Thesis Final Report

Three PNC Plaza

Pittsburgh, PA

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The Pennsylvania State University

Architectural Engineering

Structural Option

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THREE PNC PLAZA

Pittsburgh, PA

ARCHITECTURE

Three PNC Plaza is a 23-story mixed use high-rise building located in the center of Downtown Pittsburgh. The property includes 326,000 square feet of office space, 185 room Fairmont Hotel, 28 luxury condominium units, restaurant and retail space, and 334 underground parking spaces. The building features a unique granite/limestone façade at the lower levels that is met with a glass and aluminum curtain wall covering the remainder of the building.

MECHANICAL & ELECTRICAL SYSTEMS

Mechanical:

- -Gas furnaces with sealed combustion and DX cooling in the Condos
- -Vertical chilled water fan coil units with electric heat in the Hotel
- -21 VAV AHU located throughout Floors 1-13
- -3 Centrifugal Chillers, 3 induced draft cooling towers, 1 plate and frame heat exchanger, 4 variable flow primary chilled water pumps, and 4 condenser water pumps located in the Central Plant

Electrical:

-Primary Electrical Service:	(6) 1000 kVA Transformer Vaults, each fed by (2) 4" 40-PVC conduits
-Office Electrical Service:	(9) 4" conduits, encased to a 2000A, 480Y/277 volt, Main Circuit Breaker
-Hotel Electrical Service:	(9) 4" conduits, encased to a 3000A, 480Y/277 volt, Main Circuit Breaker
-Condo Electrical Service:	(9) 4" conduits, encased to a 1000A, 480Y/277 volt, Main Circuit Breaker

BUILDING STATISTICS

Size:	780,000 SF
Number of Floors:	23 Above/ 3 Below
Project Cost:	170 Million
Construction Dates:	Aug 2006 - Oct 2009
Delivery Method:	CM at Risk
Occupancy:	Mixed-Use

PROJECT TEAM

Owner:	The PNC Financial Services Group, Inc.
Design Architect:	Gensler
Local Architect of Record:	Astorino
LEED Consultant:	Paladino & Company
General Contractor:	P.J. Dick
Engineer:	Chester Engineers CJL Engineering

STRUCTURAL SYSTEM

Three PNC Plaza is a 23 story high rise building with 3 levels of underground parking. The building uses a hybrid steel frame with concrete core walls. The concrete cores extend through the building up to the 14th floor, with a size change at the 8th floor. The typical bay for the steel structure is 30' by 42.5'. These bays are typically filled with W21X44 beams and W24X62 girders at 10' on center. The parking garage is a one way post tensioned concrete deck utilizing a 9" slab on 42.5' concrete beams. The typical beam for the garage is 36" wide and 24" deep.

R. Bryan Peiffer | Penn State Architectural Engineering, Structural Option

www.engr.psu.edu/ae/thesis/portfolios/2011/rbp5023/index.html

TABLE OF CONTENTS

Acknowledgments

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He would also like to say a special thank you to his parents, Scott and Tracy Peiffer, and his friends and family. Everything he has accomplished over the past five years would not have been possible without them.

Executive Summary

The thesis study performed examined the structural redesign of Three PNC Plaza from a steel frame building with composite slabs to a strictly concrete building. The structural depth consists of the design of the new one way post tensioned floor system in long direction of the building, supported by wide shallow post-tensioned girders spanning in the short direction. The new system was designed to have minimal impact on the buildings architecture, as a result original column locations were maintained for the redesign to keep the 42.5'-0" by 30'-0" bays intact. The design utilized an 8" post-tensioned slab with 60" by 18" girders spaced at 30' center to center. The slab was broken up into 5 distinct zones, based on the number of spans as well as span distances, and each were designed individually using the ADAPT-PT program. Girders were also broken up into typical girders A, B, C and D, and each type was designed using the same program.

The existing lateral system utilized 7 distinct concrete shear wall cores located throughout the center of the build. These cores rise up until the 14th story where the steel moment frame structure assumes full capacity of lateral loads. The structural redesign follows the same premise only with a concrete moment frame. The shear walls had to be relocated resulting in 2 larger sets of core walls instead of the former 7. The new system is classified under a Dual System Design with both ordinary concrete moment frames and shear walls. The code does not specifically address post-tensioned lateral systems so several conservative assumptions were made during analysis.

Relocating the core walls of the building also resulted in the movement of vertical circulation wells. This aspect was explored in an architectural breadth to see the impact on floor plans. The mix use nature of the building makes floor plans vary greatly throughout the building. The first 14 floors of the building are mainly office level and did not see much of an impact by the change. However, the hotel/condo portions of the building through levels 15 to 23 were affected. Limited floor plans were provided but in a typical hotel/condo floor plan one of the hotel rooms had to be removed due to elevator shafts. This effect could cause potential issues for the condos on the top floors of thae building that take up more floor space.

The second breadth topic focused on the construction management aspects of the redesign. A cost estimates and new schedules were composed for each system. It was found that the systems were around the same bare material cost however the new concrete redesign required additional overhead and production costs. The concrete building was also found to take longer to construct over the original steel framed building.

Building Overview

Three PNC Plaza is a 25 story, 780,000 square foot, mixed use high-rise building located in the heart of downtown Pittsburgh, Pennsylvania as seen in figure 2 highlighted in red. The erection of this building was a significant part to revitalizing the downtown area and marked the first new high-rise built in the city in the last 20 years.

Figure 1- Three PNC Occupancy Layout

The building is mixed-use and allows for several different tenants to occupy the building as seen in figure 1. Fairmont Hotels and Resorts moved into the building in March, 2010 with 185 rooms that are located on floors 14 through 23. Along with the Fairmont Hotels, 28 Residences condominium units will occupy floors 14 through 23 in the fall of 2010. The building has 10 floors of office space located from the 3rd through 13th floor. These office spaces are home to PNC Bank and the REED Smith Law Firm. The lower floors of the building house several different retail stores, a restaurant and a wine bar.

