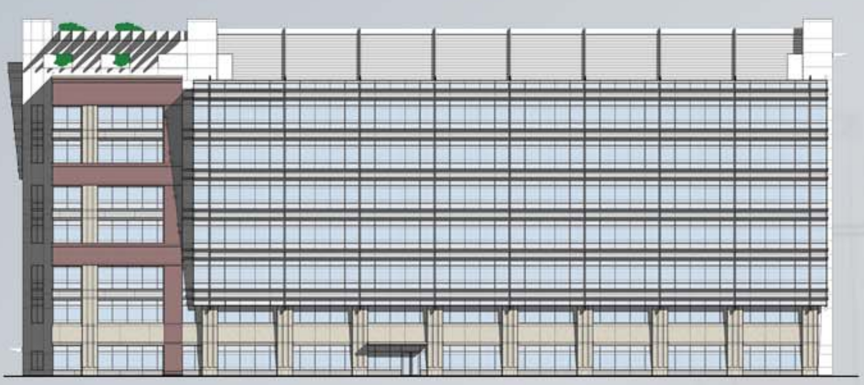
- Sloping Columns on the south face
- Sky garden
- Arcade along the north face
- Exterior facade made up of a series of precast brick and concrete panels.

Structural:

- Composite Floor System over W16 interior beams and W18-W21 Girders.
- All Beams are cambered at 1-1/4”
- The Main Lateral System consists of braced frames
  - The cross bracing of the frames are HSS 8X8 and HSS 10X10
- The Columns on the south face are sloped outward from the ground up.
- The foundation is made up of shallow strip footings

Lighting/Electrical:

- (2) 3000A 480/277 3 Phase 4W main switchboards
- The electrical lines enter the buidling through (12) 4” PVC schedule 40 conduits encased in concrete.
- (2) 2000A PLug-in busways transfer power to each floors (4) 400A Panelboards and 112.5 KVA Transformers
- The lighting of each floor will be done by the individual Tennants
- There is a 350 KW 480/277V 3 Phase 4W Diesel Emergency Generator.



Mechanical:

- Variable Air Volume System
- (2) main supply trunks to each floor serviced by the main air handling units (AHU) on the roof
- The ground floor lobby area is serviced by a seperate AHU.

Executive Summary

Parkridge Center – Phase VI is a 7 story 226,000 sq.ft. commercial office building located in Reston, VA. The building is designed to a maximum height of 115'. The south face of the building is made up of sloping columns that slope outward from the ground level to the roof. The north face of the building contains an arcade created by stepped portions of additional floor area on the second floor through the fifth floor.

The existing foundation for Parkridge 6 is a shallow foundation system made up primarily of spread footings. The typical floor is a composite system with 3 ¼" of lightweight concrete on a 2"-20 gauge steel deck. The building grid consists of 3 bays in the N-S direction spaced at 37'-2", 35'-0", and 37'-2" respectively. In the E-W direction there are 10 bays with the first bay on both ends being 25'-8" and all others 25'-0".

The existing lateral system for Parkridge 6 is a series of braced frames. In the N-S direction there are 2 frames and in the E-W direction there are 3 frames. The bracing elements of these frames are made up of HSS sections ranging from 8x8 to 12x12.

The alternative system that was studied for this report was a post-tensioned slab and beam system. For the purposes of this report the post-tensioned system was designed to keep the existing bay dimensions and if possible the existing floor to floor dimensions. The design of the post-tensioned system was accomplished by using the ACI 318-05 manual, the RISA3D application, and the PCA Column application. Excel spreadsheets were also used to expedite calculations.

The post-tensioned slab was found to be a 7 inch slab spanning from beam to beam in the short direction. The post-tensioned beams range from 28in x 38in at the edge to 28in x 34in at the interior on the roof. The concrete strength of the slab is designed to be 5000psi and the beams are 8000psi. Both the slab and beams were designed to be uncracked sections under service loading. There are also sections of beams which are not pot-tensioned to keep beam size variations to a minimum to speed up construction.

The column sizes range from 24in x 28in for the sloping columns to 30in x 34in for the interior columns. The columns are designed to have a concrete strength of 6000psi.

The lateral system was first investigated as a series of moment frames in the N-S direction. However this was found to be inadequate for the lateral loading when considering torsional effects. The next alternative was to use cast-in-place shear walls in both the N-S and E-W directions. The shear walls were designed to be 12 inches wide having a concrete strength of 6000 psi. The overall deflection of the shear walls was well within the H/400 industry standard.

The foundation system will need to be switched from a shallow foundation system to a deep foundation system. The additional loading from the self weight of the concrete system would require either caissons or piles. The foundation system was not explicitly designed in this report.

The cost of the proposed post-tensioned concrete system is approximately $3.5 million a savings of about $3.9 million over the current steel system. The cost saving however is over shadowed by the significantly extended schedule of about 9 months longer than the steel system. The post-tension concrete system will also require specialty shoring for the sloping columns.

The mechanical system was changed from having individual air conditioning units on each floor to two air cooled chiller units on the roof. This system was found to be more efficient for energy and cooling purposes, but not practical as each floor may have different tenants making the billing for the mechanical costs more difficult to split correctly as not to overcharge a tenant.

Overall I do not recommend the proposed post-tensioned floor system for Parkridge Center – Phase VI. The main reasons are the post-tensioned systems increased schedule, impact on the foundation system, and impact on the floor to floor height. It was concluded that the composite steel system was the more efficient system for this building.

Table of Contents

Acknowledgements

I would like to thank Walker and Company for allowing me to investigate Parkridge Center – Phase VI for my AE senior thesis, and for all the information they provided about the building and its design team.

I would also like to thank Structural Design Group, Ltd. for there support with any and all questions I had about the current design as well as possible alternatives. I also look forward to beginning my design career with them this summer.

Thank you to Rob Holzbach and Hickok Warner Cole Architects for the information on the architectural design and layout of the building.

Also I would like to thank all the AE faculty and message board participants for always answering my questions to the best of their ability.

Introduction

The proposed Parkridge Center – Phase VI building is a 226,000 Sq. Ft., seven story commercial office building located in Reston, VA. The building is currently designed to a maximum height of 115'. The south face of the building slopes outward from the ground level to the roof, while on the north face of the building there are stepped portions from the second floor to the 5th floor creating an arcade at ground level. All of the occupied space is above grade. There is no sub grade portion of the building other than the foundations.

Existing Gravity System

Foundations

Parkridge 6 rests on a shallow foundation system consisting of spread footings ranging in size from 5' x 5' to 20' x 20' with depths ranging from 12" to 42". The lateral resisting elements of the building rest on mat foundations. The allowable bearing pressure is 3000 psf. The slab on grade is 4" thick and is reinforced with a 6x6-10/10 welded wire mesh.

Floor System

Each floor contains the same three by ten bay core. The south most exterior bay on each floor varies based on the slope of the columns on the south face creating larger floor area on higher level floors. Floors 2 thru 5 contain extra floor area on the north side of the building above the arcade. The North-South (N-S) spans of the core three bays are 37'-2" for the exterior bays and 35'-0" for the interior bay. The East-West (E-W) spans of the core bays are 25'-8" for the first interior bay and then 25'-0" for the remaining bays. Intermediate beams are spaced at the third points of each bay and span in the N-S direction. Typical beam sizes for the core bays are W21's for the interior girders, W18's for the exterior girders, and W16's for the intermediate beams. Each beam is cambered to 1-1/4" this was done to account for serviceability issues arising from the members chosen. Each floor above grade uses a composite deck made up of 3 ¼" Lightweight concrete on 2"-20 gage steel deck. The total floor thickness is 5 1/4". The slab itself is to be reinforced with 6x6-10/10 WWM.

Columns

Each column extends 3 floors and is spliced above the slab. The columns along the south face of the building, column line A.1, are sloped outward from the ground to the roof. Typical sizes for the sloped columns begin at a W12x65 at the roof to the 7th floor, W12x96 from the 7th floor to the 4th floor, and W12x152 from the 4th floor to the foundation. Typical sizes for the interior columns range from a W12x53 at the upper floor to a W14x233 at the base of the building.

Existing Lateral System

Five braced frames make up the lateral system for the building. There are two frames in the N-S direction and three frames in the E-W direction. The diagonal members of the frames are HSS 10x10x1/2 for the N-S frames and HSS 8x8x1/2 for the E-W frames. Frames two and three are connected by two intermediate frames at the roof. The diagonal members of the two intermediate frames are HSS 8x8x1/4. Frame three is an eccentric braced frame while all the other frames are concentrically braced.

Typical Floor Plans – With Lateral Frames Highlighted

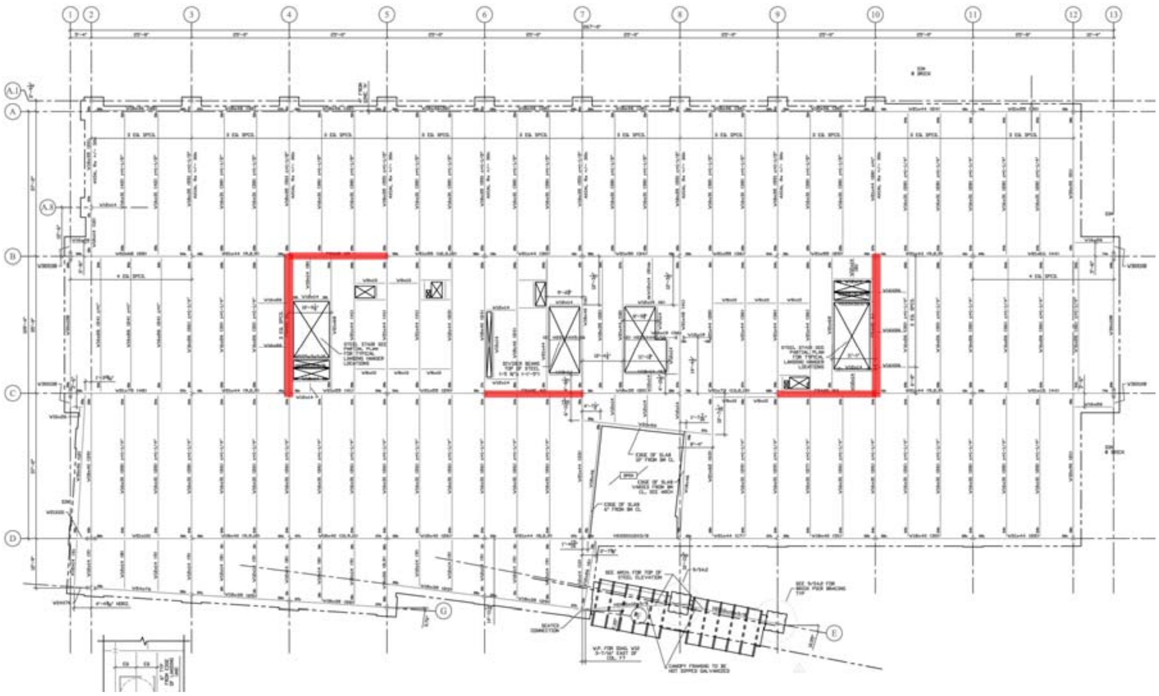


Fig F.1 – 2nd Floor plan with highlighted frames

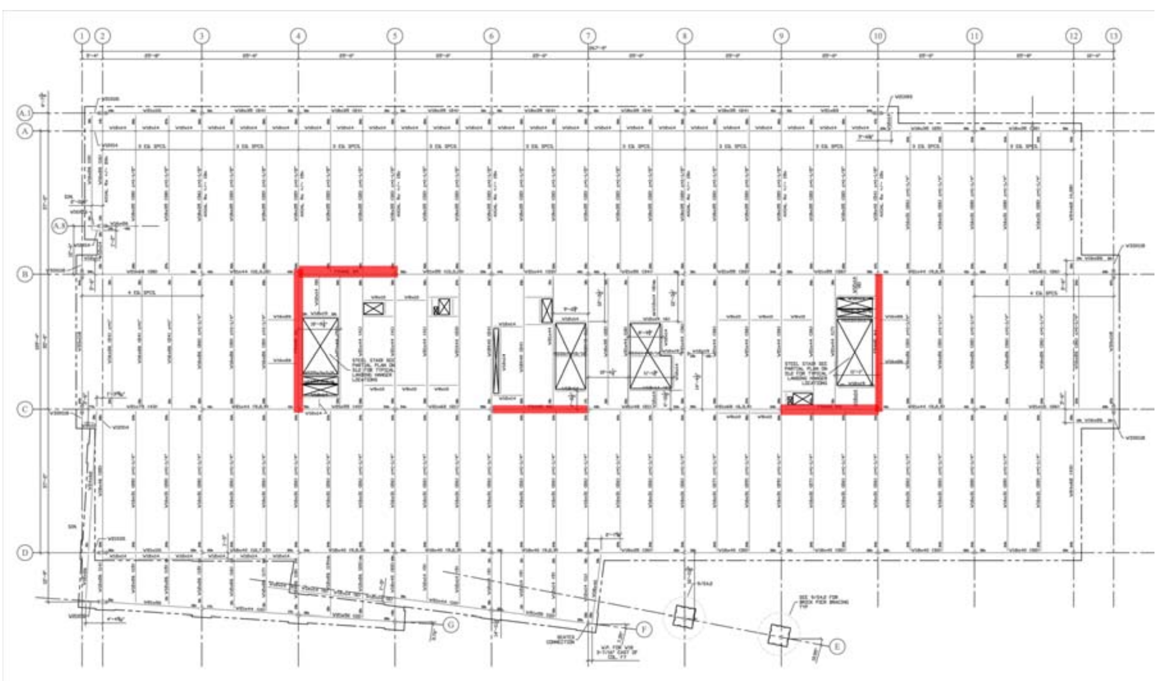


Fig F.2 – 3rd Floor plan with highlighted frames

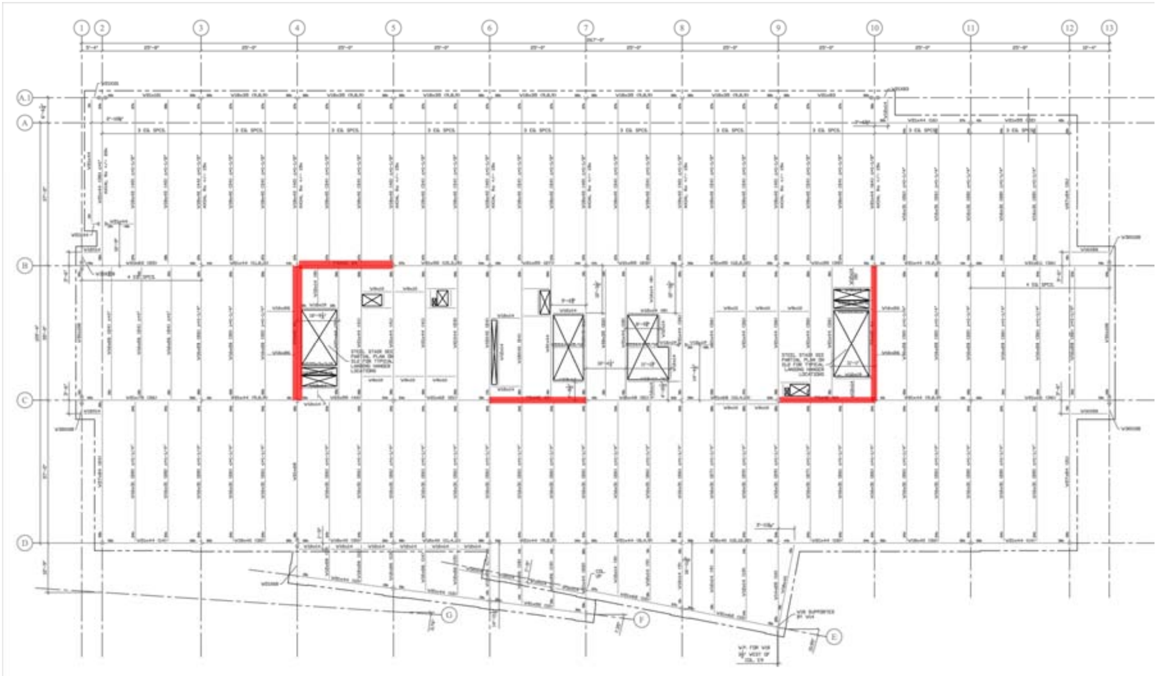


Fig F.3 – 4th floor plan with highlighted frames

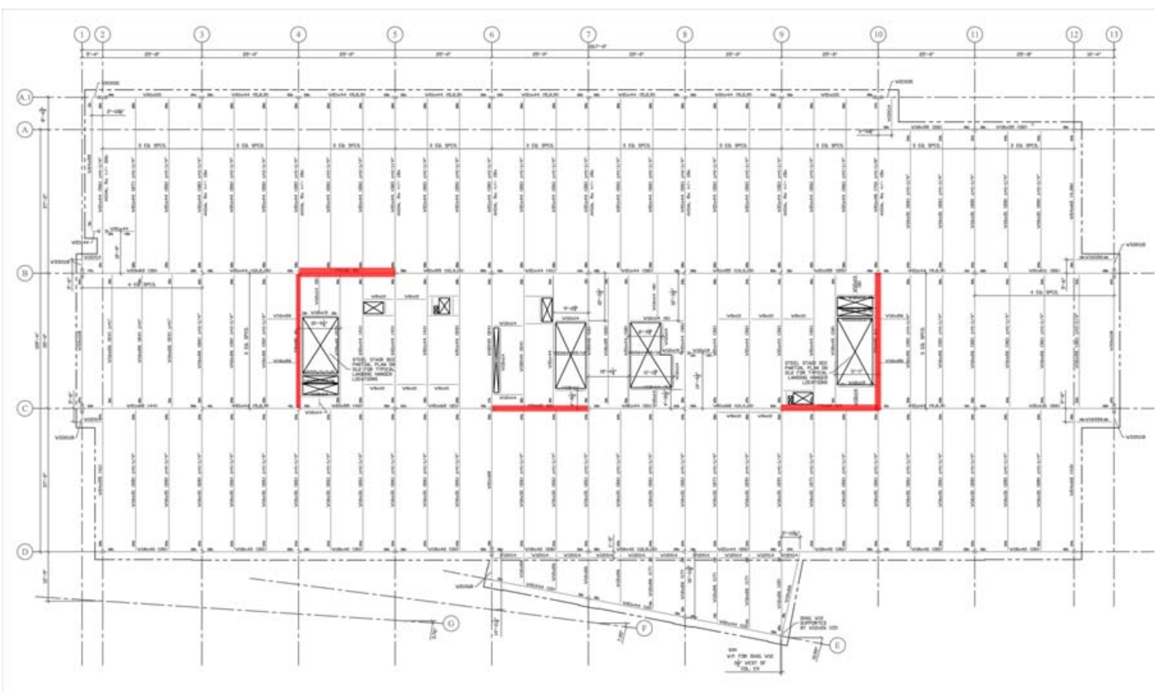


Fig F.4 – 5th Floor plan with highlighted frames

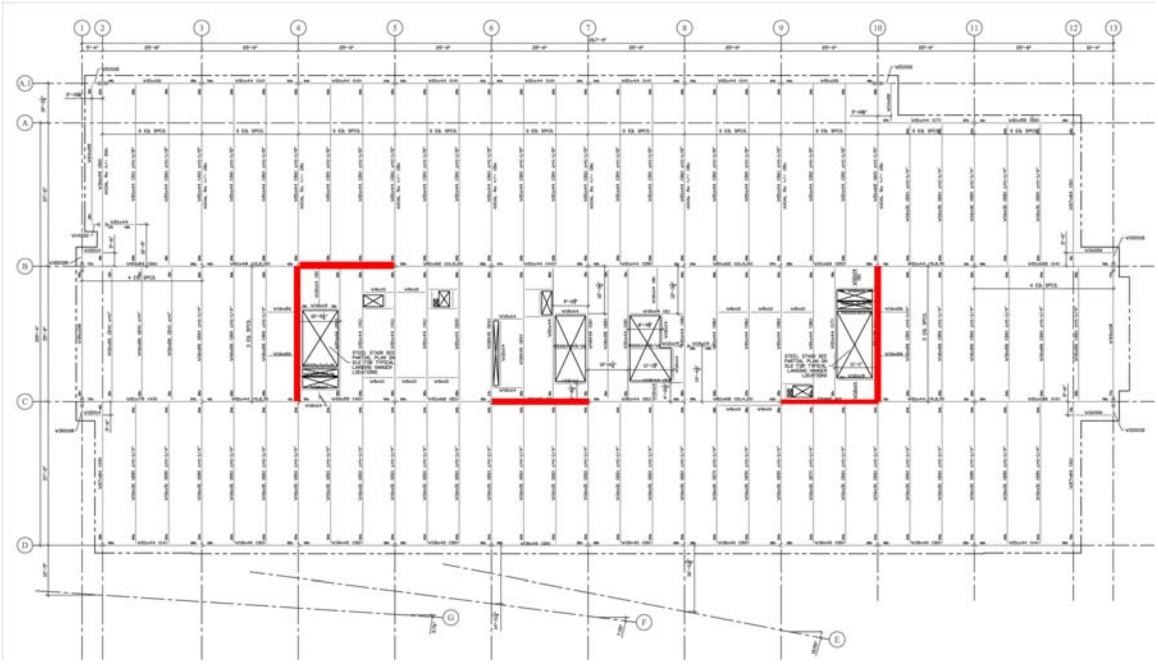


Fig F.5 – 6th Floor plan with highlighted frames

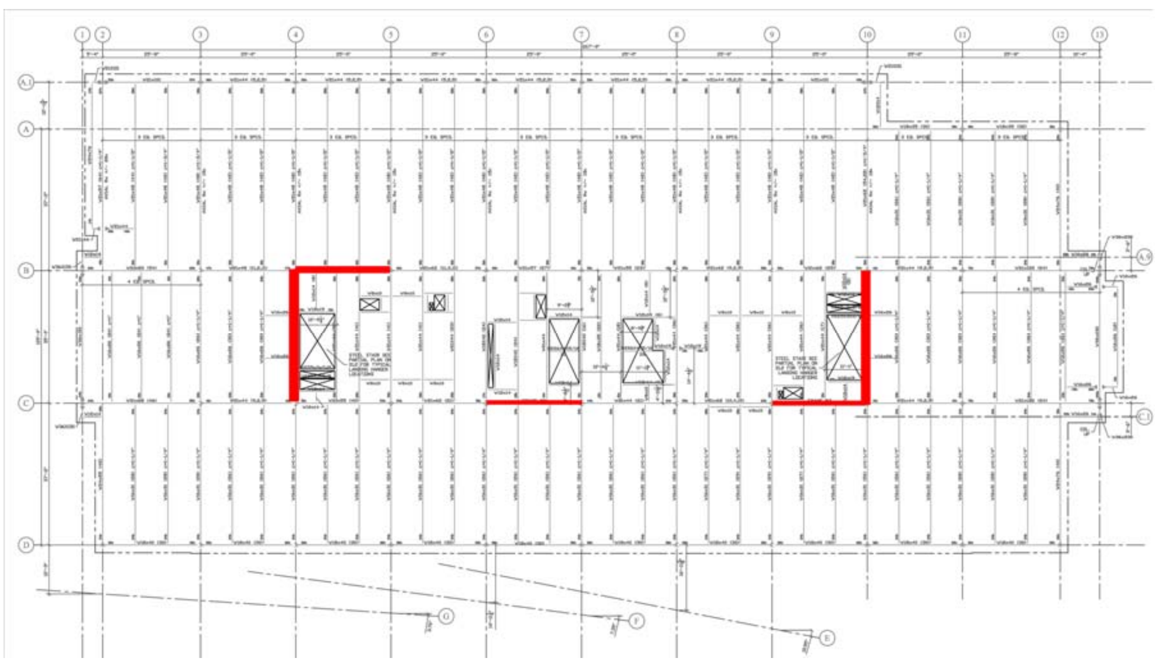


Fig F.6 – 7th floor plan with highlighted frames

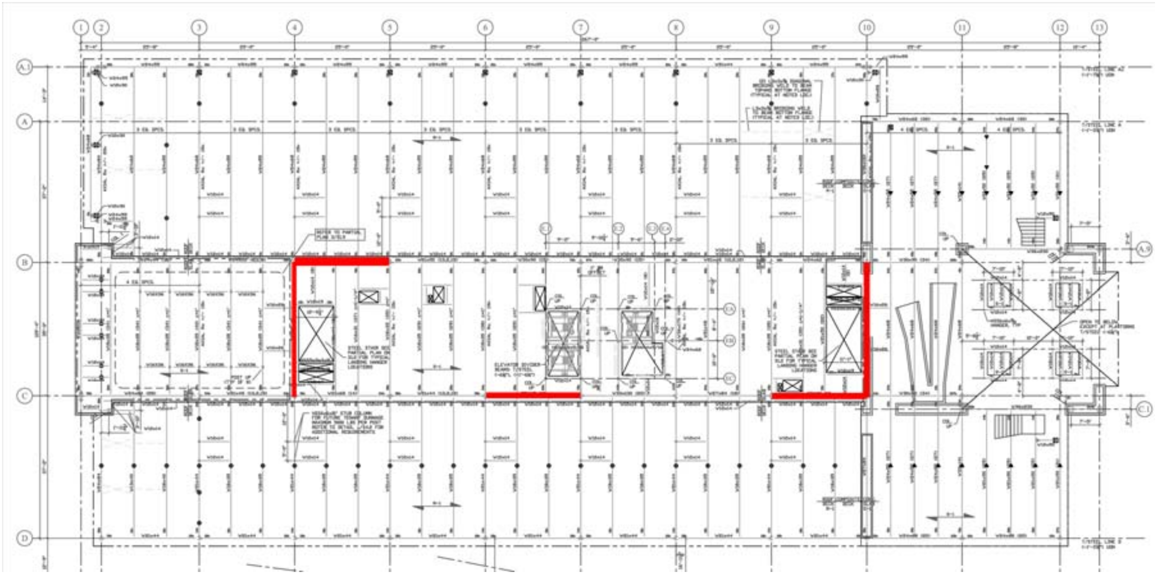


Fig F.7 – Roof plan with highlighted frames

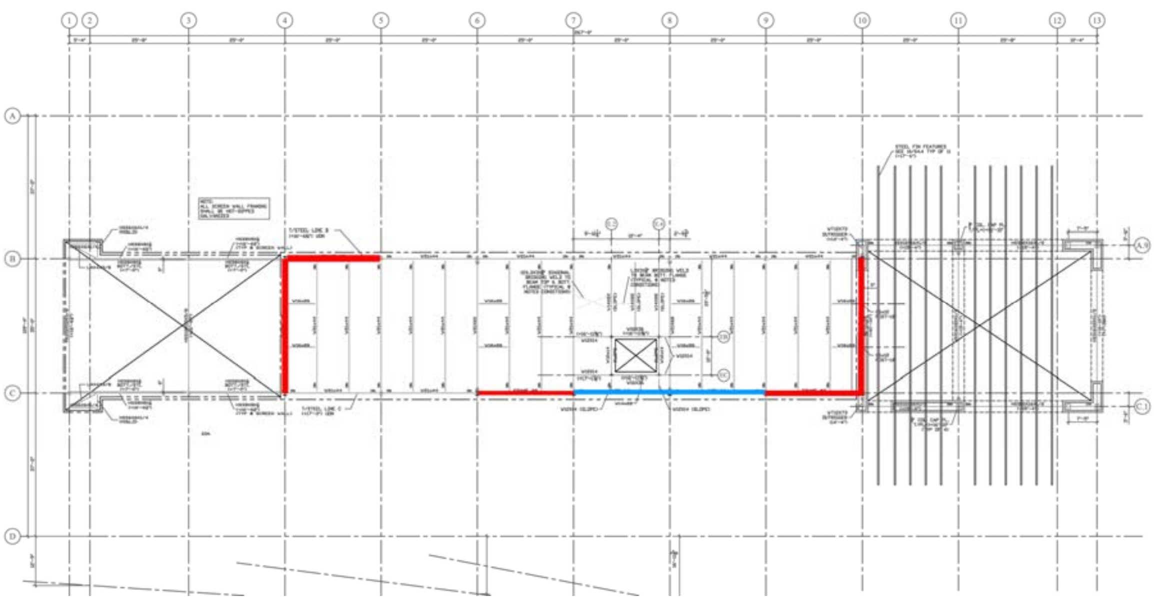


Fig F.8 – Penthouse Roof plan with highlighted frames

Existing Structure Gravity Loads

Live Loads – IBC Table 1607.1	
Roof Garden	100 PSF
Offices	70 PSF
Corridors	80 PSF
Stair and Exits	100 PSF
Lobbies and First Floor Corridors	100 PSF

Table F.1 – Live Loads

The value of live load for offices includes a 20 PSF addition for partitions. To be consistent with the original design a value of 100 PSF will be used as the live load on a typical floor.

Snow Load Chapter 7 ASCE7-05	
Pg	30 PSF
C_e	0.9
C_t	1.0
I	1.0
$P_{f,\ min}$	20 PSF
$P_{f,\ Calculated}$	18.9 PSF
P_f	20 PSF

Table F.2 – Roof Snow Load

The roof live load will be taken to be equal to the calculated snow load of 20 psf.

Dead Loads		
Typical Floor		
Composite Floor System	41 PSF	Estimated Using United Steel Deck Catalog
Misc. (Self wt., finishes, etc.)	10 PSF	Estimated Using AISC Manual of Steel Constr.
Ponding of Concrete	10 PSF	
Roof		
Deck	2 PSF	Estimated Using United Steel Deck Catalog
Insulation	3 PSF	Estimated using AISC Manual of Steel Constr.
Roofing	20 PSF	
Curtain Wall		
Glass Curtain Wall	.215 KLF	From Building Specifications
Pre-cast Assembly	.55 KLF	From Building Specifications
Roof Garden		
	160 PSF	From Materials in Specifications

Table F.3 – Dead Loads

Existing Structure Lateral Loads

Wind
(See Appendix for complete spreadsheet of wind calculation)

Total Worst Case Wind Load Each Direction	
z (ft)	P (psf)
0-15	12.503
20	13.140
25	13.650
30	14.160
40	14.924
50	15.562
60	16.071
70	16.581
80	17.091
90	17.473
100	17.728
115.17	18.212

Table F.4 – Wind Load

Seismic
(See Appendix for complete spreadsheet of seismic calculation)

Seismic Force Distribution						
Floor	w_x	h_x	k	$w_x h_x^k$	$\Sigma\, w_i h_i^k$	C_{vx}
Base	--	--	--	--	--	--
2	2561.24	15.00	1.00	38418.56	1030201.93	0.037
3	2692.77	28.33	1.00	76295.25	1030201.93	0.074
4	2563.19	41.67	1.00	106799.39	1030201.93	0.104
5	2570.64	55.00	1.00	141385.17	1030201.93	0.137
6	2536.08	68.33	1.00	173298.77	1030201.93	0.168
7	2645.26	81.67	1.00	216029.31	1030201.93	0.210
Roof	2638.54	96.67	1.00	255058.81	1030201.93	0.248
Penthouse Roof	198.98	115.17	1.00	22916.67	1030201.93	0.022
						1.000

Floor	F_x (Kips)
Base	770.19
2	28.72
3	57.04
4	79.84
5	105.70
6	129.56
7	161.50
Roof	190.68
Penthouse Roof	17.13
	770.19

Table F.5 – Seismic Floor Shear Distribution

Statement of Problem

Analysis of the current composite steel structural system showed that it is an efficient system for the applied loading. However the location of Parkridge 6 is in an area were concrete construction is primarily used. The bay sizes of the current building fit the profile for either a post-tensioned slab and beam system or a post-tensioned two-way slab system.

It was determined in a previous investigation that for the bay sizes a post-tensioned slab and beam system would be the next best alternative. The primary reason that a post-tensioned two-way slab system was not selected is the special loading conditions from the sloping columns on the south face which would.

Design Approach

To redesign Parkridge Center – Phase VI as a concrete structure the provisions in ACI 318-05 chapter 18 were followed using an excel spreadsheet. To aid in the design RISA-3D was used to determine maximum loading on each member and perform lateral load analysis. The concrete design of Parkridge 6 will include:

- Post-Tensioned Slab and Beam Design
- Concrete Moment Frame Design
- Concrete Shear Wall Design
- Concrete Columns

Assumptions

Parkridge Center – Phase VI is a commercial office building offering individual leases for each floor. The design will be geared towards maximizing rentable space. The critical assumptions or the concrete design are:

- **Maximum Bay Spacing** – the current bay spacing creates the maximum floor area while preserving the architects design intent. As such the concrete system will be designed to use the current bay spacing.
- **Constant Building Height** – the currently design floor heights will be used to control the depths of the proposed concrete members as the current design sits at the overall height limitation for the area.

Depth Analysis – Post-Tensioned Slab and Beam Floor System

The alternative system that was selected for investigation in this study is a post-tensioned slab and beam system. This system was selected primarily because it was found to be the next best alternative in a previous study done in the fall 2006 semester. It was also chosen because I wanted to extend my knowledge of concrete design.