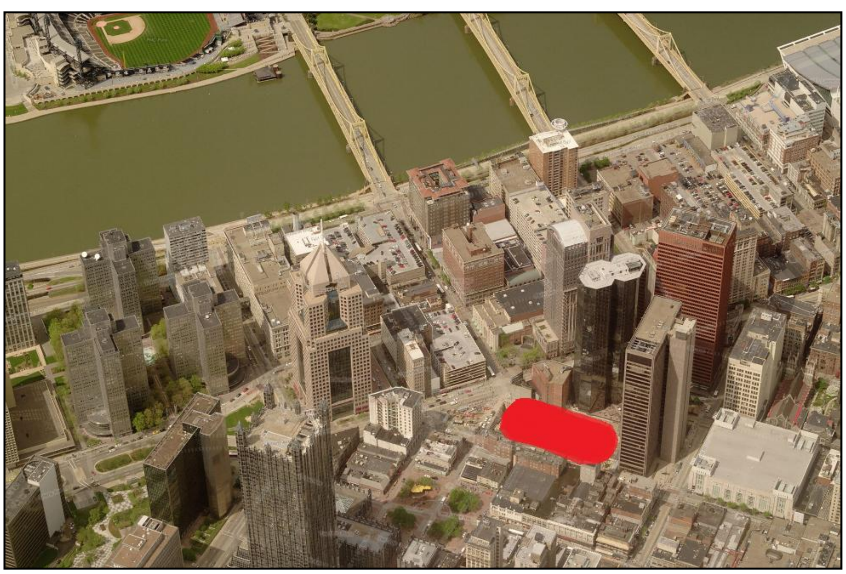


Figure 2- Three PNC Site Location

Existing Structural System

Foundation System

Pittsburgh is known for alluvial deposits which mean shallow foundations were not possible and deep foundations were required for Three PNC Plaza. Also, the Pittsburgh area soil overburden is 60' to bedrock. This means that after the 30' of excavation for the buildings parking garage structure, 30' of soil would still remain until the bedrock would be reached. Several different options for the foundation of the building were considered such as; auger cast pile, piles, H-piles, and caissons. Ultimately, the foundation system chosen for Three PNC Plaza were caissons bearing on bedrock to achieve maximum axial capacity. Four different size caissons were chosen for the foundation as seen in the Caisson Schedule in figure 3. The caissons were designed for a typical column reaction of 3500 kips. Brayman Construction Corporation was in charge of the installation of the 121 caissons for the building. A typical caisson detail has been provided in figure 2. The caissons bearing value is 15 tons per square foot and were drilled to auger refusal or socketed into the bedrock.

CAISSONS $F_{BR.}$ =30K/SQ. FT.

MARK	SIZE Ø	VERT.REINF. length=3 X DIA.	TIES	DOWELS
A.	48"	7-#10	#4@18" O.C.	4-#8 X 8'-0" DEVELOP INTO PEDESTAL
B.	54"	9-#10	#4@18" O.C.	
C.	42"	7-#9	#3@18" O.C.	
D.	60"	9-#11	#3@18" O.C.	

Figure 3- Caisson Schedule

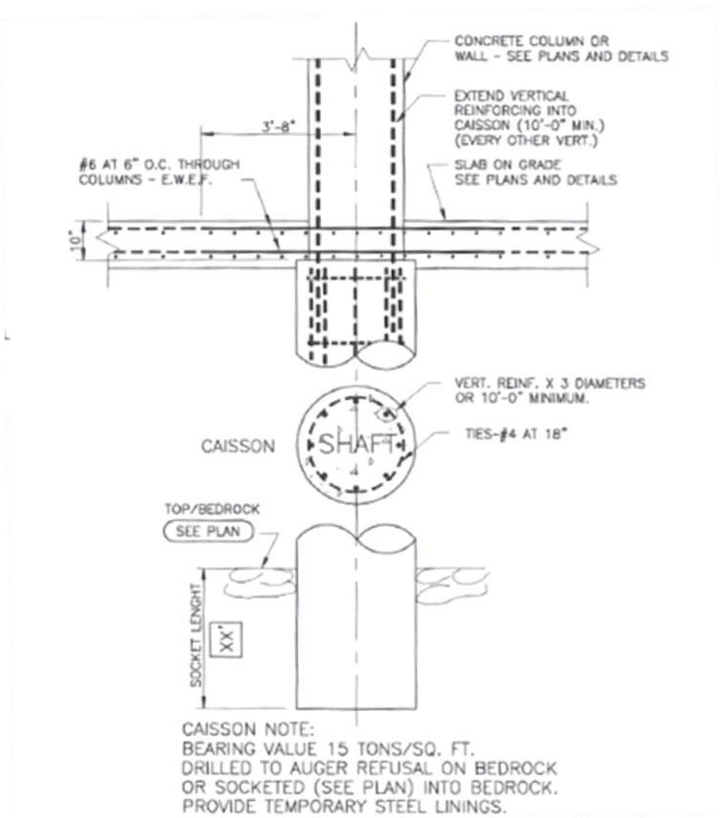


Figure 4- Caisson Detail

Floor System

Three PNC Plaza uses a composite steel and concrete floor system with a typical bay size of 30'-0" x 42'-6". The composite slab is composed of 2" 18-gauge metal floor deck with 3-½" light weight concrete, netting a total thickness of 5-½". The concrete is reinforced with one layer of 6x6-W2.1xW2.1 wielded wire fabric. The composite deck transfers its load to fill beams that are placed at 10'-0" on center and primarily W21X44 beams with W24X62 girders. This floor design is used throughout the structure and different sized fill beams are used to deal with higher load areas.

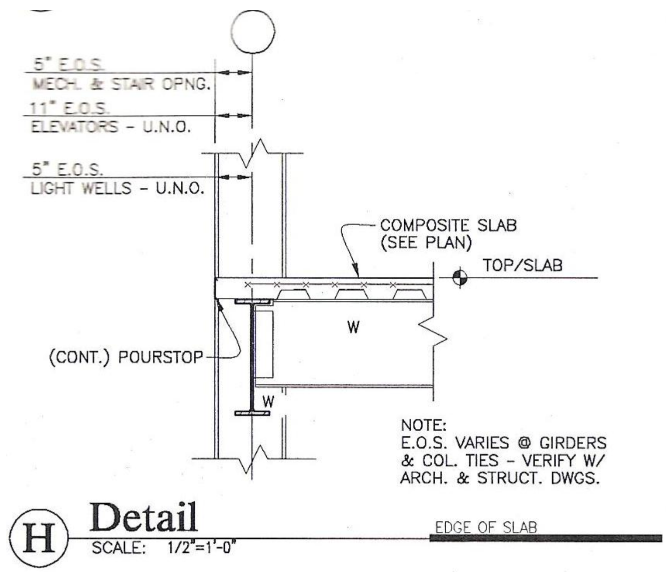


Figure 5- Composite Floor

Columns

Three PNC Plaza uses a variety of steel columns and concrete shear walls to support the gravity load of the building. The size of these columns can range in sizes from W14x68 all the way to a W14x740 in some cases. The core of the building is supported by concrete shear walls up until the 14th floor which they then switch over to steel columns. The remainder of the building is supported by steel columns from the ground floor that attach to concrete columns located in the parking garage. The steel columns attach to the concrete shear wall via reinforced corbels. The steel columns in the building are spliced together at a typical distance of 24'-0" as see in figure 6.