Proposed Floor Plans

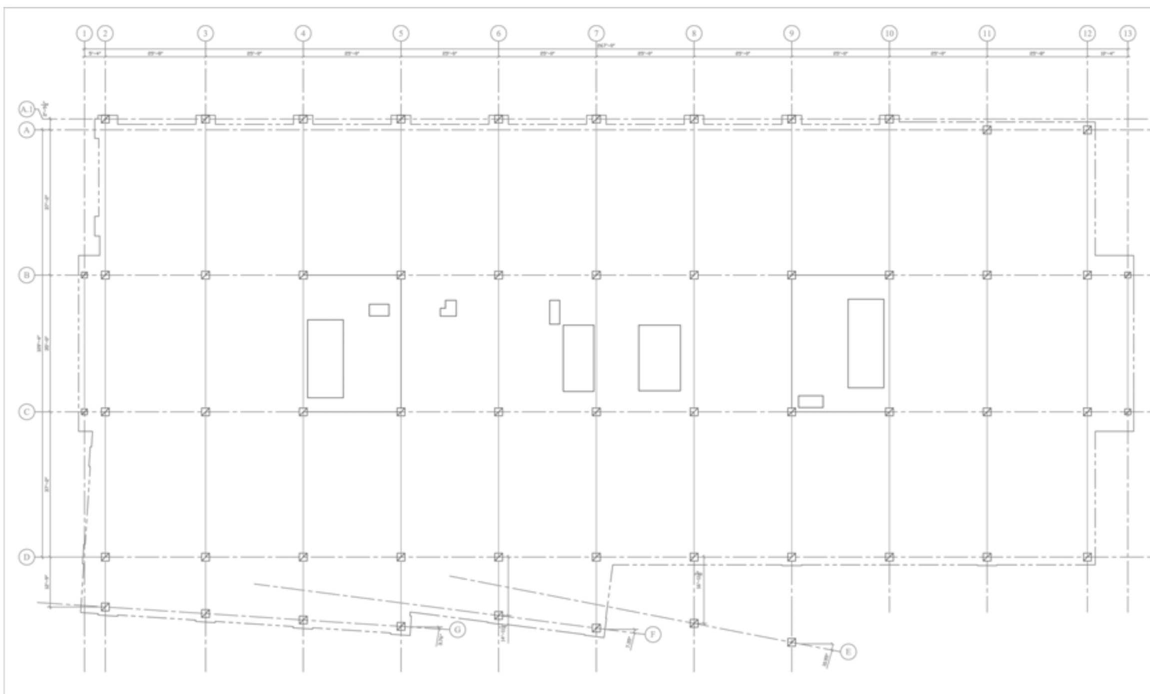


Fig. F.9 – 2nd Floor plan – concrete

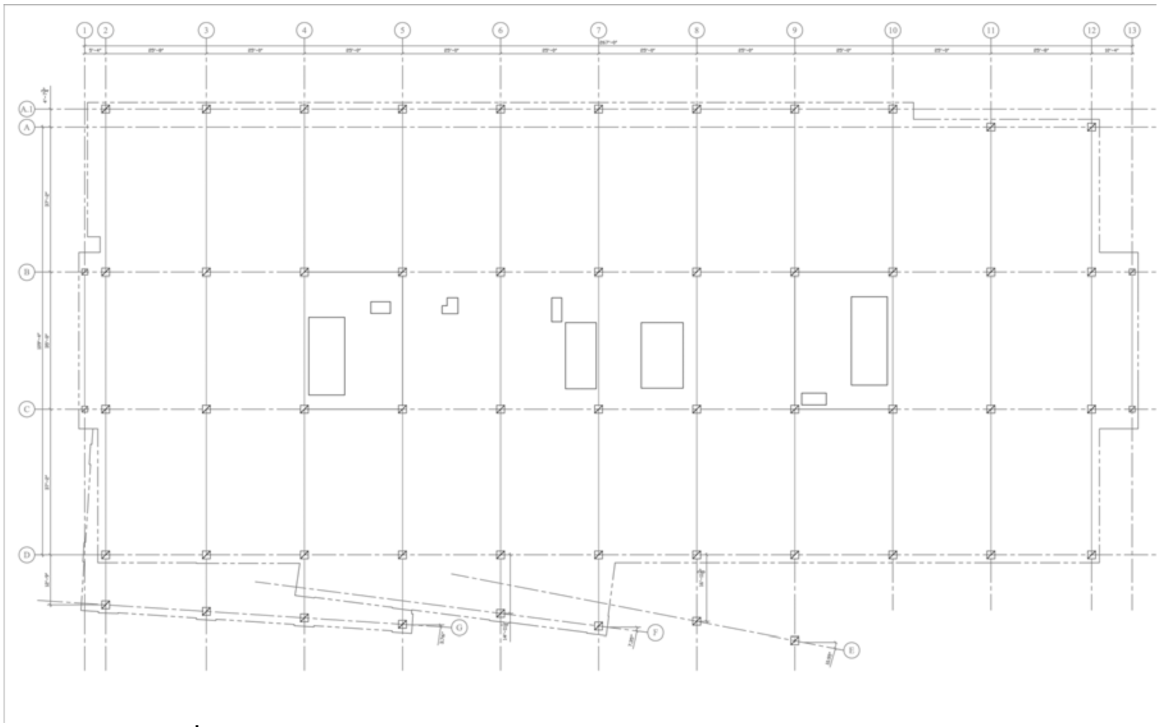


Fig F.10 – 3rd Floor Plan – Concrete

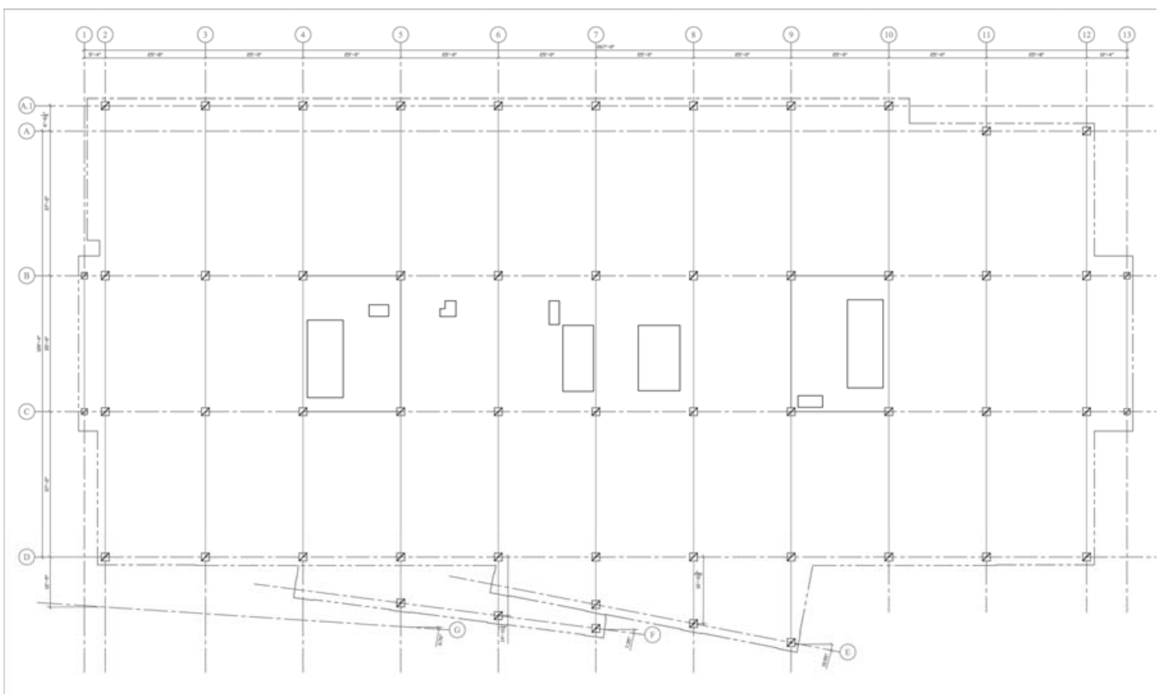


Fig F.11 – 4th Floor Plan – Concrete

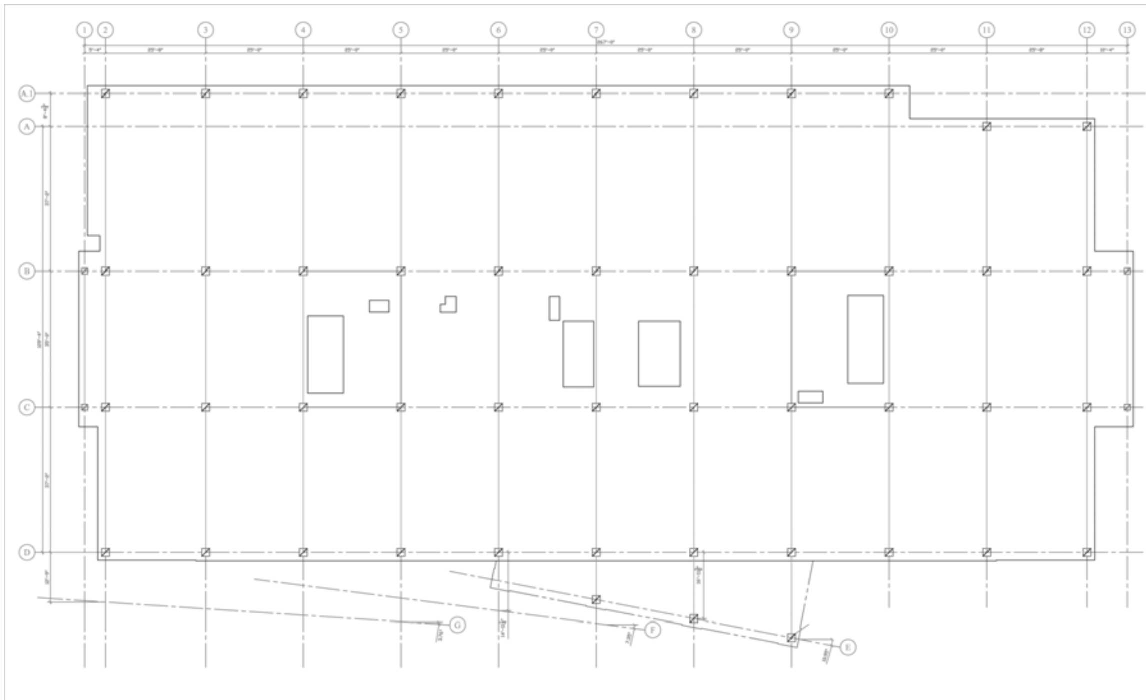


Fig F.12 – 5th Floor Plan – Concrete

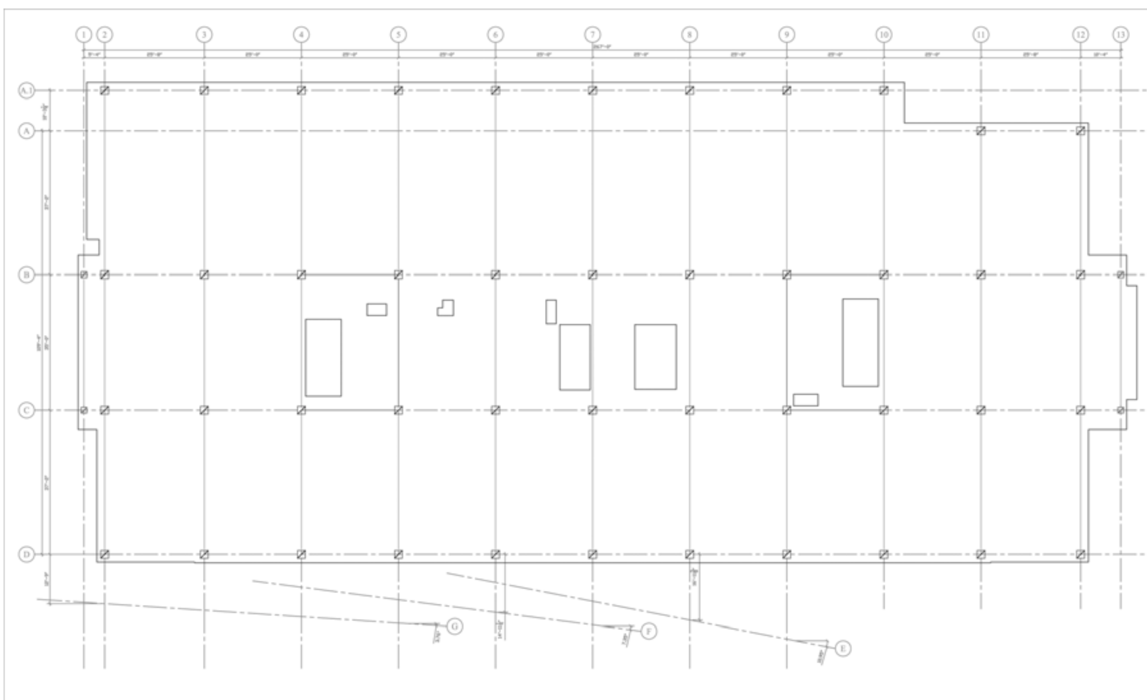


Fig F.13 – 6th Floor Plan – Concrete

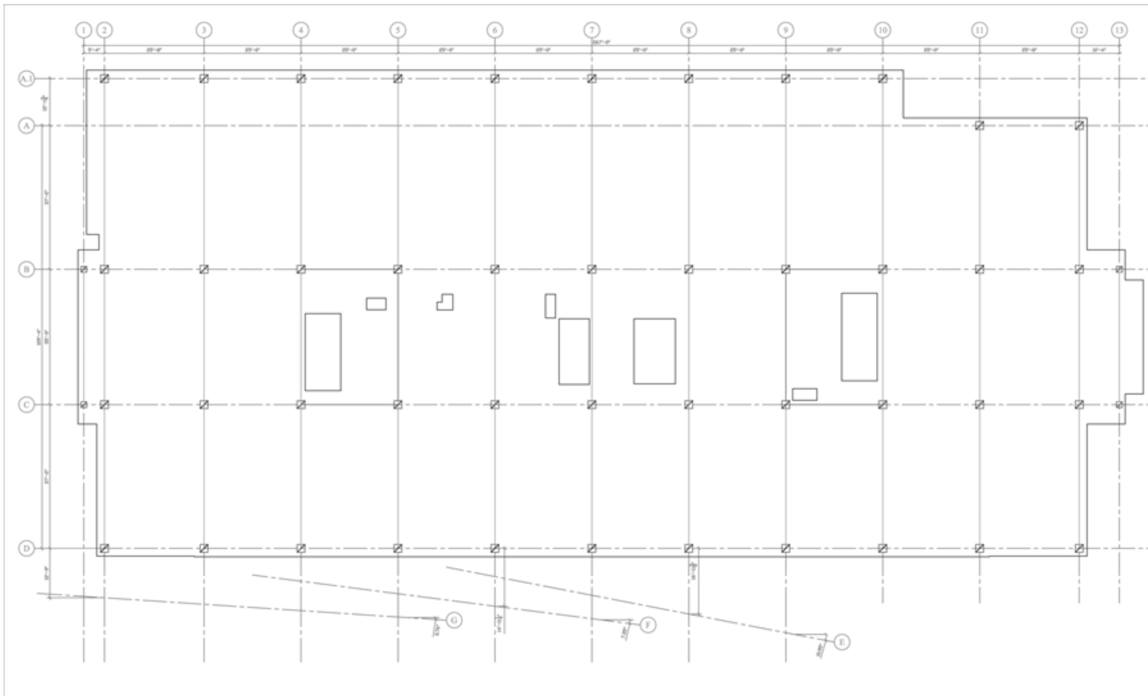


Fig F.14 – 7th Floor Plan – Concrete

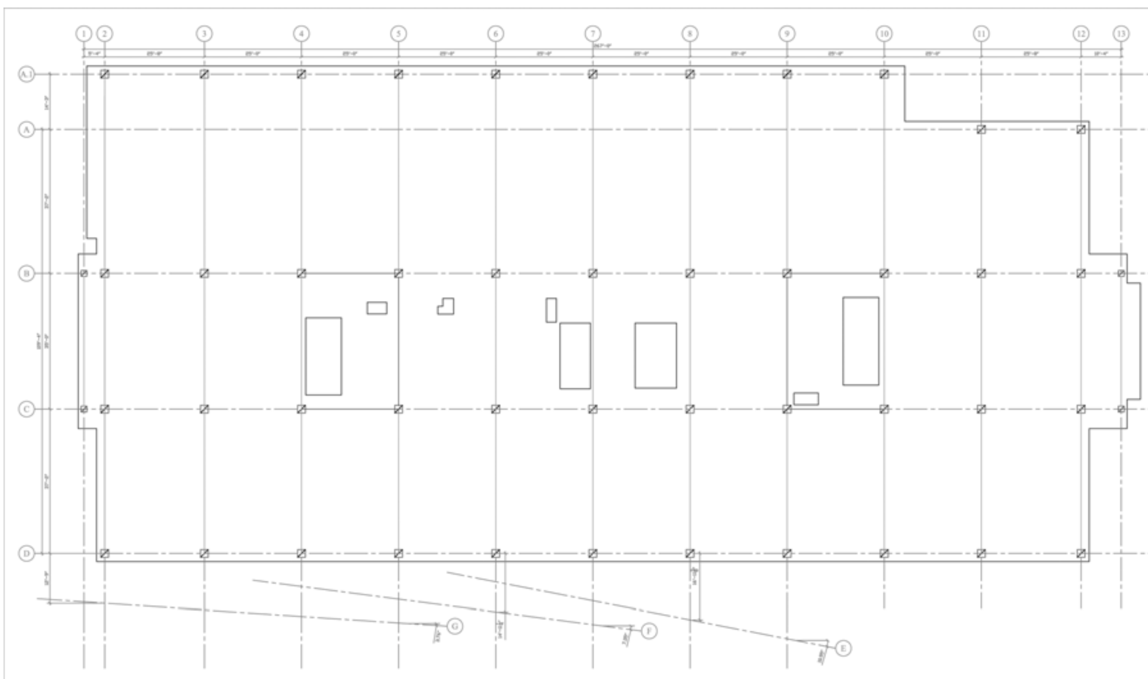


Fig F.15 – Roof Plan - Concrete

Post-Tensioned Slab

The slab spans in the east-west direction in which the bays are 25'-8" on the exterior bays and 25'-0" on the interior bays. For the calculation of shears and moments the slab was treated as being pinned to each of the supporting beams, yielding a conservative value for both moments and shears.

The slab depth was determined first by Ln/45 to meet deflection requirements. This slab depth was then checked with the applied loading and post-tensioned force to fall within the requirements of uncracked behavior under service conditions. The load balancing method was used to determine the post-tensioning force and tendon layout. The assumed strength of the concrete for the design of the slab was F'c=5000 psi.

Dimensions		
Thickness	7.00	in
Cover	1.25	in
Tendon Φ	0.50	in
a	4.00	in

Table F.6 – PT Slab Dimensions

Gravity Loading

Loads		
slab	87.50	psf
DL	20.00	psf
LL	100.00	psf
TL	207.50	psf
$w_{prestress}$	96.75	psf
w_{net}	110.75	psf

Table F.7 – Slab Loads

Interior span

Interior Span				
L	25.00	ft		
Mp	7558.59	ft-lbs		
F	22675.78	lbs/ft		
F/A	269.95	psi		
C.L. Mn	6292.61	ft-lbs		
M^f	6292.61	ft-lbs		
S	98	in^3		
				Class
f^+	500.57	psi	Tension	U
f^-	-1040.47	psi	Compression	U or T

PT Strands		
Fpu	270000	psi
Fpi	189000	psi
Pi	28.34	kips
Ap	0.15	in^2
# Strands	1	
Diameter	0.5	in
Apa	0.20	in^2

Table F.8 – Interior Span Slab PT

Exterior Span

Exterior Span				
L	25.67	ft		
Mp	13980.38	ft-lbs		
F	41941.13	lbs/ft		
F/A	499.30	psi		
C.L. Mn	7297.86	ft-lbs		
M^f	7297.86	ft-lbs		
S	98	in^3		
				Class
f^+	394.32	psi	Tension	U
f^-	-1392.91	psi	Compression	U or T

PT Strands		
Fpu	270000	psi
Fpi	189000	psi
Pi	52.43	kips
Ap	0.28	in^2
# Strands	2	
Diameter	0.5	in
Apa	0.39	in^2

Table F.9 – Exterior Span Slab PT

The preceding tables F.7 and F.8 detail the design of the interior and exterior spans respectively. The cells shaded yellow are user inputted values while the cells shaded green are calculated within the spreadsheet. Mp is calculated using w_{net} from table F.6 using the formula $WL^2/8$. The value F is the jacking force on the post-tension strand per foot of length of slab.

Post-Tensioned Slab Design Summary

The proposed post-tensioned slab was designed to be 7" thick with two strands per foot in the exterior bay and one strand per foot in the interior bays as illustrated in Fig F.9. The exterior bay strands will be jacked at 52.43 Kips/ft of slab while the interior bays will be jacked at 28.34 Kips/ft of slab.

The slab is also designed as an uncracked section and was proven to act uncracked based on the requirements of ACI 318-05.

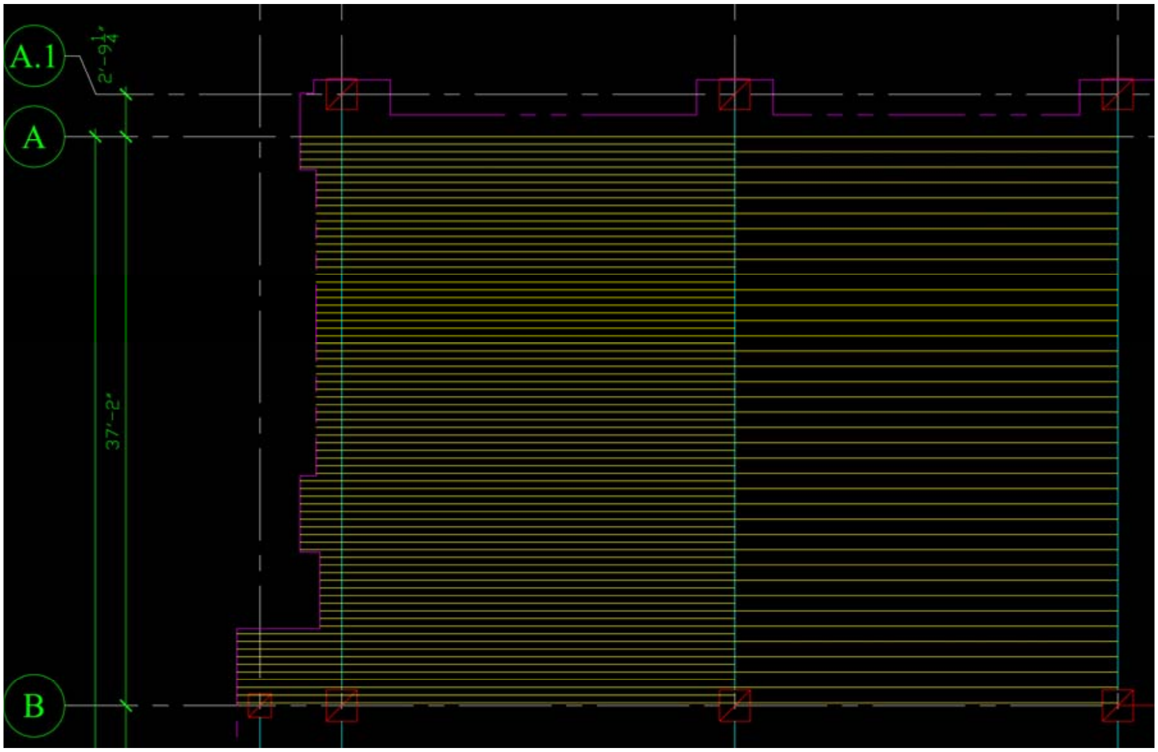


Fig F.16 – PT Slab Tendon Distribution Typ. Exterior and Interior bay

Post-Tensioned Beams

The post-tensioned beams were designed using the requirements of ACI 318-05 chapter 18. All of the post-tensioned beams were designed to be uncracked under service loads. For the analysis of the applied loads on the post Tensioned beams RISA-3D was used to create 2-dimensional frame models of representative bays. The models were then loaded with dead loads applied to all spans and live load applied in different patterns to determine to worst case moments and shears. For detailed spreadsheets used in the design of the post-tensioned beams refer to the appendix.

The flexural design of the post-tensioned beams was done using ACI 318-05 using LRFD. The beams were also treated as t-sections utilizing the slab as a flange for flexural calculations. For torsion and shear the beams were treated as rectangular sections.

The applied loads on the beams are the same as in table F.7 with the addition of the beam self weight.

Loads (Unfactored)		
Slab	87.50	psf
SW	875.00	plf
DL	20.00	psf
LL	100.00	psf
trib width	25	ft
Slab	2187.5	plf
SW	875.00	plf
DL	500	plf
LL	2500	plf
TL	6062.50	plf
w_p	3206.25	plf
w_n	2856.25	plf

Table F.10 – Typical Beam Loading

Exterior or Edge Beams

The worst case exterior beam was analyzed and designed for flexure, torsion, and shear. The resulting beam was a 28in x 38in cross section with (26) post-tensioned strands. The torsion analysis yielded #6 bars spaced as shown in table F.11.

Dist. from Column Face (ft)	s	
d	1.12	in
4.53	1.17	in
6.53	1.23	in
8.53	1.29	in
10.53	1.36	in
12.53	1.44	in
14.53	1.52	in
16.53	1.62	in
18.53	1.73	in
20.53	1.86	in
22.53	2.01	in

Table F.11 – Exterior Beam Torsion Reinf. Spacing

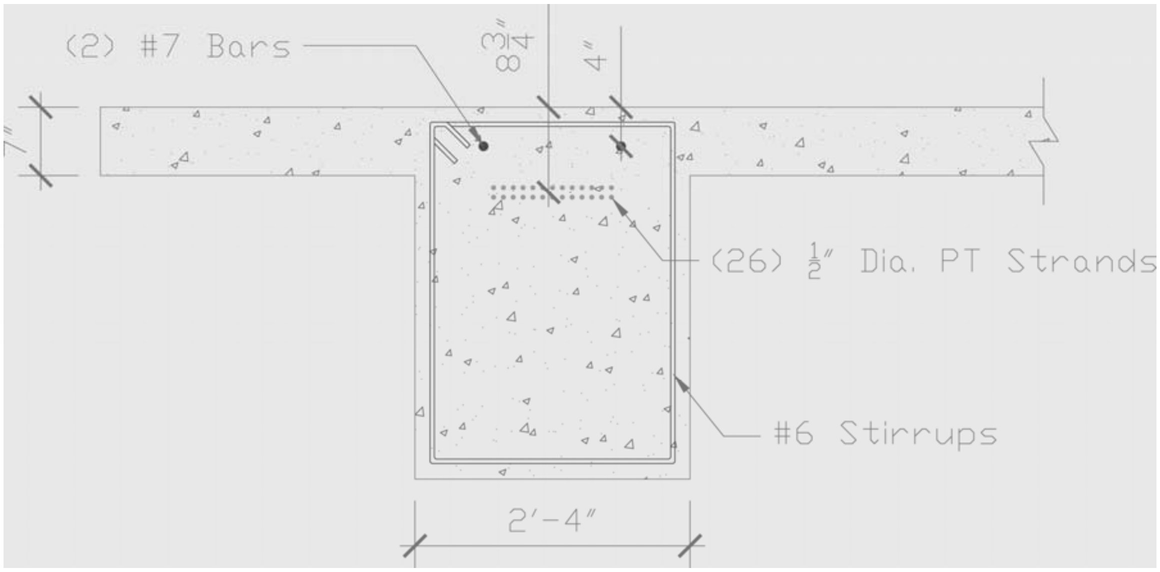


Fig. F.17 – Exterior Beam Section at Distance d from Support

It was determined through flexural analysis that (2) #7 bars were needed at the supports for this exterior beam to meet minimum area of steel (As) requirements from ACI 318-05. This beam however met all flexural strength requirements with the PT strands alone.

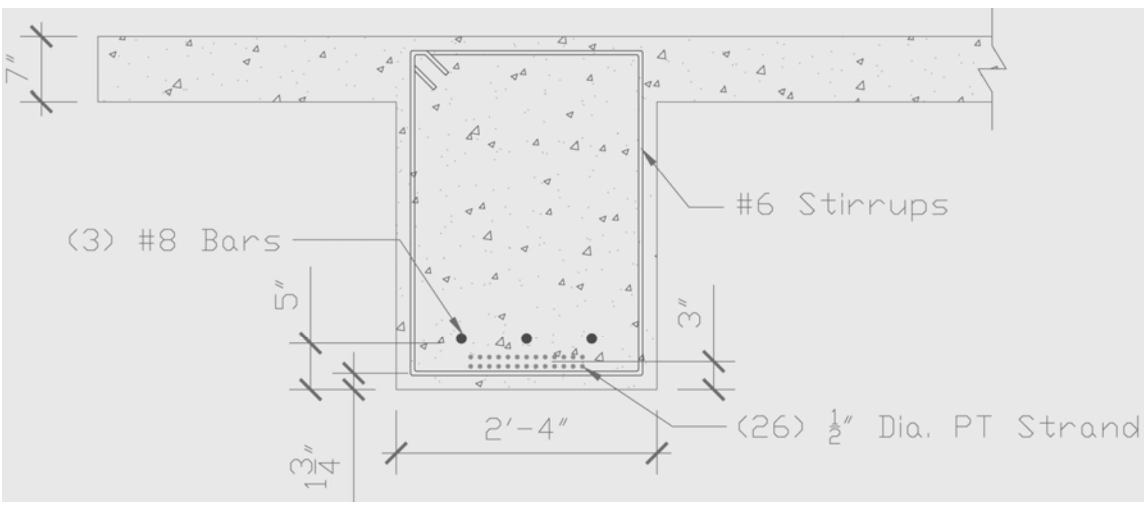


Fig. F.18 – Exterior Beam Section at Mid span

At mid span (3) #8 bars were needed to meet ACI 318-05 minimum required As. The (26) post-tensioned cables are distributed in a parabolic shape along the beams length placing the depth from the top of the slab at a minimum at the supports and a maximum at the mid span of the beam. The post-tension tendon profile follows the moment diagram of the beam.

First Interior Beams

The next critical beam to design is the first interior beam as it receives load from the longer exterior bay, 25'-8", and the minimally shorter interior bay, 25'-0". The resulting loading is a combination of torsion, flexure, and shear. Also from column lines 2-10 there is an applied axial load from the sloping columns. The results of the designs were floors 2-6 were 28in x 30in cross sections and floors 7 and Roof were 28in x 34in cross sections. The following figures detail the first interior beam on the 6th floor.

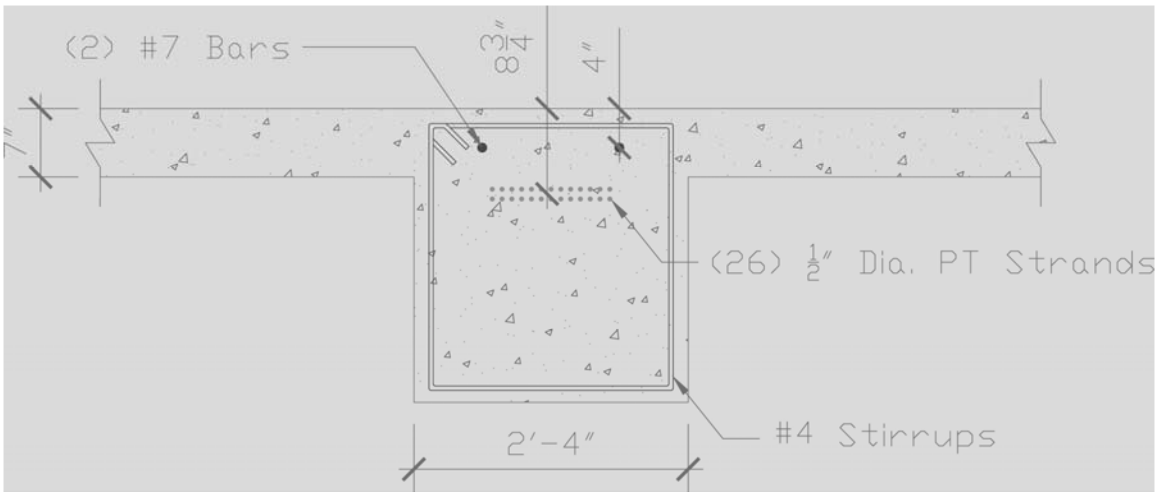


Fig. F.19 First Interior Beam section at support

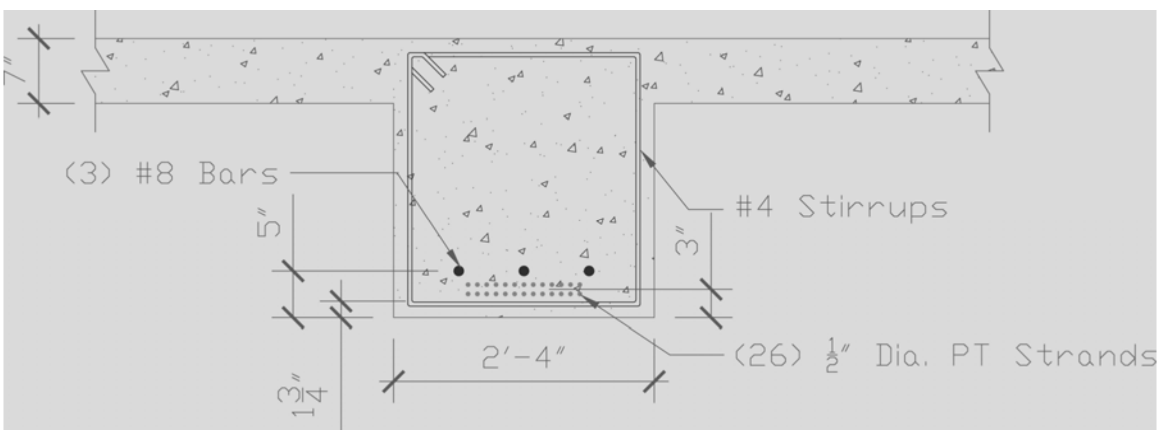


Fig. F.20 First Interior Beam section at mid span

The minimum As requirement of ACI-318 was met by adding (2) #7 bars at the supports and (3) #8 bars. Torsional and shear reinforcement consist of #4's spaced as indicated in the following table F.12.

Dist. from Column Face (ft)	s	
d	2.14	in
4.00	2.24	in
6.00	2.34	in
8.00	2.46	in
10.00	2.59	in
12.00	2.74	in
14.00	2.90	in
16.00	3.09	in
18.00	3.30	in
20.00	3.54	in
22.00	3.82	in

Table F.12 – First Interior Beam Torsion and Shear Reinf.

Non-Post-Tensioned Beams

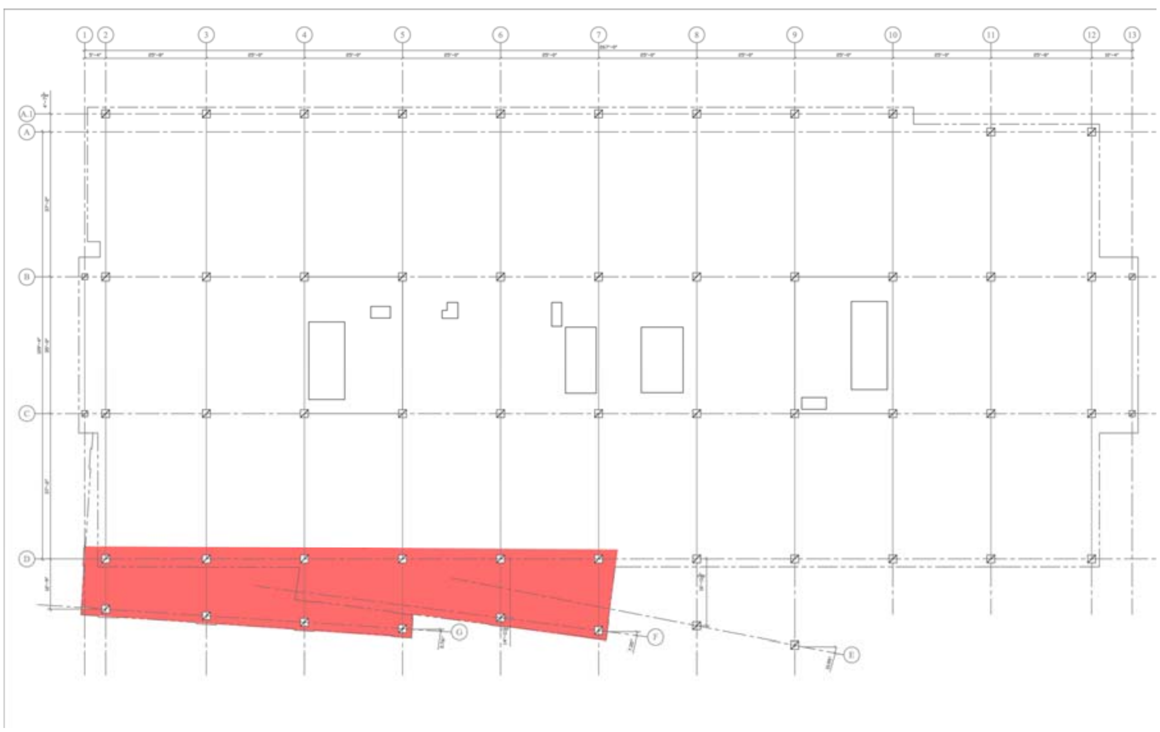


Fig. F.21 – Location of Non-PT Beams

Figure F.21 illustrates the locations of beams to be designed as reinforced concrete with no post-tensioning. The primary reason these bays were designed as reinforced sections and not post-tensioned is because their small spans make it more economical. They were also designed this way to keep standard size forms for the beams on each floor. The non-pt beams were designed to be 18in x 30in cross sections. The detailed design spreadsheet for the non-pt beams can be found in the appendix.

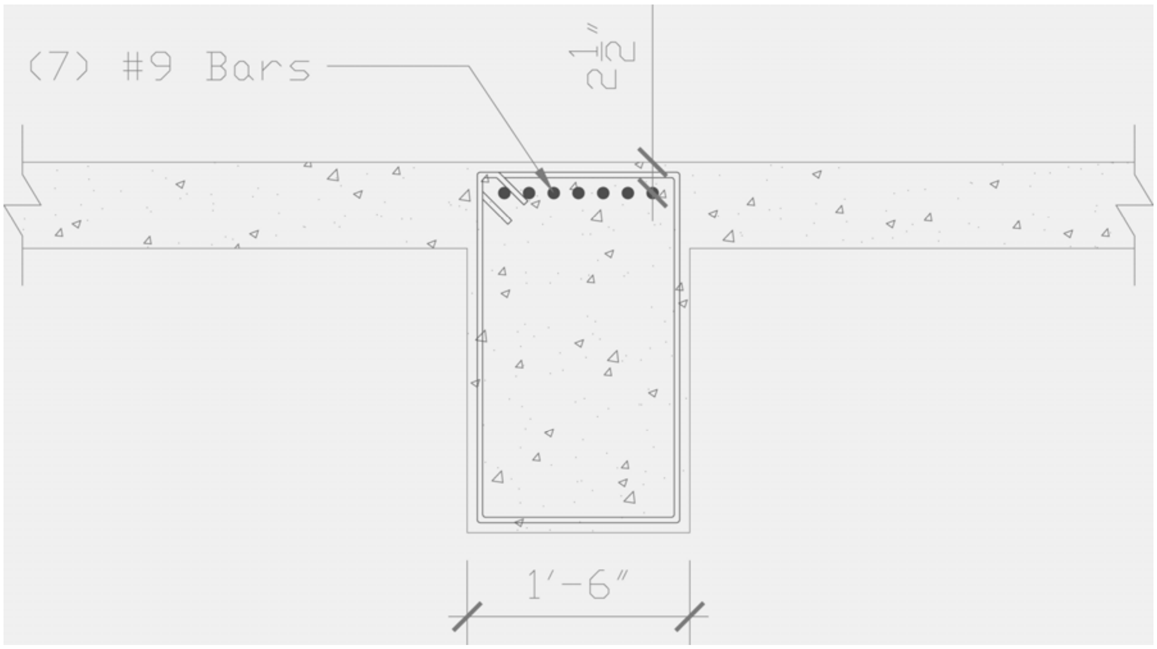


Fig. F.22 – Non_PT Beam at Support

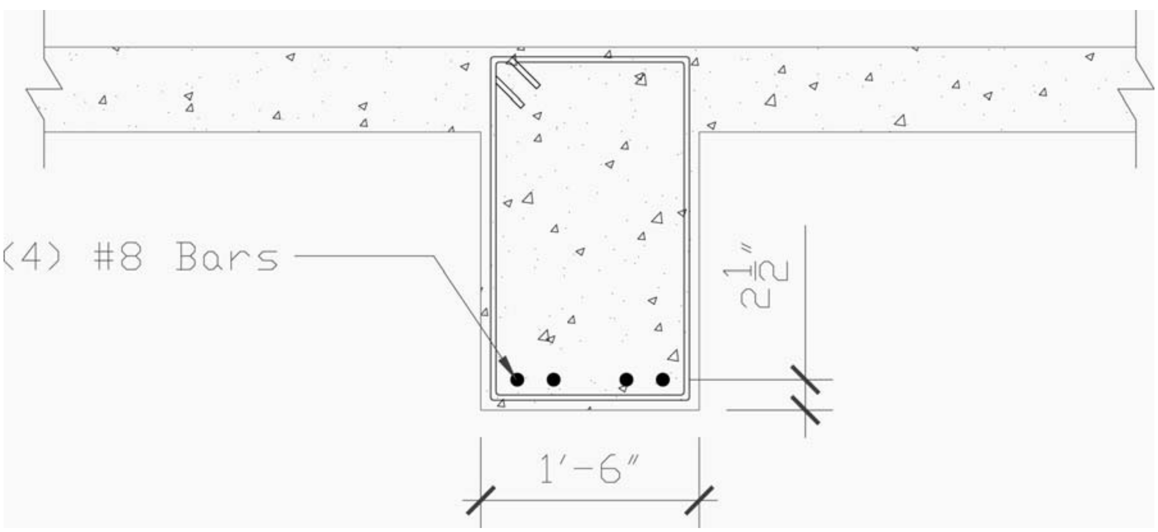


Fig. F.23 – Non_PT Beam at Mid Span

Lateral System – Concrete Moment Frames

Concrete moment frames were checked as a first alternative lateral load resisting system as they are inherently part of concrete cast-in-place construction. The frames will resist loads in the N-S direction while concrete shear walls will resist the load in the E-W direction. The controlling lateral load was determined to be seismic loading. Each frame was modeled in RISA 3D and had a 1 kip lateral load applied to the top the resulting deflection was then used to calculate the relative stiffness of each frame.

From the relative stiffness the center of gravity and center of rigidity were calculated on each floor. The resulting eccentricity of the center of rigidity from the center of gravity yielded significant increase to the applied lateral load due to the floor torsion effects. The following tables show the loading before applied torsion effects and after the torsion effects.