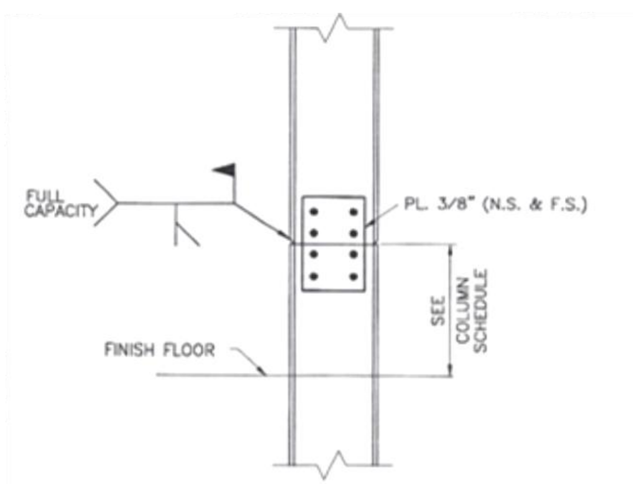


Figure 6- Splice Detail

Roof System

The roof structural system is very similar to the floor structural system used throughout the building. It utilizes the same composite deck and slab configuration along with the same typical bay dimensions. However, the fill beams are spaced closer together, at a typical spacing of 7.5 feet. These fill beams can differ in size from a W21x44 to a W27x129.

Lateral System

The main lateral resistant system used in Three PNC Plaza is a combination of several concrete shear wall cores and steel moment frames. The shear walls are located throughout the core of the building and encase the stairwells and elevators as seen in figure 7 highlighted in red. The core walls start at the lowest level of the parking garage structure and extend up until the 14th floor where they are met with steel columns and the moment frame takes over for the remainder of the building. All of the shear walls used a concrete with a compressive strength of 5000 ksi. The reinforcement for the shear walls changed depending on the location and can be seen in the shear wall Reinforcement schedule located in Appendix D. A more detailed view of the shear walls at key locations of the wall can be in figures 10-13.

Figure 7- Shear Wall Layout

Gravity Loads

Location	Design (IBC 2003)	Thesis (ASCE 7-10)
Retail	100 psf	100 psf
Office	50 psf	50 psf
Library	150 psf	150 psf
Hotel	40 psf	40 psf
Condominium	40 psf	40 psf
Ballroom	100 psf	100 psf
Garage	40 psf	40 psf
Mechanical Rooms	200 psf	-
Assembly Areas	100 psf	Depends on Area
Balconies	100 psf	1.5*Live Load
Restaurants	100 psf	-
Roof	30 psf	20 psf
Stairs and Lobby	100 psf	100 psf
Corridors	80 psf	80 psf

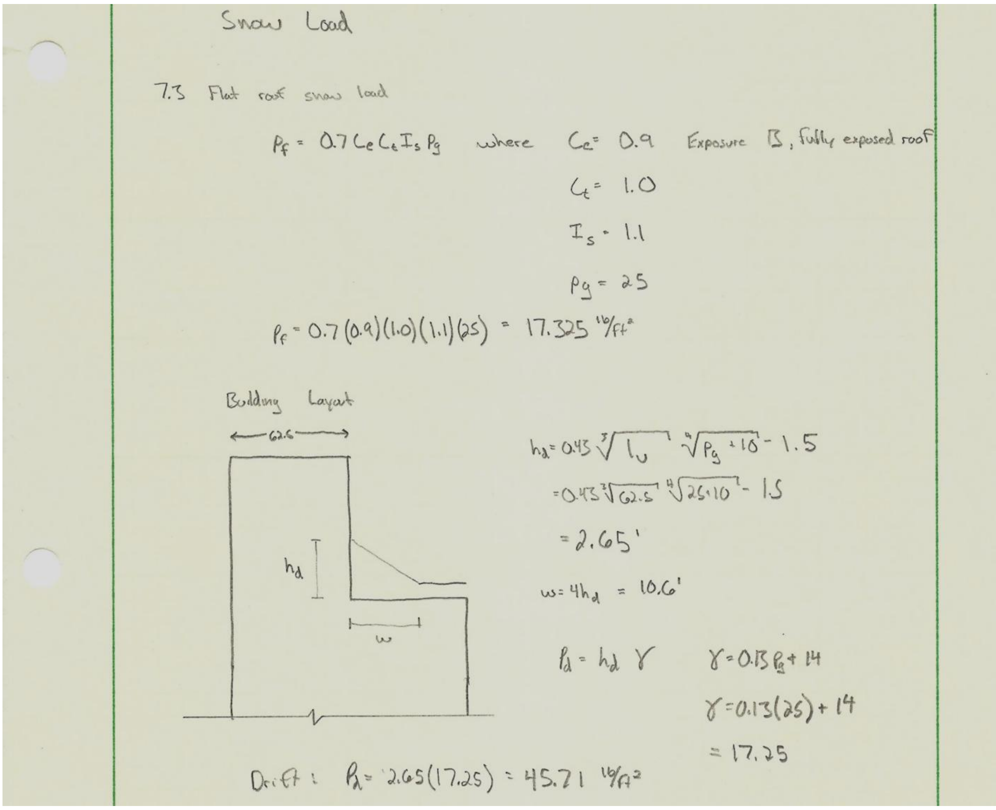
Floor Dead Loads	
Composite Decking	44 psf
Superimposed Dead Load	30 psf
Total	**74 psf**

Curtain Wall Dead Load:

Assumed curtain wall was 8” thick and that the material weighted 40psf. This resulted in a load of 60plf.

Snow Loads

Snow loads were designed by section 7.3 of ASCE 7-10. It was found that the Pf would be 17.325 lb/sq.ft. The shape of the building also results in a drift load calculation. The calculations can be seen below.