Seismic Force Distribution						
Floor	w_x	h_x	k	$w_x h_x^k$	$\Sigma\ w_i h_i^k$	C_{vx}
Base	--	--	--	--	--	--
2	4971.48	15.00	1.00	74572.25	2272881.91	0.033
3	5157.68	28.33	1.00	146134.38	2272881.91	0.064
4	5043.15	41.67	1.00	210131.14	2272881.91	0.092
5	6078.80	55.00	1.00	334333.91	2272881.91	0.147
6	4909.53	68.33	1.00	335484.57	2272881.91	0.148
7	5241.03	81.67	1.00	428017.75	2272881.91	0.188
Roof	7461.63	96.67	1.00	721291.24	2272881.91	0.317
Penthouse Roof	198.98	115.17	1.00	22916.67	2272881.91	0.010
						1.000

Floor	F_x (Kips)
Base	2699.47
2	88.57
3	173.56
4	249.57
5	397.08
6	398.45
7	508.35
Roof	856.67
Penthouse Roof	27.22
	2699.47

Table F.13 – Seismic Base Shear and Floor Distribution

Seismic - Story Shear																	
E - W																	
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	Frame 7	Frame 8	Frame 9	Frame 10	Frame 11	Frame 12	Frame 13	Frame 14	Frame 15	Shear (K)
2	88.57												22.14	22.14	22.14	22.14	88.57
3	173.56												43.39	43.39	43.39	43.39	173.56
4	249.57												62.39	62.39	62.39	62.39	249.57
5	397.08												99.27	99.27	99.27	99.27	397.08
6	398.45												99.61	99.61	99.61	99.61	398.45
7	508.35												127.09	127.09	127.09	127.09	508.35
Roof	856.67												214.17	214.17	214.17	214.17	856.67
N - S																	
2	88.57	9.17	9.17	9.17	9.17	9.17	9.17	9.17	9.17	9.17	3.00	3.00					88.57
3	173.56	17.98	17.98	17.98	17.98	17.98	17.98	17.98	17.98	17.98	5.89	5.89					173.56
4	249.57	25.85	25.85	25.85	25.85	25.85	25.85	25.85	25.85	25.85	8.47	8.47					249.57
5	397.08	41.13	41.13	41.13	41.13	41.13	41.13	41.13	41.13	41.13	13.47	13.47					397.08
6	398.45	41.27	41.27	41.27	41.27	41.27	41.27	41.27	41.27	41.27	13.52	13.52					398.45
7	508.35	52.65	52.65	52.65	52.65	52.65	52.65	52.65	52.65	52.65	17.25	17.25					508.35
Roof	856.67	88.73	88.73	88.73	88.73	88.73	88.73	88.73	88.73	88.73	29.07	29.07					856.67

Seismic - Story Shear (With Torsional Effects)																	
E - W																	
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	Frame 7	Frame 8	Frame 9	Frame 10	Frame 11	Frame 12	Frame 13	Frame 14	Frame 15	Shear (K)
2	88.57					Worst Case Loading							39.43	22.14	22.14	22.14	105.86
3	173.56												78.99	43.39	43.39	43.39	209.16
4	249.57												116.57	62.39	62.39	62.39	303.74
5	397.08												193.72	99.27	99.27	99.27	491.53
6	398.45												184.00	99.61	99.61	99.61	482.84
7	508.35												237.26	127.09	127.09	127.09	618.53
Roof	856.67												446.74	214.17	214.17	214.17	1089.24
N - S																	
2	88.57	10.31	10.66	11.29	12.83	22.67	9.17	9.17	9.17	9.17	3.00	3.00	Worst Case Loading				110.47
3	173.56	20.32	21.04	22.33	25.50	45.78	17.98	17.98	17.98	17.98	5.89	5.89					218.66
4	249.57	29.42	30.51	32.47	37.30	68.16	25.85	25.85	25.85	25.85	8.47	8.47					318.19
5	397.08	47.35	49.25	52.68	61.10	114.89	41.13	41.13	41.13	41.13	13.47	13.47					516.72
6	398.45	46.83	48.53	51.59	59.11	107.17	41.27	41.27	41.27	41.27	13.52	13.52					505.35
7	508.35	59.92	62.13	66.12	75.95	138.69	52.65	52.65	52.65	52.65	17.25	17.25					647.91
Roof	856.67	104.06	108.73	117.17	137.91	270.36	88.73	88.73	88.73	88.73	29.07	29.07					1151.27

Table F.14 – Moment Frame Seismic Loads

The additional lateral load from torsional effects produced excessive deflections that would have produced columns with dimensions 24in x 64in. In order to maintain the open floor plan outlined in my objectives another alternative lateral system was selected.

	Deflection	
Node	X (in)	Y (in)
N1	13.821	-0.125
N2	13.807	-0.508
N3	13.799	-0.667
N4	13.791	-2.339

Table F.15 – Moment Frame Node Deflections

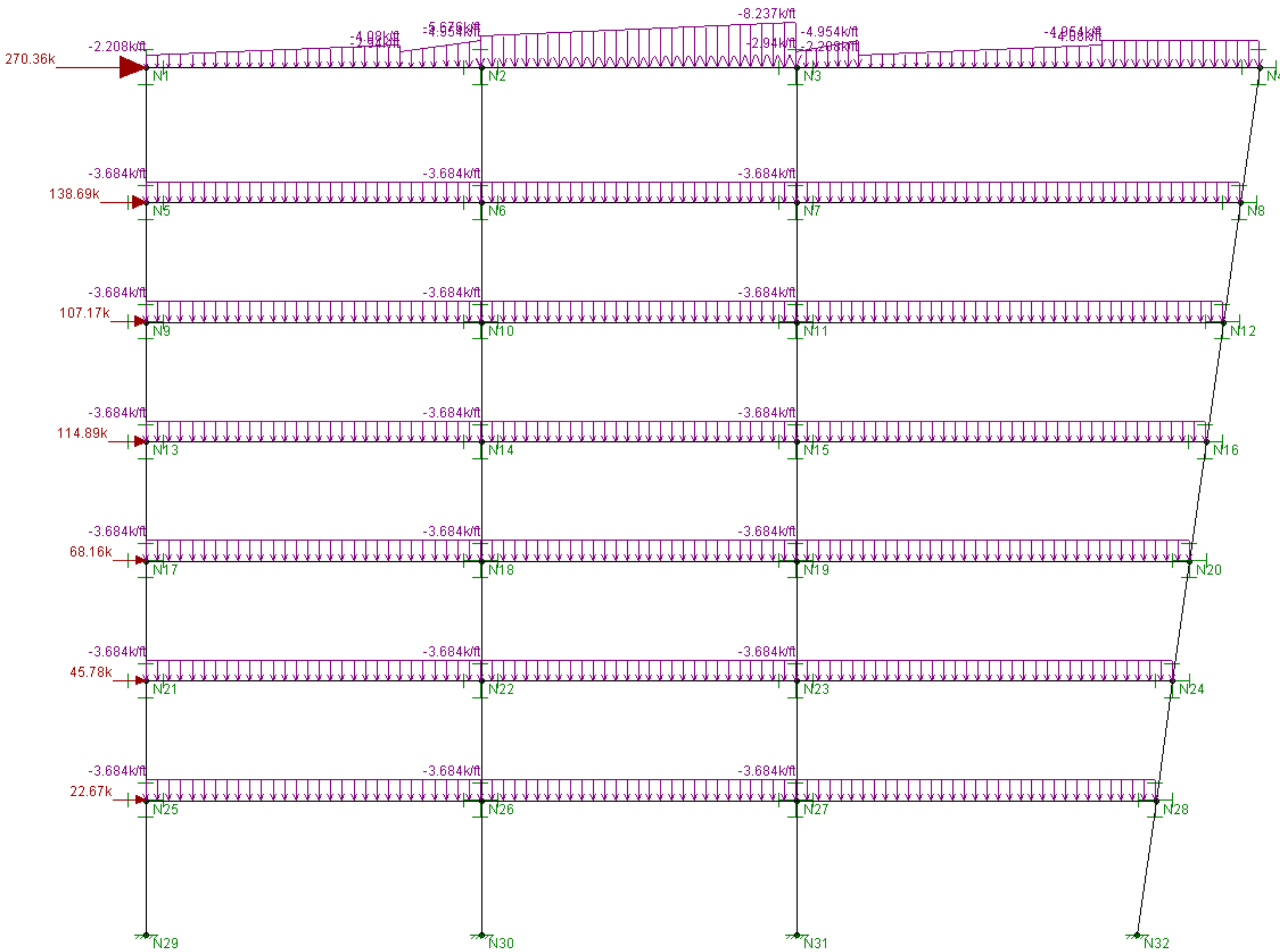


Fig. F.24 – Moment Frames With Applied Seismic Load

Lateral System - Concrete Shear Walls

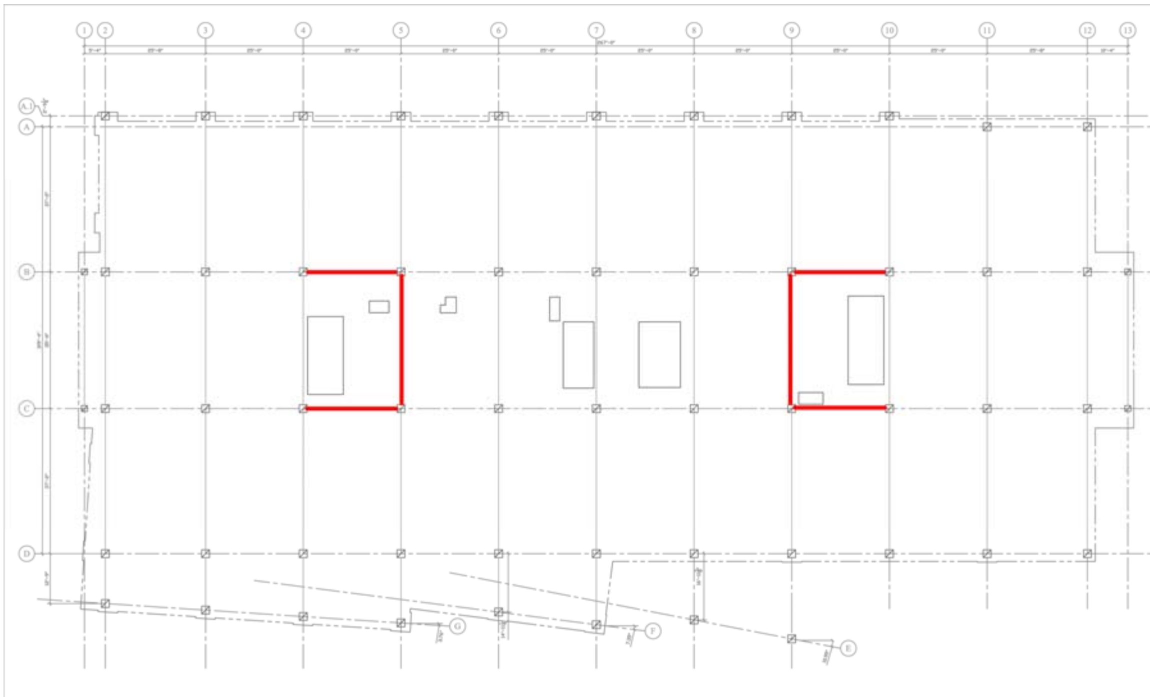


Fig. F.25 – 2nd Floor Plan with Concrete Shear Walls Highlighted

Due to the severe increase in load from torsional affects on the moment frames leading to unacceptable deflections. A second alternative to the lateral system was investigated. Concrete cast-in-place shear walls were selected and placed along the shared wall between the stairwell and mechanical room in the N-S direction and encase each stair well in the E-W direction. The shear walls in the E-W direction were assumed to carry only lateral loads.

The shear walls were analyzed using RISA 3D by drawing a plate member and then meshing it into smaller more accurate areas. The bounding columns were included in the analysis. The wall was modeled as 12" thick.

The locations of the shear walls illustrated in fig. F.25 moves the center of rigidity much closer to the center of mass making torsional effects minimal.

Seismic - Story Shear								
E - W								
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	Shear (K)
2	94.32			23.58	23.58	23.58	23.58	94.32
3	184.83			46.21	46.21	46.21	46.21	184.83
4	265.78			66.44	66.44	66.44	66.44	265.78
5	422.87			105.72	105.72	105.72	105.72	422.87
6	424.32			106.08	106.08	106.08	106.08	424.32
7	541.36			135.34	135.34	135.34	135.34	541.36
Roof	912.30			228.07	228.07	228.07	228.07	912.30
N - S								
2	94.32	47.16	47.16					94.32
3	184.83	92.42	92.42					184.83
4	265.78	132.89	132.89					265.78
5	422.87	211.43	211.43					422.87
6	424.32	212.16	212.16					424.32
7	541.36	270.68	270.68					541.36
Roof	912.30	456.15	456.15					912.30

Table F.16 – Seismic Loading on Shear Walls W/O Torsion Effects

Seismic - Story Shear (With Torsional Effects)								
E - W								
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	Shear (K)
2	94.32			26.35	26.35	23.58	23.58	99.85
3	184.83			49.79	49.79	46.21	46.21	192.00
4	265.78			68.43	68.43	66.44	66.44	269.74
5	422.87			111.35	111.35	105.72	105.72	434.14
6	424.32			111.48	111.48	106.08	106.08	435.12
7	541.36			139.56	139.56	135.34	135.34	549.80
Roof	912.30			270.92	270.92	228.07	228.07	997.99
N - S								
2	94.32	49.10	47.16					96.26
3	184.83	94.93	92.42					187.34
4	265.78	134.28	132.89					267.16
5	422.87	215.38	211.43					426.81
6	424.32	215.94	212.16					428.10
7	541.36	273.63	270.68					544.31
Roof	912.30	486.14	456.15					942.29

Table F.17 – Seismic Loading on Shear Walls W/ Torsion Effects

Building Height (ft)	H/400 (in)	Shear Wall N-S (in)	Shear Wall E-W (in)
96.67	2.90	1.03	1.03

Table F.18 – Shear Wall Deflections compared to H/400

The overall building drift as shown in table F.18 is within the allowable H/400 industry standard drift limit. The N-S walls are reinforced with (23) #11 bars and the E-W walls are reinforced with (19) #11 bars to resist the uplift force created by the lateral loads.

N-S		
Uplift (kips)	Bar Size	# of Bars
1891	11	23
E-W		
Uplift (kips)	Bar Size	# of Bars
1536	11	19

Table F.19 – Shear Wall Reinf. To Resist Uplift

Concrete Columns

PCA Col was used to design the columns in the concrete system for Parkridge 6. RISA 3D was used to obtain the combined axial and flexural loading on the columns. A selection of four columns was designed for the purposes of this report as the represent the worst case columns of their type. The columns locations are shown in the following figure F.26. All the column designs had slenderness included in the design. Column Interaction Diagrams and PCA Col output can be found in the appendix for each column.

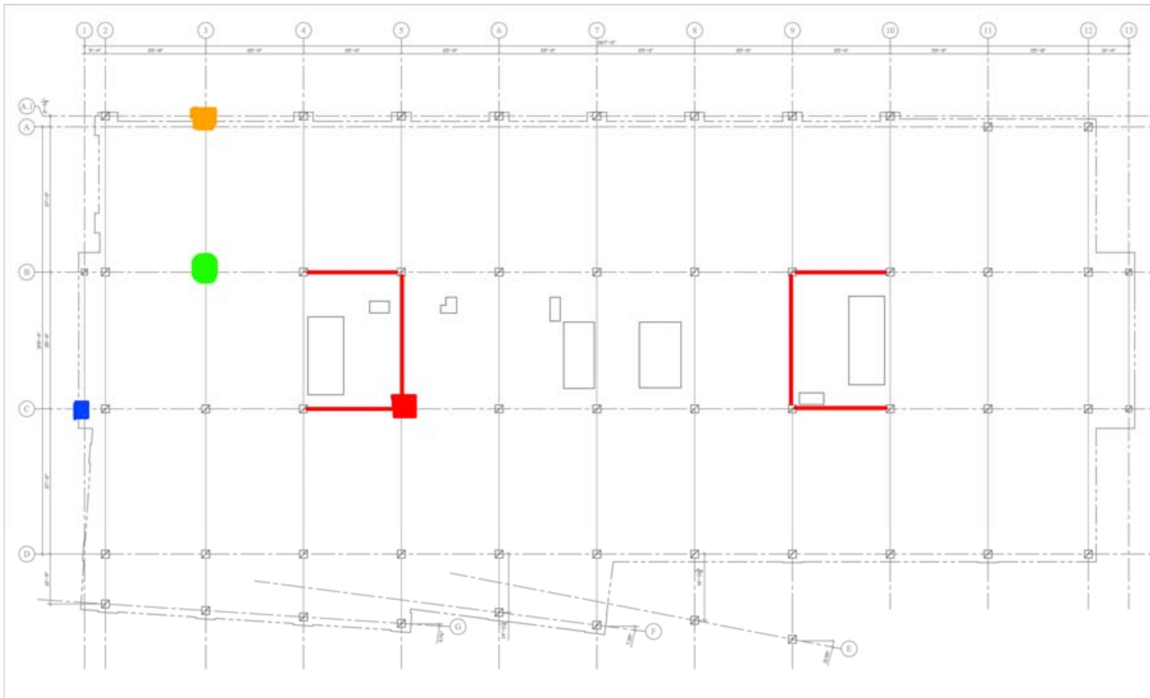


Fig. F.26 – 2nd Floor plan with Columns Analyzed Indicated

The column indicated by a red square in figure F.26 was designed to be a 24 in x 24in section reinforced with (16) #11 bars.

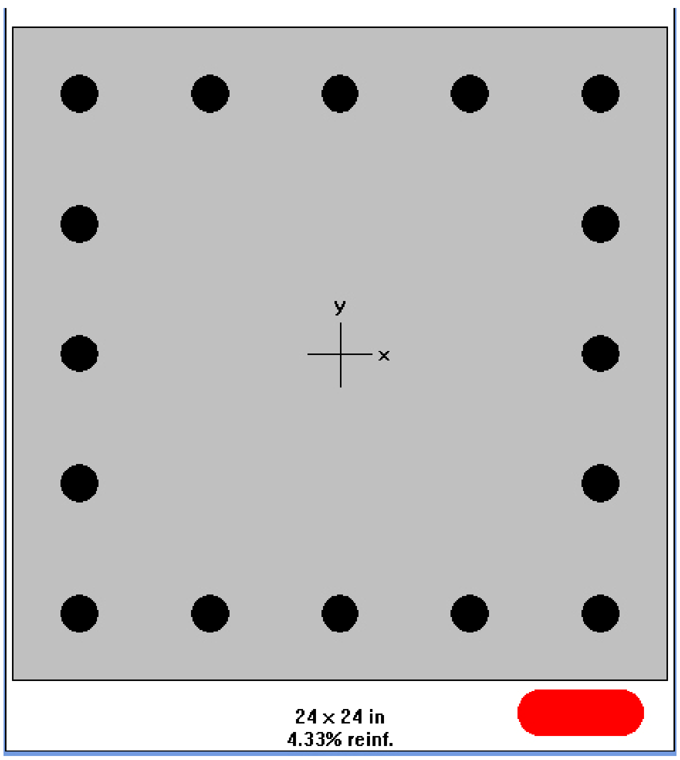


Fig. F.27 – Shear Wall Edge Column Cross Section

The column indicated by an orange block in figure F.26 is the worst case sloping column. These columns posed unique design problems through out my project through adding tension into the floor system and creating self induced moments into the column itself. These columns were design as 24in x 28in sections with (20) #10 bars.

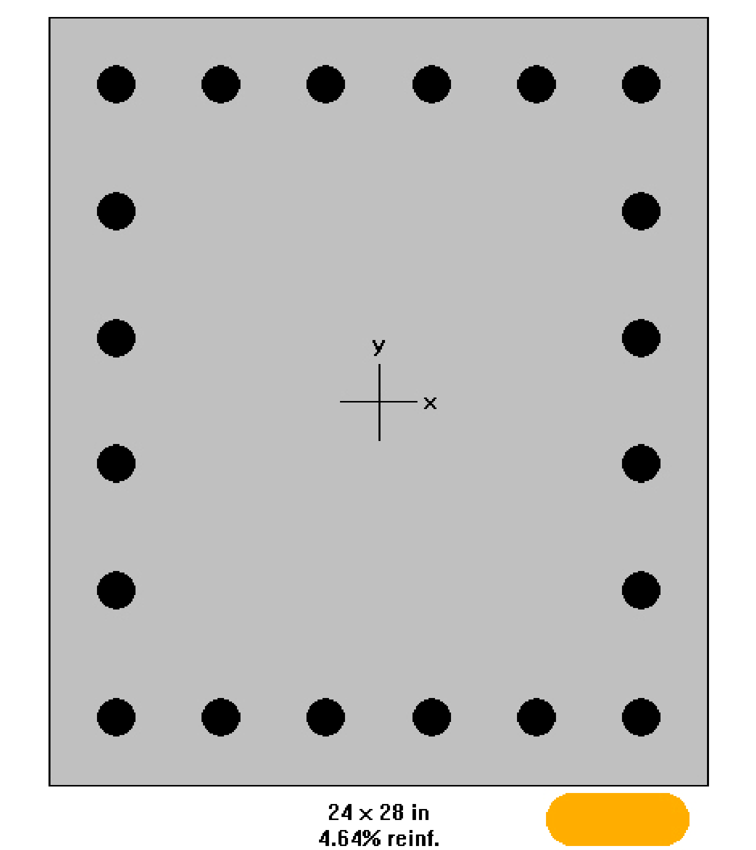


Fig. F.28 – Sloped Column Cross Section

The final column that was investigated was the column supporting the edge beams in the center bays. The columns are indicated by the blue box in figure F.26. The column was designed as an 18in x 18in cross section with (4) #9 bars.