Snow Load

7.3 Flat roof snow load

$P_f = 0.7 C_e C_t I_s P_g$ where $C_e = 0.9$ Exposure B, fully exposed roof

$C_t = 1.0$

$I_s = 1.1$

$P_g = 25$

$P_f = 0.7(0.9)(1.0)(1.1)(25) = 17.325$ lb/ft²

Building Layout

$h_d = 0.43\sqrt[3]{l_u}\sqrt[4]{P_g+10} - 1.5$

$= 0.43\sqrt[3]{62.5}\sqrt[4]{25+10} - 1.5$

$= 2.65'$

$w = 4h_d = 10.6'$

$P_d = h_d \gamma$ $\quad \gamma = 0.13P_g + 14$

$\gamma = 0.13(25) + 14$

$= 17.25$

Drift: $P_d = 2.65(17.25) = 45.71$ lb/ft²

Figure 8- Snow Load Calculations

Proposal

Structural Depth

Problem Statement

The existing structural system for Three PNC Plaza consists of concrete shear walls and a steel moment frame with a lightweight concrete composite decking system. It was found throughout technical reports 1 to 3 that the current system used in the building is most likely the optimal design for the building. Therefore, when exploring alternative systems to be used for the building, they may not result in the most effective system for the building. However, it is being proposed to change the current system in the building to one that utilizes only concrete material. The change to a concrete material could reduce costs depending on which system is utilized and provided a smaller floor depth.

Problem Solution

Research from Technical Report #2 provided insight into what systems would be valid alternatives for the building. It resulted in the Two-Way Post-Tensioned slab being the front runner from a concrete system stand point due to its ability to span long distances. However, a further investigation into the system will be required to see if it is an adequate alternative. The Two-Way Flat Plate system would also be another system that could be used but would require additional columns added to reduce the 42.5' spans found in the building. With the new system in place the concrete shear walls will have to be redesigned and extended throughout the entire building to provide adequate lateral resistance. Due to the increased weight of the building loads will have to be reexamined for both gravity and lateral. The new system will then be compared to the current system to see the strengths and weaknesses of each.

Breadth Topics

Construction Breadth

A construction management breadth analysis will be performed to see the impact of the new structural system will have on the cost and schedule of the building. The new cost will be done using RS Means for an initial estimate and further research will be devoted on how to provide a more thorough analysis. Once the new cost and schedule information has been obtained it will be compared to the original cost and schedule to determine if the new design is practical from a cost/time perspective.

Architectural Breadth

Due to the changing of the structural design for the building aspects of the architecture could possibly be impacted and need redesigned. A further investigation into the impact on the exterior curtain wall facade will be performed. Also, changes to the floor plan layouts of the building will be investigated. Key areas of interest will be the impact the new structural system has to the ballroom and mezzanine levels.

Design of Proposed Gravity System

Initial Column Layout

The existing column layout made 42.5'-0" by 30'-0" exterior bays and 20'-0" by 30'-0" interior bays. This layout was preserved in the redesign to avoid any major architectural impacts to the floor plans from new column locations. New columns were added to the interior section of the structure. A preliminary size for the columns of 36" by 36" was used during this stage of the design as a general starting size. This column size of 36" was originally chosen to match with existing concrete columns in the parking structure located underneath the building. The column layout can be seen highlighted in blue in figure 9 below.

Figure 9 - Initial Column Layout

Preliminary Slab/Girder Sizes

Preliminary sizes for slabs, girders, and columns were determined by research from online reads, existing building plans, and standard practice rules. The overwhelming result from the research suggested starting with an 8" slab to handle the 30' girder spacing along with the 100 psf live loading and 20 psf superimposed dead load. Typical size of 60" wide by 20" deep would be the starting size for the girders and should be capable of spanning the need 42.5'-0" at the longest span lengths. A preliminary concrete strength of 5000 psi was chosen. Also, it was assumed that half the girder width could support half the tributary area. At this point the lateral system was not being considered and a typical girder layout was assumed without shear walls being present in the design as seen in figure 10. This resulted in the typical girder spanning over both exterior spans and the interior for a total of 3 spans.

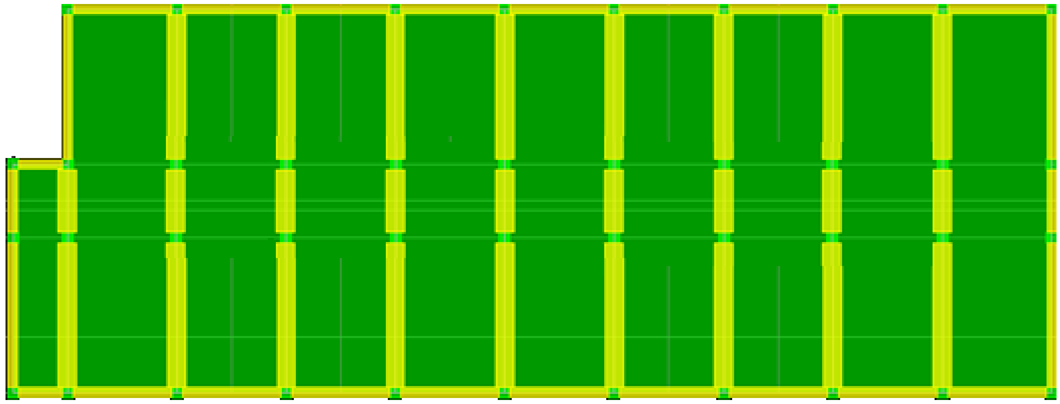


Figure 10 – Initial Post-Tension Girder Layout

Design of Post-Tensioned Slab

With the preliminary size for the slab found the next step performed was sectioning the floor plan for the ground level to the 14th floor into four design areas. These areas were labeled A through D, as seen in figure 11. This step was then repeated for the additional levels 15 through 23 as seen in figure 12. These design areas vary mainly in number of spans and also span lengths. To design these slab sections the computer program ADAPT-PT version 8 was utilized in accordance to the newest code in the program (ACI318-05 / IBC 2006). The concrete strength used for the slab and columns were 6000 psi and 10000 psi respectively. Serviceability Design Requirements were used in accordance to pre-stressed Class U properties from Chapter 18 of ACI 318-08. The stress limits used for the design can be seen in figure 11. Slab sections A and D were designed first due to not needing to know the final position of shear walls throughout the core of the building. The remaining sections, B, C and E, in the interior span were designed after the shear walls had been finalized.