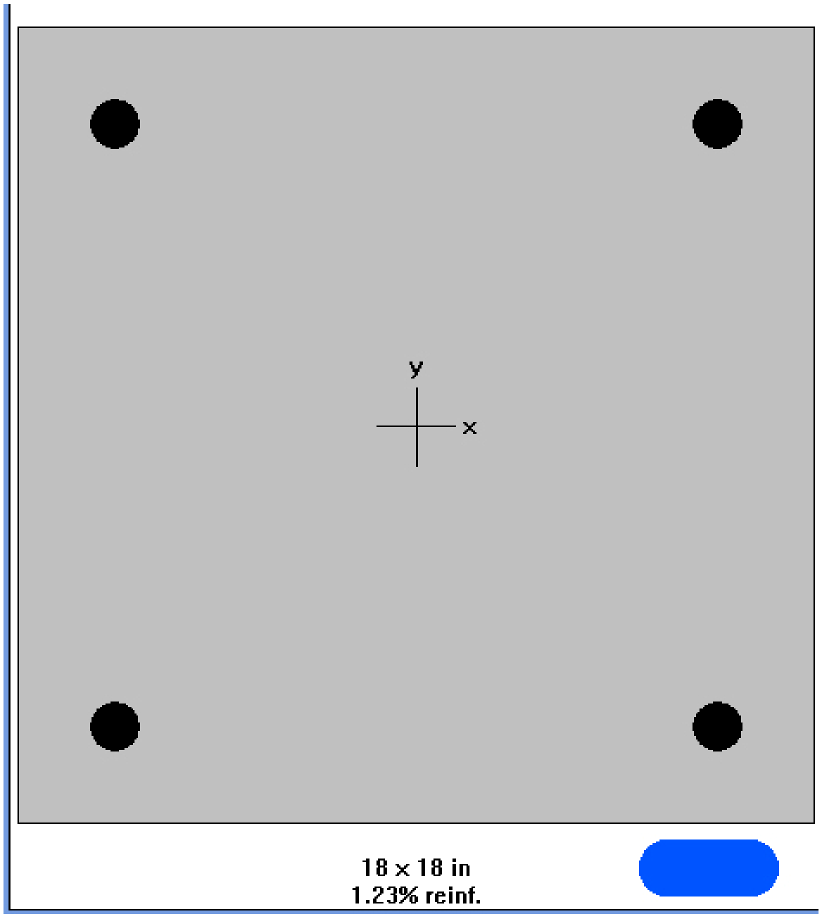


Fig. F.29 – Edge Frame Column Section

The worst case first interior column of the sloped column area is located at the column highlighted by a green box in figure F.26 and was designed as a 30in x 34in section with (20) #11 bars.

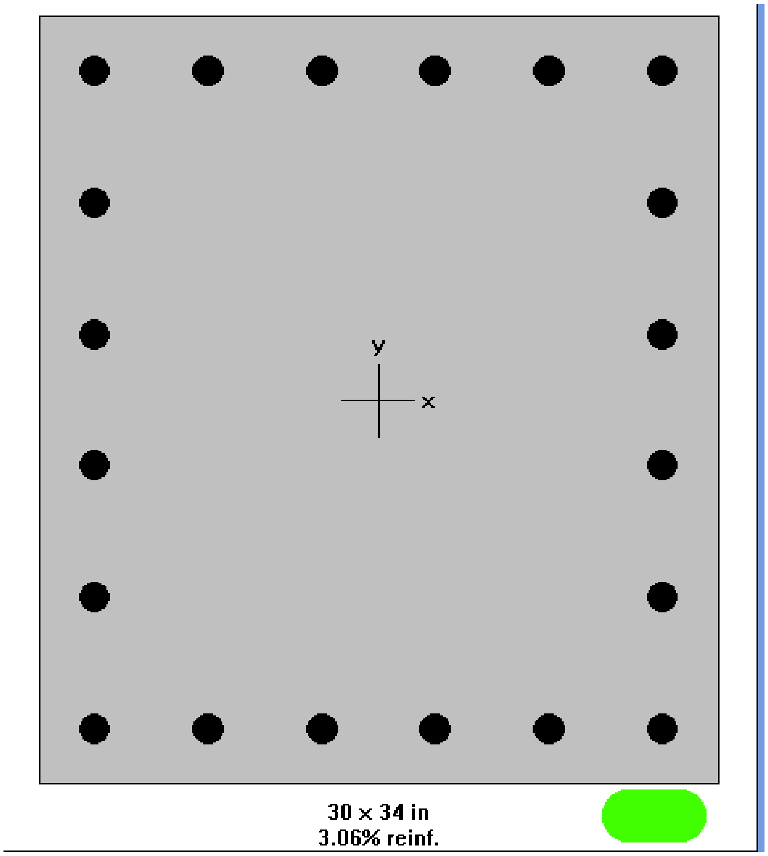


Fig. F.29 – First Interior Column Cross Section

Foundations

The additional loading of a concrete structure over that of the existing composite steel system will have a significant impact on the foundation design. The current foundations are designed as shallow foundation system of spread footings. A detailed design of a new foundation system is outside the scope of this report and has not been done.

The most likely outcome of a full foundation redesign with the additional loads created by a concrete structure would yield a deep foundation system using caisons, piles, or possibly mini piles. These deeper foundations would be required due mainly to significant increase in self weight of each member in the building. A quick calculation taking the applied axial force at the base of a column in pounds divided by the allowable bearing pressure of the soil of 3000 psf yields a foundation 1000 sq.ft. of surface in contact with the soil to create a spread footing this size would be both uneconomical and impractical.

Breadth – Construction Management

Cost Analysis

The cost analysis of each system was done by making detailed take offs of members and materials in each design and comparing the total structural systems costs based on labor, material, and equipment. MC^2 Ice software was used to estimate the costs for each structural system.

Existing Steel System	
Labor	$6,408,362
Material	$975,741
Equipment	$109,651
Total	**$7,493,754**
New PT Concrete System	
Labor	$1,844,563
Material	$1,650,612
Equipment	$42,550
Total	**$3,537,725**

Concrete - Total Savings / Loss	
Labor	$4,563,799
Material	($674,871)
Equipment	$67,101
Total	**$3,956,029**

Table F.20 – Cost Comparison

This cost comparison does not include the additions that will take place to the foundations system due to the proposed concrete systems. With additional foundation info the system costs would be within approximately $1.5 million.

The difference in labor costs can be directly related to the number of structural elements that need to be placed. Another impact to labor cost id the steel system requires a significantly larger build team made up of highly trained workers for the erection process. Concrete system does not require as large a labor force.

The high material costs in the concrete can be directly associated to the cost of the formwork. The formwork makes up 65% of the costs of the concrete system. Material costs for the concrete was also slightly adjusted to make up for the fact that the estimating software does not contain data for the required strength of concrete needed for the post-tensioned beams. The adjustment made was adding and additional 20% of concrete material.

Schedule

The scheduling was done using Microsoft project and was only done for the superstructure of each system. The existing composite steel construction was scheduled to be completed in 73 days with a 30 day lead time for fabrication and delivery of steel. The proposed concrete was scheduled to be completed in approximately 262 days allowing for concrete curing time and staged jacking of the post tension cables. If the projects started structural construction 4/11/07 the concrete would be finished almost 1 year from now if no work is done on Saturday and Sunday. The composite steel system would finish on 7/20/07.

Additional concerns created by the concrete schedule would be the need to employ the use of heaters during the placing of concrete during winter months as well as provide protection from the cold for curing concrete. These issues will also have impact on the total system costs that were not included in the previous estimate.

Constructability

Both systems provide unique challenges during construction however the Steel system itself will be the easier system to construct.

The sloping columns will pose significant issues for the concrete system as each portion of the column will have to be shored until the interior concrete beams and slabs have cured to a sufficient strength to hold and transfer the load from the sloped column to the interior columns. In the steel system a second crane which most likely will already be on site to assist with construction would be used to hold the column in place while another crane lifts the intermediate beam in place. The second crane will immediately be able to release the column as the steel beam and connecting interior column will have been design to adequate strength for construction.

Also the jacking of the post-tensioning in the concrete system will need to be monitored by on site inspectors and engineers to ensure that to much or not enough force is applied to the tendons.

The complexity of the placement of the post-tensioned strands will provide some issues. The position of the strands will need to be checked by on site engineers to be sure they are with in allowable tolerances.

Breadth – Mechanical

Current Parkridge Center – Phase VI utilizes a VAV system with additional air conditioning, A/C, units located on each floor. I have proposed to remove the A/C from each floor and replace with a more efficient chiller system on the roof.

To design the chiller the loads on the A/C units were needed in units of tons. The following table lists the loads on each of the A/C units:

AC Unit	Tot. MBH	Tons
1	425.38	35.45
2	414.35	34.53
3	597.29	49.77
4	638.33	53.19
5	0.00	0.00
6	0.00	0.00
7	529.65	44.14
8	616.16	51.35
9	643.47	53.62
10	640.49	53.37
11	596.72	49.73
12	638.07	53.17
13	637.75	53.15
14	634.91	52.91
15	596.34	49.70
16	637.47	53.12
17	593.51	49.46
18	613.58	51.13
19	596.54	49.71
20	637.50	53.13
21	592.84	49.40
22	612.88	51.07
23	0.00	0.00
24	0.00	0.00
	Total:	991.10

Table F.21 – A/C Unit Loads

Using the total load in tons I selected an air cooled chiller model 30XA from Carrier. The 30XA chiller is capable of handling 500 tons of load. I selected to use two chillers as to maintain uninterrupted service for maintenance of a unit or unexpected failure of a unit.

After talking with the mechanical team for the original project I learned that using chillers on the roof is indeed a more efficient system. However, this building is a commercial office building meaning each floor has the possibility of being rented by a different tenant and the billing of each floor would be possible using the individual A/C units. The billing using the chillers on the roof would possibly yield lower total energy costs for the building but a process to divide the costs between the individual tenants would need to be agreed to by each current tenant and any tenant in the future.

Conclusions and Recommendations

This investigation was done to find an alternative structural system that maintains the current floor layout and floor heights. To accommodate the requirements a post-tensioned concrete system using cast-in-place shear walls was chosen.

The chosen system succeeds is keeping the current layout of the floor decreasing overall usable area by a minimal amount. The system however would cause an increase in general floor to floor height to account for mechanical systems and other miscellaneous materials that need to be placed above the ceiling. Also the post-tensioned system would create a need for deeper foundations adding to cost.

The cost of the post-tensioned system has a significant savings when compared with the current composite steel system. The schedule requirements of the post-tensioned system however negate the cost benefits by increasing the general construction time of the building yielding a longer gap between cost of construction and income from tenant leasing.

If this building had a different use such as a new campus facility that would not have profit based on when the building opened I would recommend this system. However, Parkridge Center – Phase VI is comprised of rentable space that cannot turn a profit for the owner until it is complete. Based on this I would not recommend the post-tensioned system for Parkridge Center-Phase VI.

Appendix

Minimum Slab Thickness - Post-Tensioned One-Way		
L_n	25.67	ft
$L_n/45$	**7.00**	in
Self wt.	**87.5**	psf

Preliminary Column Size

Allowable Load		
F'c	4000	psi
Fy	60000	psi
Ast	12	in^2
Ag	576	in^2
Φ	0.7	
Pn,max	**1477.06**	**kips**

Column Dimensions		
b	h	ag
24	24	576
Reinforcement		
# Bars	Bar Size	Area
12	9	12

Column Actual Load		
Trib. Area	1030.5	sf
DL	20	psf
Slab SW	87.5	psf
LL	100	psf
Wu	289	psf
Col. SW	720	plf
Stories	7	
Actual	**2089.74**	**kips**

FA.1 – Minimum Slab Thickness and Preliminary Column Size

Concrete Properties		
F'c	5000	psi
F'ci	3750	psi
Loads		
slab	87.50	psf
DL	20.00	psf
LL	100.00	psf
TL	207.50	psf
$w_{prestress}$	96.75	psf
w_{net}	110.75	psf
Dimensions		
Thickness	7.00	in
Cover	1.25	in
Tendon Φ	0.50	in
a	4.00	in

Interior Span				
L	25.00	ft		
Mp	7558.59	ft-lbs		
F	22675.78	lbs/ft		
F/A	269.95	psi		
C.L. Mn	6292.61	ft-lbs		
M^f	6292.61	ft-lbs		
S	98	in^3		
				Class
f^+	500.57	psi	Tension	U
f^-	-1040.47	psi	Compression	U or T

PT Strands		
Fpu	270000	psi
Fpi	189000	psi
Pi	28.34	kips
Ap	0.15	in^2
# Strands	1	
Diameter	0.5	in
Apa	0.20	in^2

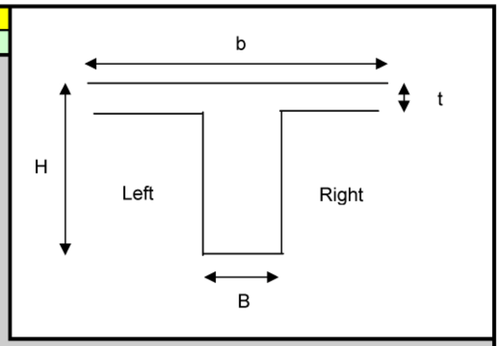
Exterior Span				
L	25.67	ft		
Mp	13980.38	ft-lbs		
F	41941.13	lbs/ft		
F/A	499.30	psi		
C.L. Mn	7297.86	ft-lbs		
M^f	7297.86	ft-lbs		
S	98	in^3		
				Class
f^+	394.32	psi	Tension	U
f^-	-1392.91	psi	Compression	U or T

PT Strands		
Fpu	270000	psi
Fpi	189000	psi
Pi	52.43	kips
Ap	0.28	in^2
# Strands	2	
Diameter	0.5	in
Apa	0.39	in^2

FA.2 – PT Slab Design

B4-3

User Input	
Calculated	



Span	37.17	ft
L/16	28	in
Depth	30	in
Depth Overide	30	in

Dimensions		
t	7.00	in
b	76.755	in
H	30	in
B	18	in

Slab Spans (min)		
Left	25	ft
Right	2	ft

Concrete Properties		
F'c	4000	psi
sw	150	pcf

Ec	3605.00	ksi
G	1567.39	ksi

Assumptions		
a	7.00	in
d	27.5	in
Φ	0.9	
Fy	60	ksi

Service Loading		
Slab	1.620	klf
Self	0.563	klf
DL	0.370	klf
LL	1.850	klf

Section properties						
A_{flange}	537.285	in^2	3.5	in	1880.498	in^3
A_{beam}	540	in^2	15	in	8100	in^3
y_t	9.26	in			9980.498	in^3
y_b	20.74	in				
I	73610.02	in^4				
S_t	7945.39	in^3				
S_b	3549.95	in^3				
A	1077.29	in^2				
r	8.27	in				

Applied Moments (factored)		
Ends	693.45	ft-kips
Midspan	346.73	ft-kips

Determine Required As (a=hf)		
As,ends	6.42	in^2
As,midspan	3.21	in^2
As,min	1.57	in^2
	1.65	in^2

Guess As,ends		
Bar Size	9	
#	7	
As,actual	7	in^2
Guess As,midspan		
Bar Size	8	
#	5	
As,actual	3.95	in^2

a,ends (Guess As,ends)		
a	6.86	in
a,midspan (Guess As,midspan)		
a	0.91	in

Required As		
As,ends	6.40	in^2
As, midspan	2.85	in^2

Actual As,ends		
Bar Size	9	
#	7	
As,actual	7	in^2
Actual As,midspan		
Bar Size	8	
#	4	
As,actual	3.16	in^2

Actual a,ends		
a	6.86	in
Actual a, midspan		
a	0.73	in

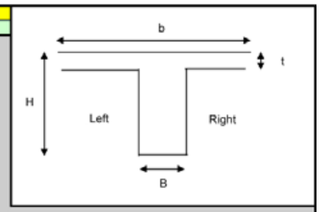
Check Tension Control, Ends		
c	8.07	in
c/d	0.29	TC
Check Tension Control, Midspan		
c	0.85	in
c/d	0.03	TC

Moment Capacity, Ends			Moment Capacity, Midspan		
Mn	842.40	ft-kips	Mn	428.76	ft-kips
ΦMn	758.16	ft-kips	ΦMn	385.88	ft-kips
	Ok			Ok	

FA.3 – Non-PT Beams Spreadsheet

B7-1 (Column Lines 3-10)

User Input	
Calculated	



Spans	3		
Span 1	37.17	ft	
Span 2	35.00	ft	72.17
Span 3	49.31	ft	121.48
L/20	30	in	
Depth	34	in	

Depth Overide	34	in

Dimensions		
t	7.00	in
b	105	in
H	34	in
B	28	in

Slab Spans		
Left	25	ft
Right	25	ft

Concrete Properties		
F'c	8000	psi
F'ci	5600	psi

Ec	5098.23	ksi
G	2216.62	ksi

Section properties						
A_{flange}	735	in^2	3.5	in	2572.5	in^3
A_{beam}	952	in^2	17	in	16184	in^3
y_t	11.12	in			18756.5	in^3
y_b	22.88	in				
I	158126.99	in^4				
S_t	14222.28	in^3				
S_b	6910.62	in^3				
A	1687.00	in^2				
r	9.68	in				

Kern points	
k1	-4.10
k2	8.43

Loads (Unfactored)		
Slab	87.50	psf
SW	991.67	plf
DL	20.00	psf
LL	100.00	psf
trib width	25	ft
Slab	2187.5	plf
SW	991.67	plf
DL	500	plf
LL	2500	plf
TL	6179.17	plf
w_p	3311.25	plf
w_n	2867.92	plf

Post-Tension 49.32 ft Span

PT Force		
Mp	1006.51	ft-kips
a	15.00	in
F	805.20	kips
F/A	477.30	psi

PT Strands		
Fpu	270000	psi
Fpi	189000	psi
Pi	1006.51	kips
Ap	5.33	in^2
# Strands	28	
Diameter	0.5	in
Apa	5.50	in^2

Max + & - Moment From Analysis		
Max M+	554	ft-kips
Max M-	343	ft-kips

Stress @ Service loads								
Positive Moment								Class
f	-477.30	-	467.44	=	-944.73	psi	Compression	U or T
		+	962.00	=	484.70	psi	Tension	U
Negative Moment								
f	-477.30	+	289.41	=	-187.89	psi	Compression	U or T
		-	595.61	=	-1072.90	psi	Compression	U or T

Post-Tension 35 ft Span

PT Force		
Mp	507.04	ft-kips
a	7.56	in
F	805.20	kips
F/A	477.30	psi

PT Strands		
Fpu	270000	psi
Fpi	189000	psi
Pi	1006.51	kips
Ap	5.33	in^2
# Strands	28	
Diameter	0.5	in
Apa	5.50	in^2

Max + & - Moment From Analysis		
Max M+	374	ft-kips
Max M-	166	ft-kips

Stress @ Service loads								
Positive Moment								Class
f	-477.30	-	315.56	=	-792.86	psi	Compression	U or T
		+	649.44	=	172.14	psi	Tension	U
Negative Moment								
f	-477.30	+	140.06	=	-337.24	psi	Compression	U or T
		-	288.25	=	-765.55	psi	Compression	U or T

Post-Tension 37.17 ft Span

PT Force		
Mp	571.86	ft-kips
a	8.52	in
F	805.20	kips
F/A	477.30	psi

PT Strands		
Fpu	270000	psi
Fpi	189000	psi
Pi	1006.51	kips
Ap	5.33	in^2
# Strands	28	
Diameter	0.5	in
Apa	5.50	in^2

Max + & - Moment From Analysis		
Max M+	350	ft-kips
Max M-	209	ft-kips

Stress @ Service loads								
Positive Moment								Class
f	-477.30	-	295.31	=	-772.61	psi	Compression	U or T
		+	607.76	=	130.46	psi	Tension	U
Negative Moment								
f	-477.30	+	176.34	=	-300.96	psi	Compression	U or T
		-	362.92	=	-840.22	psi	Compression	U or T

FA.4 – PT Beam Spreadsheet

Ultimate Strength Design - 49.3125 ft Span						
Load Factors						
DL	1.2					
LL	1.6					
Factored Loads						
slab	2625	plf				
SW	1190	plf				
DL	600	plf	TDL	4415	plf	
LL	4000	plf				
TL	8415	plf				
Max + & - Moment from Analysis						
Max M+	1620	ft-kips	support			
Max M-	1037	ft-kips	midspan			
Flexure at Midspan						
Fse	146459.68	psi		Cover to PT	4	in
ρ_p	0.0017					
Span/Depth	17.40					
Fps	202296.31	psi				
				Additional Reinforcement		
Fsy	256500	psi		Cover	2	in
Fse+30000	176459.68	psi		Bar Size	9	
Fse+60000	206459.68	psi		# Bars	3	
				As	3	in^2
Fs, design	202296.31	psi		Fy	60	ksi
				ρ_{rs}	0.0009	
Fps	1112.18	kips				
Frs	180	kips				
a	1.810	in				
Mn	3163.01	ft-kips				
Φ	0.90					
ΦMn	2846.71	ft-kips	OK			
As,min	2.56	in^2	OK			
q	0.051		OK, No Compression Steel			

Flexure at Support			
Fse	146459.68	psi	
ρ_p	0.0093		
Span/Depth	17.40		
Fps	165015.85	psi	
Fsy	256500	psi	
Fse+30000	176459.68	psi	
Fse+60000	206459.68	psi	
Fs, design	165015.85		
Fps	907.22	kips	
Frs	94.8	kips	
a	5.263	in	
Mn	2301.13	ft-kips	
Φ	0.90		
ΦMn	2071.02	ft-kips	OK
As,min	1.245	in^2	OK
q	0.206		OK, No Compression Steel

Cover to PT	13	in

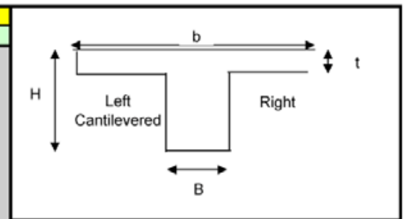
Additional Reinforcement		
Cover	2	in
Bar Size	8	
# Bars	2	
As	1.58	in^2
Fy	60	ksi
ρ_{rs}	0.0018	

Shear at Support		
Vu	240.33	kips
vc	299.87	psi
vu	286.11	psi

FA.5 – PT Beam Flexural Analysis Spread Sheet

1B2-6

User Input
Calculated



Span	47.44	ft
L/20	29	in
Depth	38	in
Depth Overide	38	in

Dimensions		
t	7.00	in
b	100.02	in
H	38	in
B	28	in
d	30.4	in

Slab Spans		
Left	2.67	ft
Right	25.67	ft

Column Dimensions		
w	24	in
d	24	in

Concrete Properties		
F'c	8000	psi
F'ci	5600	psi
SW	150	pcf

Ec	5098.23	ksi
G	2216.62	ksi

Loading		
DL	20	psf
LL	100	psf

Cover @ Support		
PT	8.75	in
Reinf.	4	in

Section properties						
A_{flange}	700.14	in^2	3.5	in	2450.49	in^3
A_{beam}	1064	in^2	19	in	20216	in^3
y_t	12.85	in			22666.49	in^3
y_b	25.15	in				
I	214415.18	in^4				
S_t	16688.00	in^3				
S_b	8524.93	in^3				
A	1764.14	in^2				
r	11.02	in				

Kern points	
k1	-4.83
k2	9.46

Factor For Shear	
Φ	0.75

Cantilevered Slab Load (Factored)					
DL	344.43	plf			
LL	427.20	plf			
Total	771.63	plf	Ecc.	2.50	ft

Interior Slab Load (Factored)					
DL	3311.43	plf			
LL	4107.20	plf			
Total	7418.63	plf	Ecc.	14.00	ft

Beam Load (Factored)		
DL	2982.03	plf
LL	2480.80	plf
Total	5462.83	plf

Uniform Torque	
101942.82	ft-lbs/ft

Beam Shear @ Column Face		
Vu	129578.2	lbs

Beam Torsion @ Column Face		
Tu	2418084	ft-lbs

Critical Section Vu		
Vu	115.74	kips
Critical Section Tu		
Tu	2159.83	ft-kips

Allowable Shear (no Φ factor)		
Vc	152267.28	lbs

Acp	1064	in^2
Pcp	132	in
fpc	985.14	psi

Check Tu against Tcr			
Tcr	1114.64	ft-kips	Need Torsional Reinf.

Assume	1.75	in	to torsion reinf.

xo	24.50	in
yo	34.50	in
Aoh	845.25	in^2
Ao	718.46	in^2
Ph	118.00	in

Check Member Cross Section			
0.250	≤	0.671	Ok

Required At, Assuming θ=45°		
At	0.80	s

Required Av		
Av	0.03	s

Bar Size	6

2At+Av	0.88	in^2

Dist. from Column Face (ft)	s	
d	1.12	in
4.53	1.17	in
6.53	1.23	in
8.53	1.29	in
10.53	1.36	in
12.53	1.44	in
14.53	1.52	in
16.53	1.62	in
18.53	1.73	in
20.53	1.86	in
22.53	2.01	in

FA.6 – Torsion and Shear Spreadsheet

Seismic Loading
ASCE7-05
Calculation of Building Weight

Floor	Area		DL		Weight	
1	--	SF	--	KSF	--	kips
2	31705.80	SF	0.0895	KSF	2837.67	kips
3	32715.30	SF	0.0895	KSF	2928.02	kips
4	32211.40	SF	0.0895	KSF	2882.92	kips
5	32643.40	SF	0.0895	KSF	2921.58	kips
6	31963.60	SF	0.0895	KSF	2860.74	kips
7	32443.76	SF	0.0895	KSF	2903.72	kips
Roof	18122.80	SF	0.0925	KSF	1676.36	kips
Garden	6694.84	SF	0.16	KSF	1071.17	kips
Mechanical	7959.25	SF	0.14	KSF	1114.30	kips
Penthouse roof	7959.25	SF	0.025	KSF	198.98	kips
				Total:	21395.46	kips

Floor	Beam Wt.	
1	--	kips
2	1090.735	kips
3	1116.628	kips
4	1130.045	kips
5	2193.918	kips
6	1078.584	kips
7	1239.130	kips
Roof	1239.686	kips
Beam Total:	9088.73	kips

Floor	Col. Wt.	
1	468.00	kips
2	415.90	kips
3	415.90	kips
4	431.89	kips
5	383.90	kips
6	383.90	kips
7	432.00	kips
Roof	159.30	kips
Column Total:	3090.79	kips

Precast Panels								
Wall	Perimeter		Height		DL		Weight	
1	765.81	LF	15.00	Ft	0.055	KSF	631.80	kips
2	855.25	LF	13.33	Ft	0.055	KSF	627.18	kips
3	950.65	LF	13.33	Ft	0.055	KSF	697.14	kips
4	815.85	LF	13.33	Ft	0.055	KSF	598.29	kips
5	790.08	LF	13.33	Ft	0.055	KSF	579.39	kips
6	799.50	LF	13.33	Ft	0.055	KSF	586.30	kips
7	807.50	LF	15.00	Ft	0.055	KSF	666.19	kips
						Total:	4386.29	kips

Total Building Weight:	**37961.27**	**Kips**

Calculation of Base Shear

S_s	0.200	
S_1	0.080	
S_{ms}	0.320	
S_{m1}	0.192	
S_{ds}	0.213	
S_{d1}	0.128	
R	3	
Ω_0	3	
C_d	2.5	
I	1	
Ct	0.016	
x	0.75	
h	115.17	ft
Ta	0.56	
C_s	0.071	
C_sW	2699.47	kips

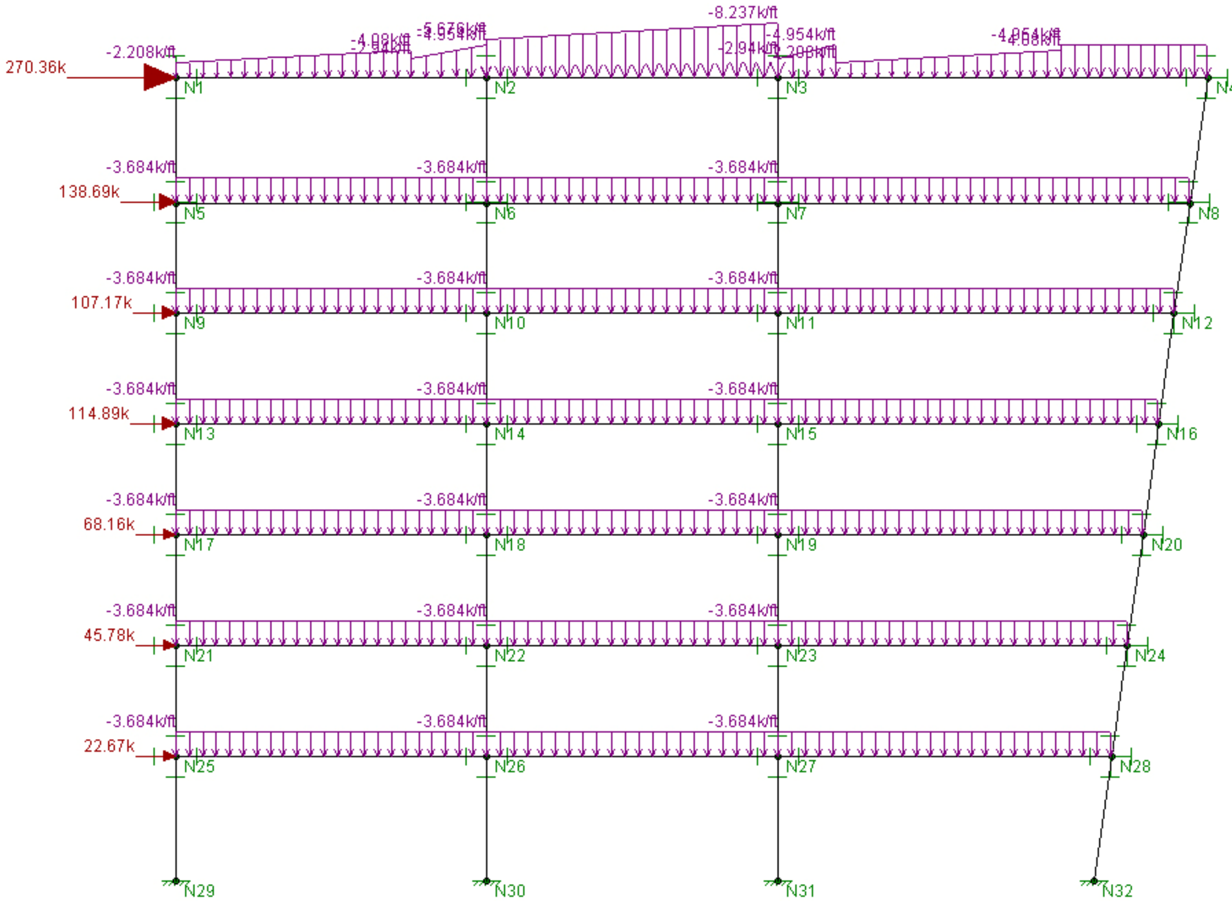
FA.7 – Seismic Load Determination Spreadsheet

E - W Frames			
Frame	Δ	Rigidity (1/Δ)	% Flr. Shear (R/ΣR)
12	0.003	333.33	25.00%
13	0.003	333.33	25.00%
14	0.003	333.33	25.00%
15	0.003	333.33	25.00%
	Tot.	1333.33	100.00%
N - S Frames			
Frame	Δ	Rigidity (1/Δ)	% Flr. Shear (R/ΣR)
1	0.019	52.63	10.36%
2	0.019	52.63	10.36%
3	0.019	52.63	10.36%
4	0.019	52.63	10.36%
5	0.019	52.63	10.36%
6	0.019	52.63	10.36%
7	0.019	52.63	10.36%
8	0.019	52.63	10.36%
9	0.019	52.63	10.36%
10	0.058	17.24	3.39%
11	0.058	17.24	3.39%
	Tot.	508.17	100.00%

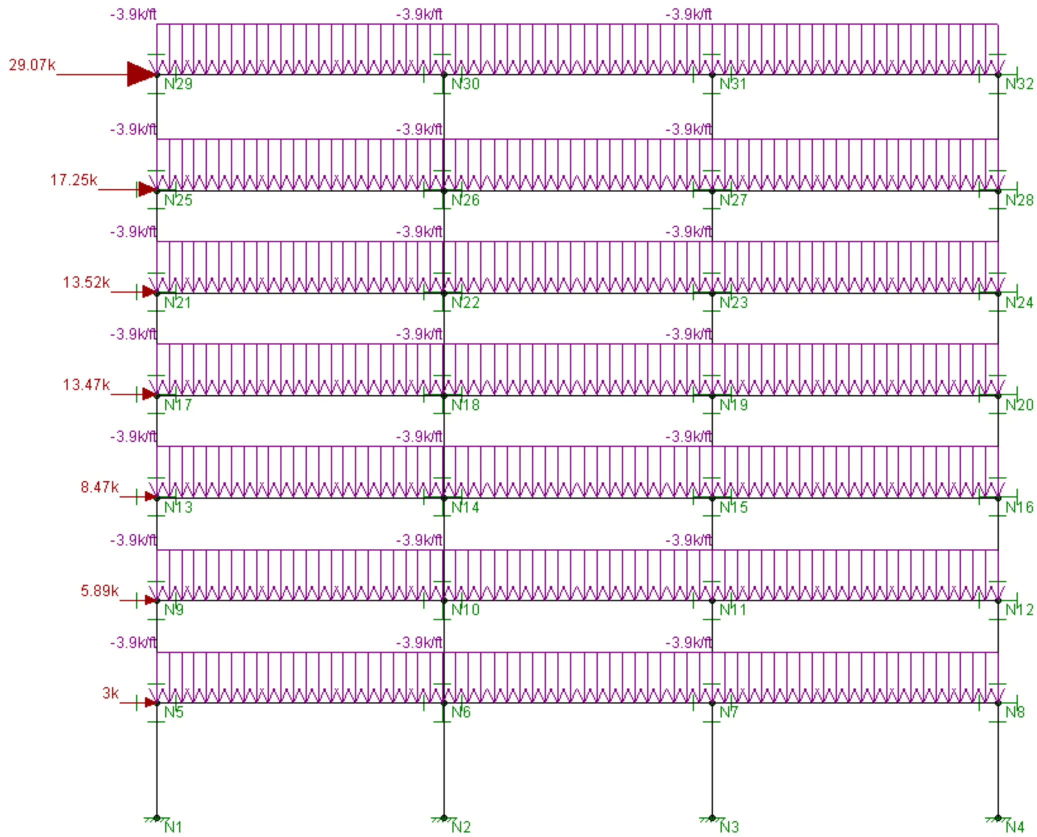
FA.8 – Moment Frames Rigidity

Seismic Torsional Force								
Element	**floor**	**$F_{torsion}$**	**Element**	**floor**	**$F_{torsion}$**	**Element**	**floor**	**$F_{torsion}$**
Frame 1	2	1.140	Frame 6	2	-7.975	Frame 11	2	-0.290
	3	2.348		3	-16.423		3	-0.598
	4	3.572		4	-24.990		4	-0.910
	5	6.227		5	-43.567		5	-1.587
	6	5.565		6	-38.929		6	-1.418
	7	7.265		7	-50.823		7	-1.851
	Roof	15.335		Roof	-107.286		Roof	-3.907
Frame 2	2	1.487	Frame 7	2	-3.078	Frame 12	2	17.288
	3	3.062		3	-6.339		3	35.602
	4	4.660		4	-9.646		4	54.173
	5	8.124		5	-16.817		5	94.446
	6	7.259		6	-15.027		6	84.391
	7	9.477		7	-19.618		7	110.176
	Roof	20.006		Roof	-41.412		Roof	232.575
Frame 3	2	2.114	Frame 8	2	-1.907	Frame 13	2	17.288
	3	4.354		3	-3.928		3	35.602
	4	6.625		4	-5.976		4	54.173
	5	11.550		5	-10.419		5	94.446
	6	10.321		6	-9.310		6	84.391
	7	13.474		7	-12.155		7	110.176
	Roof	28.443		Roof	-25.658		Roof	232.575
Frame 4	2	3.656	Frame 9	2	-1.382	Frame 14	2	-17.288
	3	7.529		3	-2.845		3	-35.602
	4	11.456		4	-4.329		4	-54.173
	5	19.973		5	-7.548		5	-94.446
	6	17.846		6	-6.744		6	-84.391
	7	23.299		7	-8.805		7	-110.176
	Roof	49.183		Roof	-18.587		Roof	-232.575
Frame 5	2	13.502	Frame 10	2	-0.355	Frame 15	2	-17.288
	3	27.804		3	-0.731		3	-35.602
	4	42.307		4	-1.112		4	-54.173
	5	73.759		5	-1.938		5	-94.446
	6	65.907		6	-1.732		6	-84.391
	7	86.043		7	-2.261		7	-110.176
	Roof	181.634		Roof	-4.773		Roof	-232.575