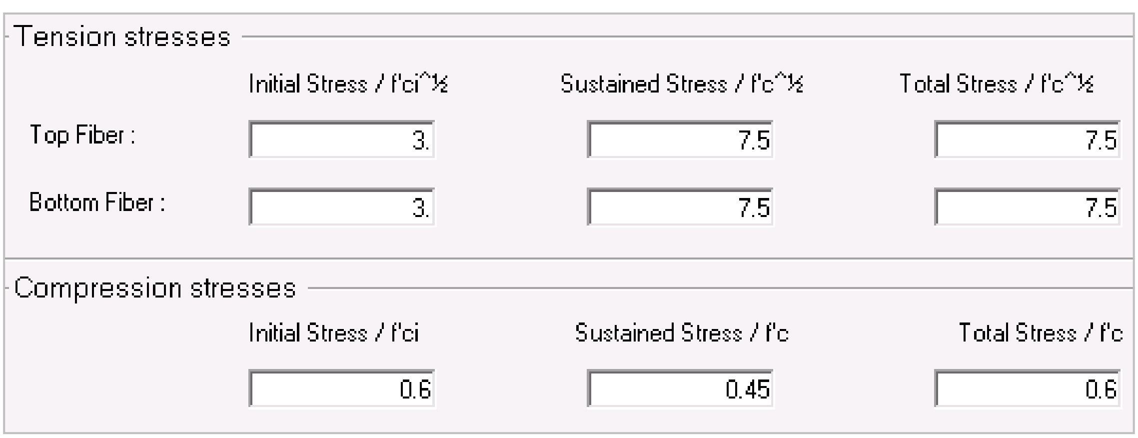


Figure 10 – PT Slab Limits

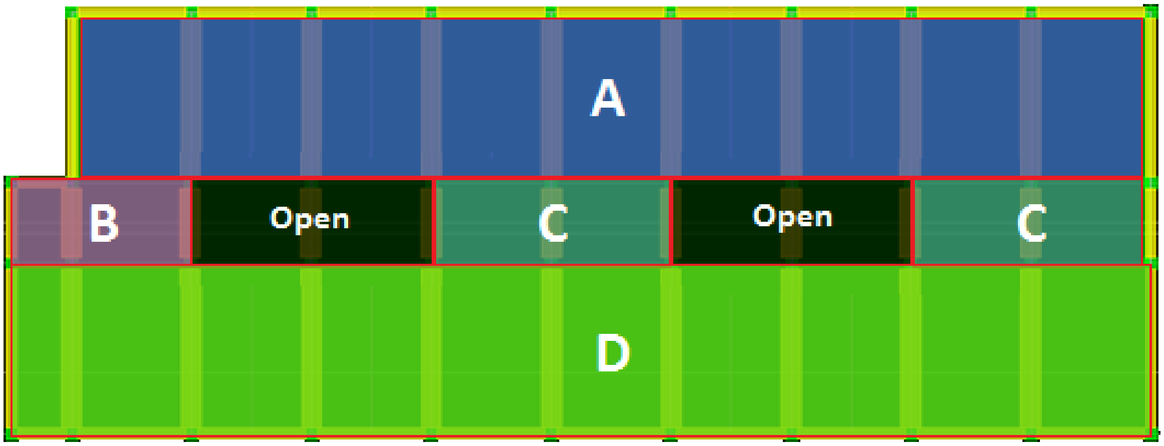


Figure 12 – Slab Zones Floors 1-14

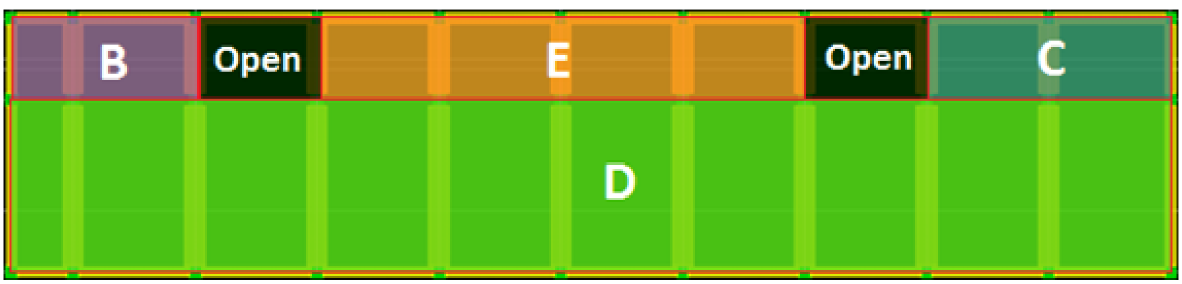


Figure 13 – Slab Zones Floors 15-23

Using the ADAPT-PT modeler, 1'-0" wide slab sections were modeled to determine the required amount of Post-Tensioned force and the number of tendons per foot. The loading for the slab sections were 100 psf live loading and 20 psf superimposed dead load. The use of live load reduction was performed for the 100 psf live load in accordance to ASCE 7-10. Due to the mix used nature of the building the original design called for a variety of 80 psf and 100 psf live loads throughout the building. For this proposal the 100 psf live load was used through the entire building to allow for a more uniform design. Rebar sizes were set to #4 throughout all slabs.

Slab Zone A Design

The design for Slab Zone A consisted of nine, 30'-0" spans. Due to larger moments on the exterior spans more post tensioning force would be required for those spans. The force applied in these bays could not exceed the pre-compression stress of 300 psi due the P/A limit. Also, the minimum average effective pre-stress had to be greater than 125 psi as per ACI 18.12.4. This resulted in the maximum allowable force of 28.8 kips to be applied at the exterior bays. Even at the maximum force the post tensioning still was not enough for the design. To increase the pre-compression tensile zone service condition limits the concrete strength was increased to 6000 psi. This resulted in the tension stress limit increasing from 530 psi to 581 psi making the design feasible. The analysis was performed again and it was found that two tendons with a force of 28.8 kips in the exterior bays along with one tendon with a force 20 kips in the interior bays were adequate for the design. Since the exterior bays require two tendons per foot, one of the tendons will have to be anchored in the adjacent interior bay. A summary of the design can be seen in Figure 14 located below with a more detailed design in appendix C.

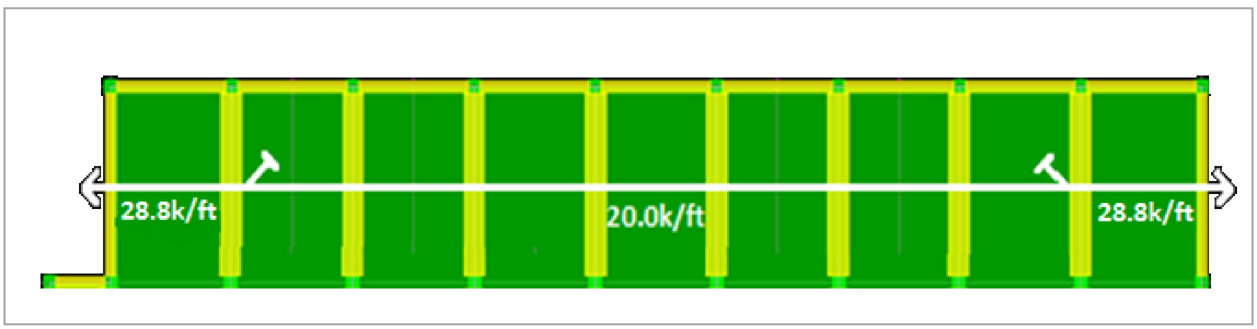


Figure 14 – Slab A Tendon Layout

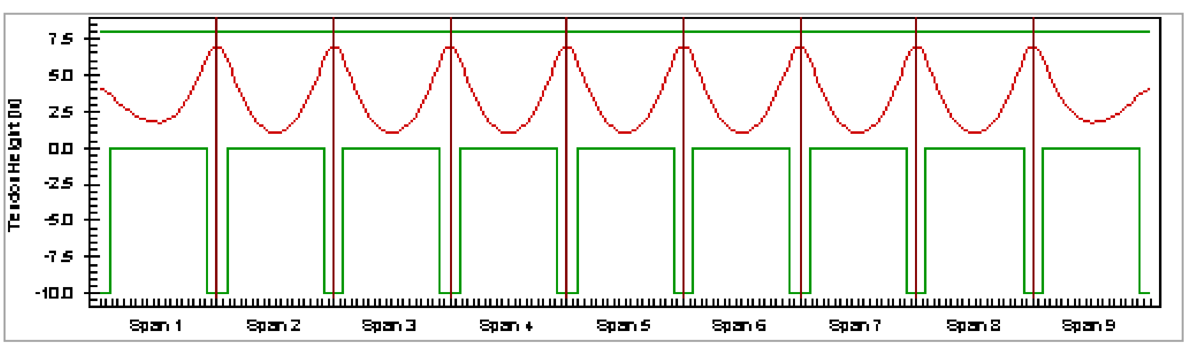


Figure 15 – Slab A Design

Deflection values were taken from the ADAPT-PT program as seen in figure 16. These values along with hand deflection calculations were compared to code limits. The values were compared to the limit, L/360, of 1". Hand and computer calculations came in well below this allowed value for all slabs.

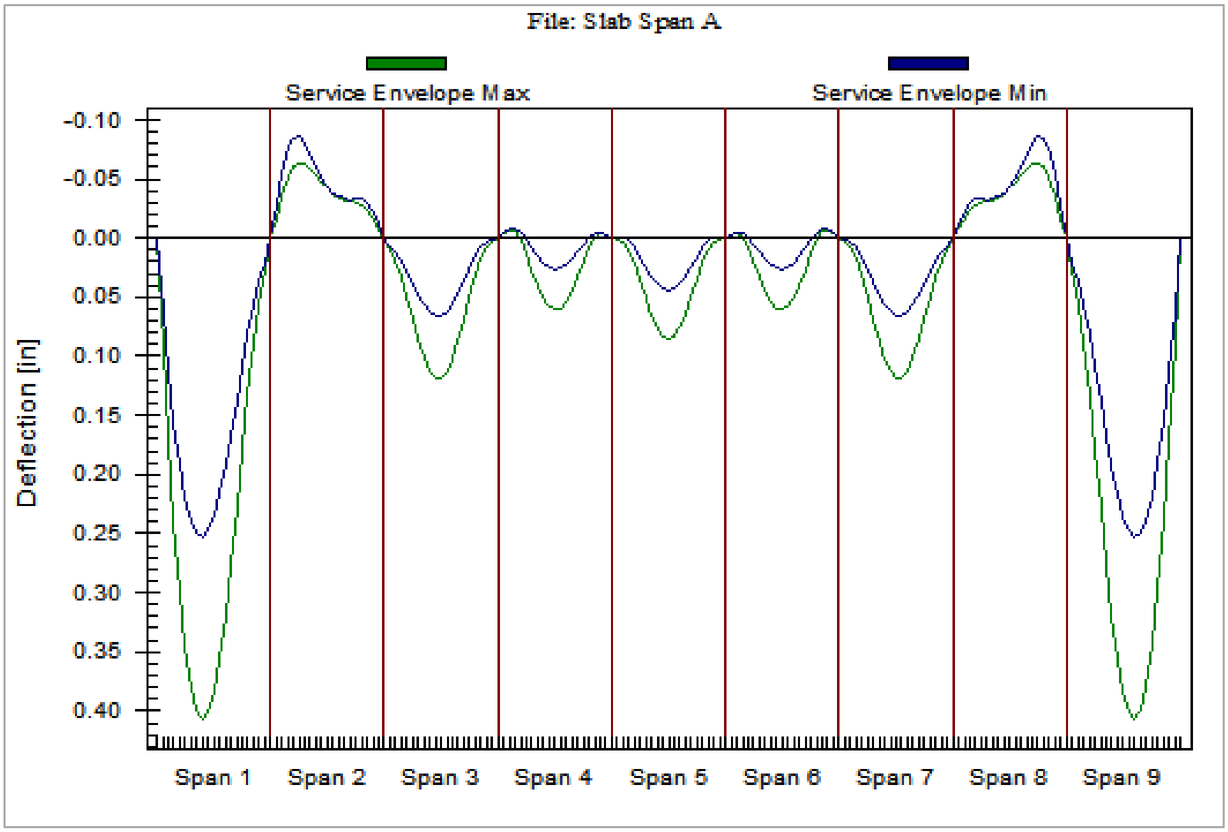


Figure 16 – Slab A Deflections

The reaming slab zones were designed following the same procedure as slab A. Each zone took several attempts to find the specific design required to meet code. The final designs are summarized in the figures below, more detailed information on each individual slab can be found in appendix C.

<u>Slab Zones B and C</u>

Both slab zones B and C were 2 span slabs located over the typically 20'-0" by 30'-0" interior bays of the building. The Slabs were sectioned off from one another to allow for the vertical circulation wells to pass through the building. The main difference between the designs for each slab was that slab B had a 15' span and a 30' span while slab C had two 30' spans. They were both modeled with 30" wide girders at the end of each span with a 60" girder located over the middle support. The smaller span in slab B made it challenging to achieve a design that passed code requirements. The stresses had to be reduced by increasing the tendon height at the center of the slab resulting in a decrease of the upward force created by the tendon to bring the stresses within the code limits. Slab C was a straight forward design due to the symmetry of the spans. Both of the slabs required 28.8 kip provided by 2 strands per foot. A summary of each design can be seen in figures 17 and 18 below with further details in appendix C.