FA.9 – Moment Frame Torsional Distribution



FA.10 – Moment Frames with Seismic Loading RISA Model



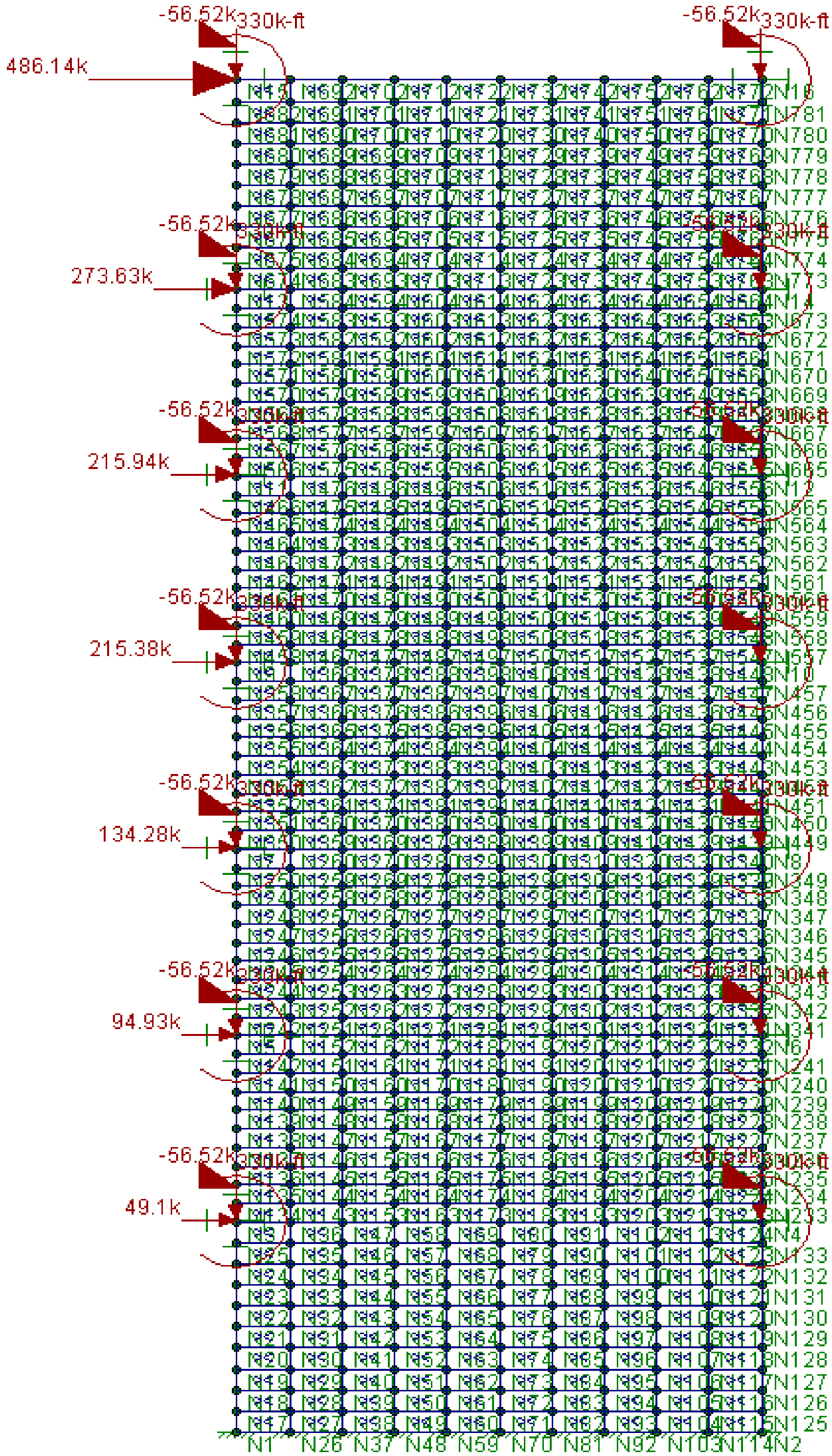
FA.11 – Moment Frames at Non Sloped Columns RISA Model

E - W Frames			
Frame	Δ	Rigidity (1/Δ)	% Flr. Shear (R/ΣR)
3	0.003	333.33	25.00%
4	0.003	333.33	25.00%
5	0.003	333.33	25.00%
6	0.003	333.33	25.00%
	Tot.	1333.33	100.00%
N - S Frames			
Frame	Δ	Rigidity (1/Δ)	% Flr. Shear (R/ΣR)
1	0.0009	1111.11	50.00%
2	0.0009	1111.11	50.00%
	Tot.	2222.22	100.00%

FA.12 – Shear Wall Rigidities

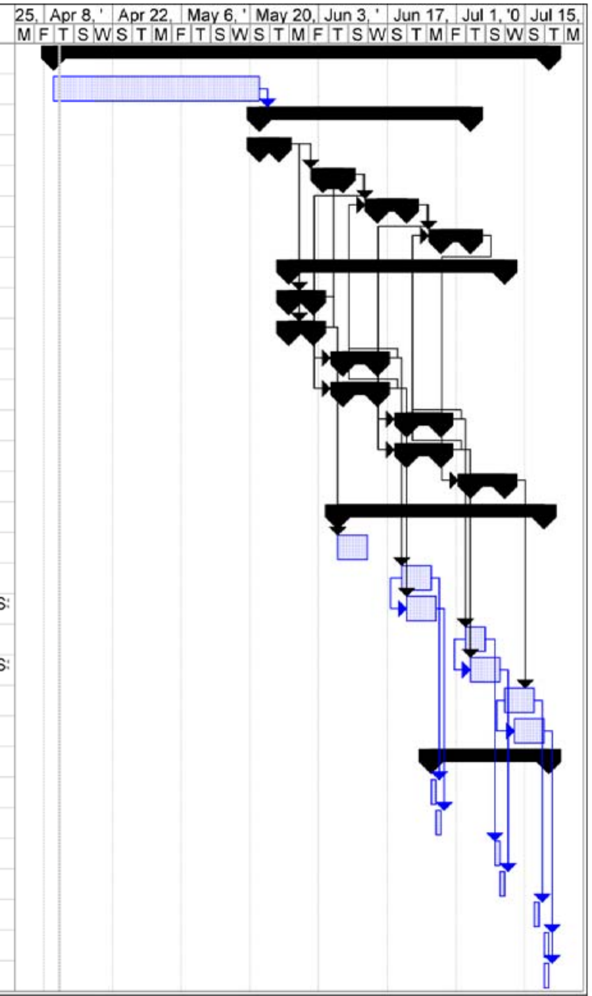
Seismic Torsional Force					
Element	floor	$F_{torsion}$	Element	floor	$F_{torsion}$
Frame 1	2	1.936	frame 6	2	-2.766
	3	2.510		3	-3.585
	4	1.388		4	-1.982
	5	3.944		5	-5.635
	6	3.780		6	-5.399
	7	2.953		7	-4.218
	Roof	29.992		Roof	-42.846
Frame 2	2	-1.936			
	3	-2.510			
	4	-1.388			
	5	-3.944			
	6	-3.780			
	7	-2.953			
	Roof	-29.992			
frame 3	2	2.766			
	3	3.585			
	4	1.982			
	5	5.635			
	6	5.399			
	7	4.218			
	Roof	42.846			
frame 4	2	2.766			
	3	3.585			
	4	1.982			
	5	5.635			
	6	5.399			
	7	4.218			
	Roof	42.846			
frame 5	2	-2.766			
	3	-3.585			
	4	-1.982			
	5	-5.635			
	6	-5.399			
	7	-4.218			
	Roof	-42.846			

FA.13 – Seismic Torsional Force in Shear Walls



FA.14 – N-S Shear Wall RISA Model

ID		Task Name	Duration	Start	Finish	Predecessors
1		**Composite Steel**	**73 days?**	**Tue 4/10/07**	**Thu 7/19/07**	
2		Lead Time	30 days	Tue 4/10/07	Mon 5/21/07	
3		**Columns**	**31 days**	**Tue 5/22/07**	**Tue 7/3/07**	**2**
4		**Ground-3rd floor**	**4 days**	**Tue 5/22/07**	**Fri 5/25/07**	
9		**3rd-5th**	**4 days**	**Mon 6/4/07**	**Thu 6/7/07**	**4,25,31**
14		**6th-7th**	**4 days**	**Fri 6/15/07**	**Wed 6/20/07**	**9,37,43**
19		**Roof**	**4 days**	**Thu 6/28/07**	**Tue 7/3/07**	**14,49,55**
24		**Beams**	**32 days?**	**Mon 5/28/07**	**Tue 7/10/07**	
25		**2nd**	**5 days?**	**Mon 5/28/07**	**Fri 6/1/07**	**4**
31		**3rd**	**5 days?**	**Mon 5/28/07**	**Fri 6/1/07**	**4**
37		**4th**	**5 days?**	**Fri 6/8/07**	**Thu 6/14/07**	**9**
43		**5th**	**5 days?**	**Fri 6/8/07**	**Thu 6/14/07**	**9**
49		**6th**	**5 days?**	**Thu 6/21/07**	**Wed 6/27/07**	**14**
55		**7th**	**5 days?**	**Thu 6/21/07**	**Wed 6/27/07**	**14**
61		**Roof**	**5 days?**	**Wed 7/4/07**	**Tue 7/10/07**	**19**
67		**Metal Deck**	**30 days**	**Thu 6/7/07**	**Wed 7/18/07**	
68		2nd	4 days	Thu 6/7/07	Tue 6/12/07	31FS+3 days
69		3rd	4 days	Wed 6/20/07	Mon 6/25/07	37FS+3 days
70		4th	4 days	Thu 6/21/07	Tue 6/26/07	43FS+3 days,69S
71		5th	4 days	Tue 7/3/07	Fri 7/6/07	49FS+3 days
72		6th	4 days	Wed 7/4/07	Mon 7/9/07	55FS+3 days,71S
73		7th	4 days	Wed 7/11/07	Mon 7/16/07	61
74		Roof	4 days	Fri 7/13/07	Wed 7/18/07	73SS+2 days
75		**Floor Slabs**	**18 days?**	**Tue 6/26/07**	**Thu 7/19/07**	
76		2nd	1 day?	Tue 6/26/07	Tue 6/26/07	69
77		3rd	1 day?	Wed 6/27/07	Wed 6/27/07	70
78		4th	1 day?	Mon 7/9/07	Mon 7/9/07	71
79		5th	1 day?	Tue 7/10/07	Tue 7/10/07	72
80		6th	1 day?	Tue 7/17/07	Tue 7/17/07	73
81		7th	1 day?	Thu 7/19/07	Thu 7/19/07	74
82		Roof	1 day?	Thu 7/19/07	Thu 7/19/07	74



Project: Steel Schedule.mpp
Date: Wed 4/11/07

Task
Split
Progress
Milestone
Summary
Project Summary
External Tasks
External Milestone
Deadline

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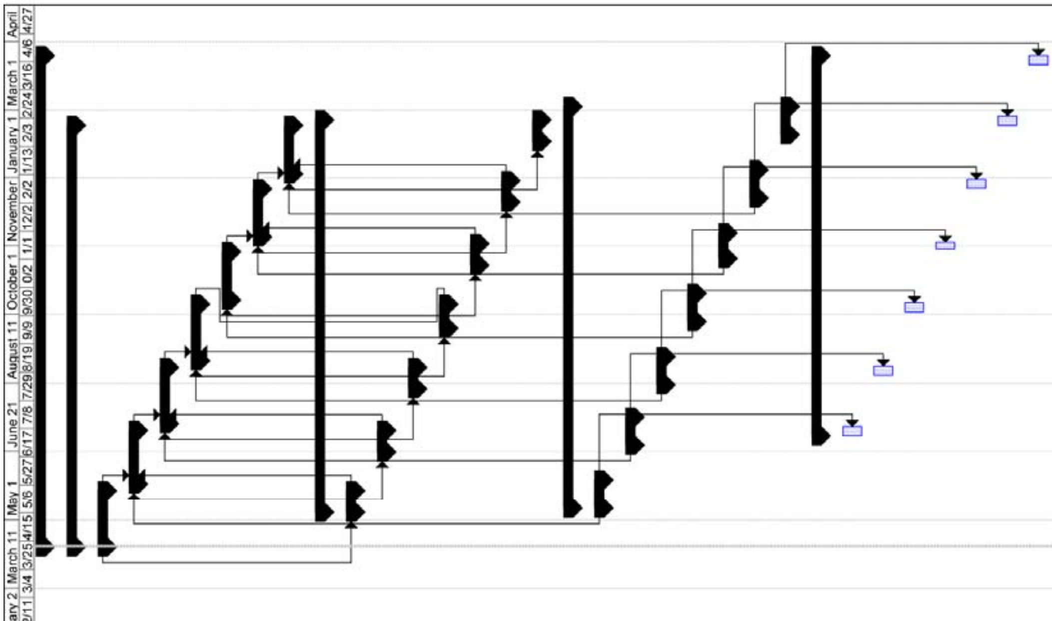
FA.15 – Steel Schedule From Microsoft Project

ID	Task Name	Duration	Start	Finish	Predecessors
1	Parkridge Center - Phase VI - PT Redesign	262 days?	Wed 4/11/07	Thu 4/10/08	
2	Concrete Columns & Walls	224 days	Wed 4/11/07	Mon 2/18/08	
3	Ground Floor-2nd Floor	30 days	Wed 4/11/07	Tue 5/22/07	
260	2nd Floor - 3rd Floor	30 days	Mon 5/28/07	Fri 7/6/07	3,1673,2073SS+12 days
517	3rd Floor - 4th Floor	30 days	Thu 7/12/07	Wed 8/22/07	260,1730,2102SS+12 days
766	4th Floor - 5th Floor	30 days	Tue 8/28/07	Mon 10/8/07	517,1787,2131SS+12 days
1003	5th Floor - 6th Floor	26 days	Fri 10/12/07	Fri 11/16/07	766,1844,2160SS+12 days
1226	6th Floor - 7th Floor	26 days	Wed 11/28/07	Wed 1/2/08	1003,1901,2189SS+12 days
1449	7th Floor - Roof	26 days	Mon 1/14/08	Mon 2/18/08	1226,1958,2218SS+12 days
1672	Beams	210 days?	Mon 5/7/07	Fri 2/22/08	
1673	2nd Floor	12 days?	Mon 5/7/07	Tue 5/22/07	3SS+18 days
1730	3rd Floor	12 days?	Thu 6/21/07	Fri 7/6/07	260SS+18 days
1787	4th Floor	12 days?	Tue 8/7/07	Wed 8/22/07	517SS+18 days
1844	5th Floor	12 days?	Fri 9/21/07	Mon 10/8/07	766SS+18 days
1901	6th Floor	12 days?	Wed 11/7/07	Thu 11/22/07	1003SS+18 days
1958	7th Floor	12 days?	Mon 12/24/07	Tue 1/8/08	1226SS+18 days
2015	Roof	12 days?	Thu 2/7/08	Fri 2/22/08	1449SS+18 days
2072	Slabs	213 days?	Thu 5/10/07	Mon 3/3/08	
2073	2nd Floor	15 days?	Thu 5/10/07	Wed 5/30/07	
2102	3rd Floor	15 days?	Tue 6/26/07	Mon 7/16/07	
2131	4th Floor	15 days?	Fri 8/10/07	Thu 8/30/07	
2160	5th Floor	15 days?	Wed 9/26/07	Tue 10/16/07	
2189	6th Floor	15 days?	Mon 11/12/07	Fri 11/30/07	
2218	7th Floor	15 days?	Thu 12/27/07	Wed 1/16/08	
2247	Roof	15 days?	Tue 2/12/08	Mon 3/3/08	
2276	PT Jacking	203 days	Tue 7/3/07	Thu 4/10/08	
2277	2nd Floor	5 days	Tue 7/3/07	Mon 7/9/07	2073FF+28 days
2278	3rd Floor	5 days	Fri 8/17/07	Thu 8/23/07	2102FF+28 days
2279	4th floor	5 days	Wed 10/3/07	Tue 10/9/07	2131FF+28 days
2280	5th Floor	5 days	Mon 11/19/07	Fri 11/23/07	2160FF+28 days
2281	6th Floor	5 days	Thu 1/3/08	Wed 1/9/08	2189FF+28 days
2282	7th Floor	5 days	Tue 2/19/08	Mon 2/25/08	2218FF+28 days
2283	Roof	5 days	Fri 4/4/08	Thu 4/10/08	2247FF+28 days



Project: Concrete Schedule
Date: Wed 4/11/07

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

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FA.16 – Concrete Schedule From Microsoft Project

	Labor	Material	Equipment	Subcontract	Temp Matl	Equip Rental	Other	Totals
%								
	$6,408,362	$975,741	$109,651	$0	$0	$0	$0	$7,493,754
0.00%	$0							$0
	$1,751,271							$1,751,271
	354,472							
0.00%		$0						$0
0.00%			$0					$0
0.00%					$0			$0
0.00%						$0		$0
0.00%							$0	$0
	$6,408,362	$975,741	$109,651	$0	$0	$0	$0	$9,245,025
0.00%								$0
0.00%								$0
Overall								
0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	
$0	$0	$0	$0	$0	$0	$0	$0	$0
0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	
$0	$0	$0	$0	$0	$0	$0	$0	$0
0.00%								$0
$0								$0
0.00%								$0
0.00%								$0
								$9,245,025
								$0
								$9,245,025

FA.17 – Steel Estimate Summary

	Labor	Material	Equipment	Subcontract	Temp Matl	Equip Rental	Other	Totals
%								
	$1,844,563	$1,650,612	$42,550	$0	$0	$0	$0	$3,537,724
0.00%	$0							$0
	$480,236							$480,236
	87,676							
0.00%		$0						$0
0.00%			$0					$0
0.00%					$0			$0
0.00%						$0		$0
0.00%							$0	$0
	$1,844,563	$1,650,612	$42,550	$0	$0	$0	$0	$4,017,960
0.00%								$0
0.00%								$0
Overall								
0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	
$0	$0	$0	$0	$0	$0	$0	$0	$0
0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	
$0	$0	$0	$0	$0	$0	$0	$0	$0
0.00%								$0
$0								$0
0.00%								$0
0.00%								$0
								$4,017,960
								$0
								$4,017,960

FA.18 – Concrete Estimate Summary

FA.19 – Mechanical – Alternate Chiller

Selection Guide

Model 30RA (R-22)	010	015	018	022	025	030	035	040	045	050	055
Capacity (tons)	10	14	16	22	24	27	35	38	43	47	54
Length (in.)	82	82	82	108	108	108	108	108	108	108	108
Approx. Weight (lb.)	946	1,068	1,209	1,523	1,589	1,705	2,844	2,914	3,218	3,313	3,515

Model 30RB (R-410a)	060	070	080	090	100	110	120	130	150	160	170
Capacity (tons)	57	67	76	87	96	106	119	127	145	153	167
Length (in.)	95	95	95	142	142	142	189	189	189	236	236
Approx. Weight (lb.)	4,705	4,911	5,258	6,590	6,813	7,067	8,238	8,593	9,808	10,900	11,235

Model 30RB (R-410a) cont.	190	210	225	250	275	300	315	330	345	360	390
Capacity (tons)	189	202	214	238	260	283	306	320	333	355	377
Length (in.)	283	283	283	330	377	424	472	472	472	519	566
Approx. Weight (lb.)	12,647	13,018	13,351	14,752	16,199	17,590	19,310	19,645	19,980	21,392	22,804

Model 30XA (R-134a)	080	090	100	110	120	140	160	180	200	220	240
Capacity (tons)	76	84	95	104	113	134	155	173	197	215	234
Length (in.)	142	189	189	189	189	236	236	283	283	330	330
Approx. Weight (lb.)	7,674	8,704	8,931	9,071	9,216	11,505	11,748	13,590	13,712	14,727	14,887

Model 30XA (R-134a) cont.	260	280	300	325	350	400	450	500
Capacity (tons)	254	274	293	310	330	390	431	465
Length (in.)	377	377	377	424	424	471	518	518
Approx. Weight (lb.)	16,853	17,022	17,362	18,834	19,040	23,953	25,975	26,269

Benefits at a Glance

For Building Owners & Managers
- Reduced operating costs
- Quiet operation
- Reliable operation
- Environmentally sound refrigerant

For Consulting Engineers
- Simple to select
- ASHRAE 90.1 compliant
- High efficiency
- Reduced energy consumption
- HFC refrigerants

For Contractors
- Streamlined installation
- Reduced installation expenses
- Reliable performance
- Easy to start-up and operate
- Plug-and-play hydronics

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FA.20 – Mechanical – Alternate Chiller Load Table