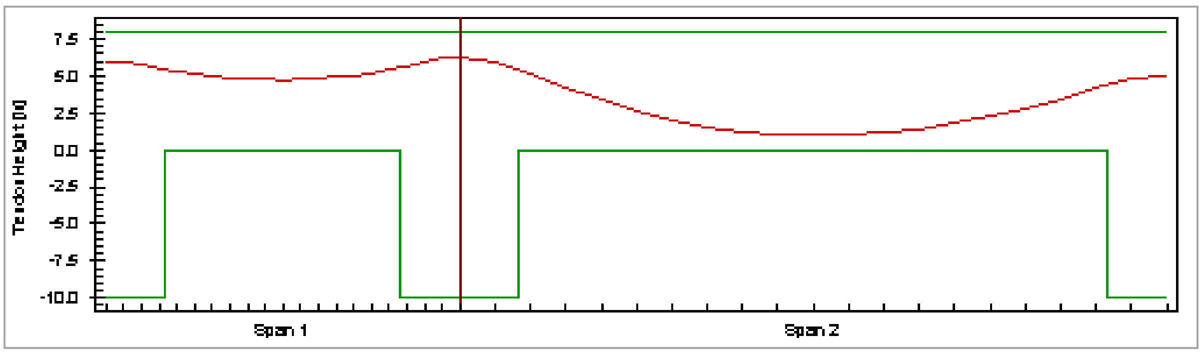


Figure 17 – Slab Zone B

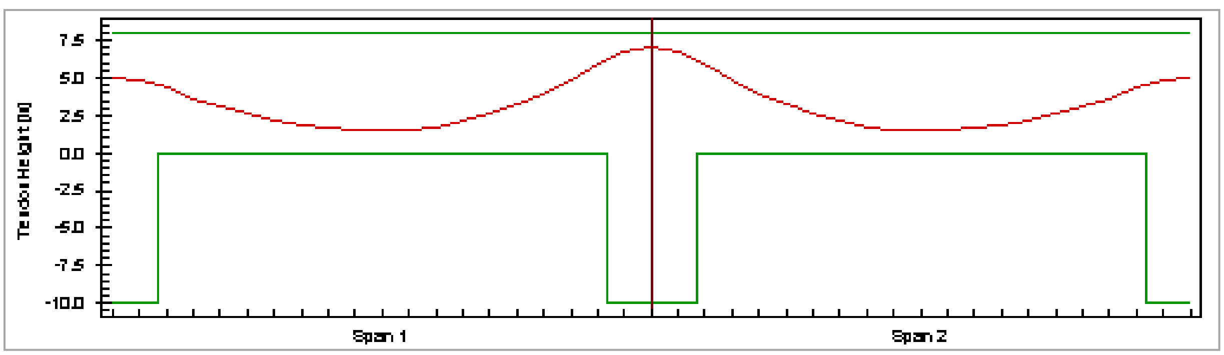


Figure 18 – Slab Zone C

Slab Zone D and E

Slab D was very similar to slab A only that it had the extra 15' span at the east end of the building making it a total of 10 spans. The forces required for the PT varies over the spans with the greatest amount on the far right span of 28.8 kip. The PT force for each span can be found in figure 19 along with the tendon layout in figure 20. The design for slab E was a relatively easy slab to design due to the symmetric spans, the layout can be seen in figure 21.

						<— Tendon Control Point Height —>		
	Number of strands	PT Force per unit	PT Force	P/A	%DL balanced	Left	Center	Right
1	1	18.0	18.0	188	124	4.00	1.75	7.00
2	1	18.0	18.0	188	57	7.00	1.00	7.00
3	1	18.0	18.0	188	57	7.00	1.00	7.00
4	1	18.0	18.0	188	57	7.00	1.00	7.00
5	1	18.0	18.0	188	57	7.00	1.00	7.00
6	1	18.0	18.0	188	57	7.00	1.00	7.00
7	1	18.0	18.0	188	57	7.00	1.00	7.00
8	1	18.0	18.0	188	57	7.00	1.00	7.00
9	1	25.0	25.0	260	79	7.00	1.00	7.00
10	2	28.8	28.8	300	57	7.00	1.75	4.00

Figure 19 – Slab Zone D Design Parameters

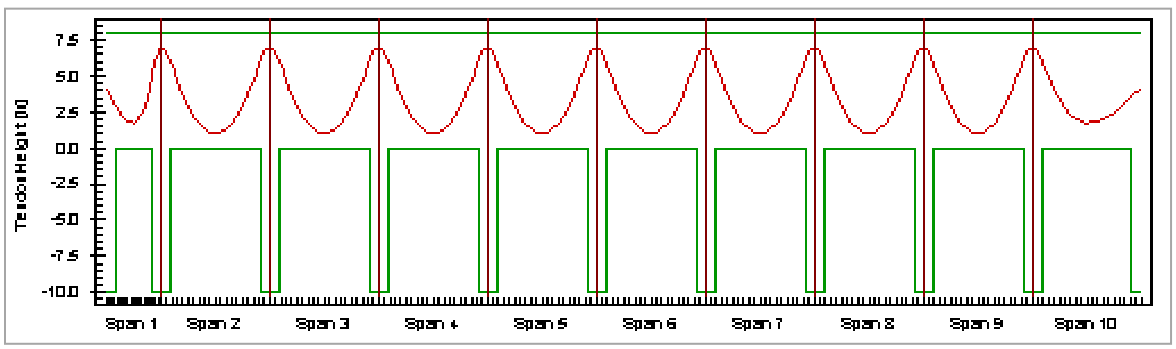


Figure 20 – Slab Zone D

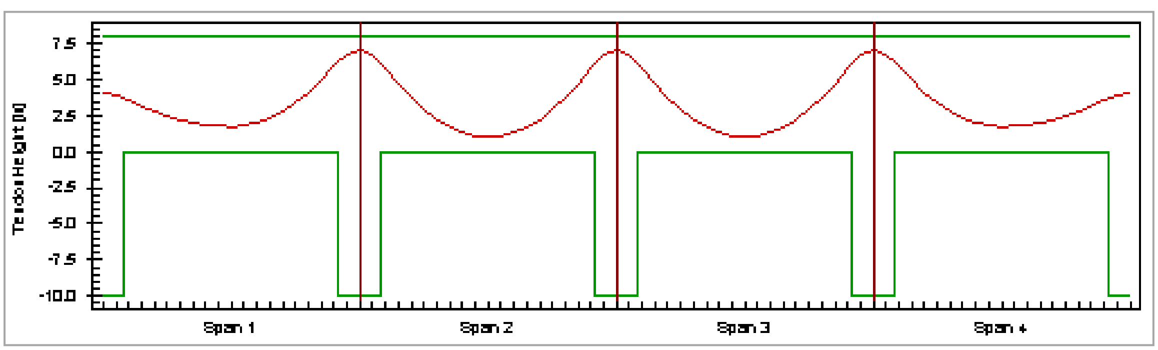


Figure 21 – Slab Zone E

Design of Post-Tensioned Girders

Wide-shallow post-tensioned girders will support the one-way slab system. They will be spanning in the short direction of the building (North to South), typically over 3 spans. The designs for the girders were broken up into separate groups by color, as seen in figure 22, to simplify the design process. This resulted in 4 different types of girders throughout the building. The main differences in the girders were the tributary area each one was designed for. This resulted in different girder sizing and post tensioning for certain spans. The initial size for the girders was 60" by 20" with the assumption that half the tributary area could be supported by half the girder width for certain girders.

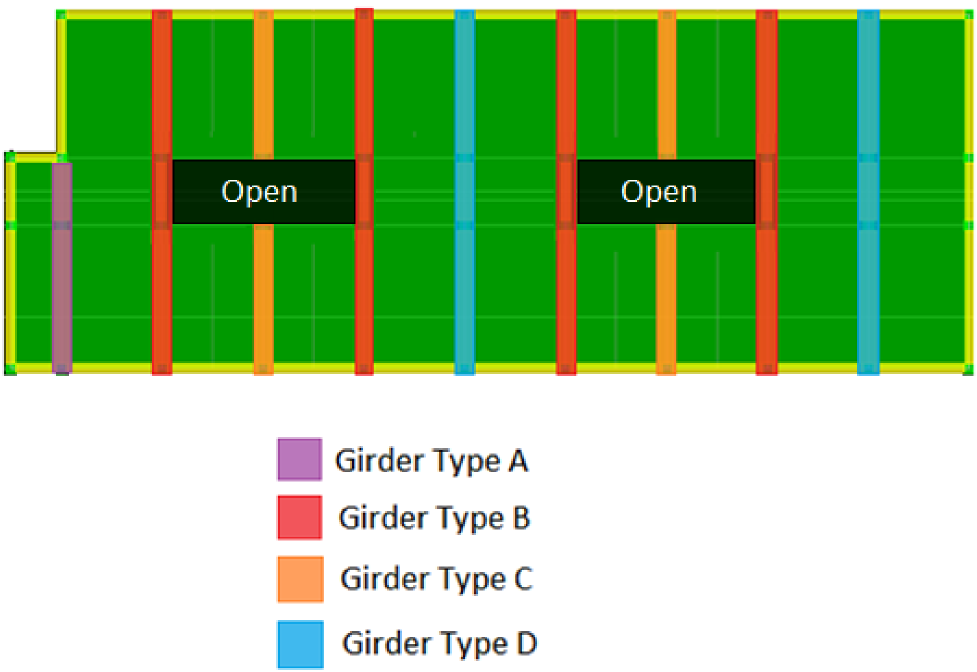


Figure 22 – Girder Types

Further research showed that Preliminary thickness can be assumed by equation, L/30, which would produce a girder depth of exactly 17". A more conservative value of 18" depth was used during design. Hand calculations can be seen in appendix C for girder type D, however due to complexity, computer analysis was used for final design and hand calculations were compared to the outputs. The girders were essentially modeled as "T-Beams" because part of the slab would act with the beam. The effective width was used during calculations as per ACI 318-8.12. Girders were designed with the same parameters that were used when designing the slabs. Live load reductions were utilized when applicable in the designs. Further details than provided below for each design can be found in appendix C.

Girder A Design

Girder A is located throughout all floors of the building. For the first 14 stories of the structure it is located at the west end, however, it becomes the predominate girder used through the remainder of the building (stories 15 to 23). The girder spans over the 20' interior bay and one of the 42.5' exterior bays. This resulted in the spans requiring different PT forces. This was accomplished by reducing the amount of tendons from one span to another span. The 20' span required a force of 440 kips provided by 17 strands with a PT force of 88.0 kips per a strand. The 42.5' span required a greater force of 740 kips which was provided by 28 strands with a force of 69.9 kips per a strand.

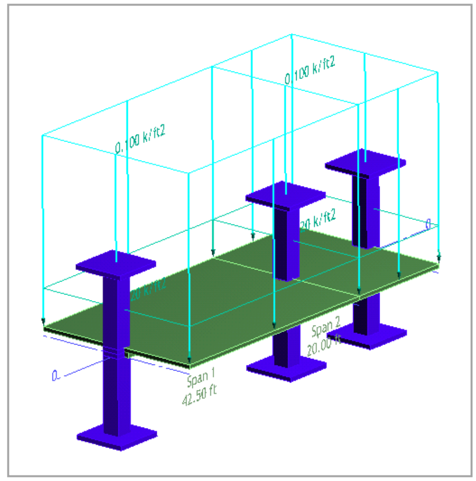


Figure 23 – Girder A Model

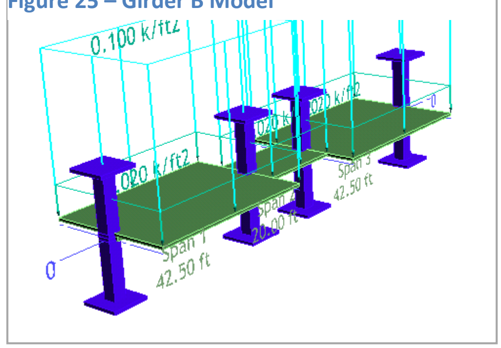
<— Tendon Control Point Height —>

	Number of strands	PT Force per unit	PT Force	P/A	%DL balanced	Left	Center	Right	s
1	28	69.6	740.0	213	81	12.50	1.75	16.00	
2	17	88.0	440.0	126	56	16.00	11.00	12.50	

Figure 24 – Girder A Design Parameters

Girder B Design

The design for girder B altered from the other girders due to the opening along one side of the mid span. Since the beam would only be required to support half of the tributary area the width was reduced to 30" instead of 60" over the mid span. Also, the beam was modeled without considerations of the effective flange width over the mid span because of the opening. The final design for the girder had much higher PT forces located in the exterior bays as expected. The summary of forces and tendon control points for the girder can be seen in figure 26.

Figure 25 – Girder B Model


<— Tendon Control Point Height —>

	Number of strands	PT Force per unit	PT Force	P/A	%DL balanced	Left	Center	Right	s
1	29	70.6	750.0	216	82	12.50	1.75	16.00	
2	10	50.0	250.0	144	54	16.00	13.25	16.00	
3	29	70.6	750.0	216	82	16.00	1.75	12.50	

Figure 26 – Girder B Design Parameters

<u>Girder C Design</u>

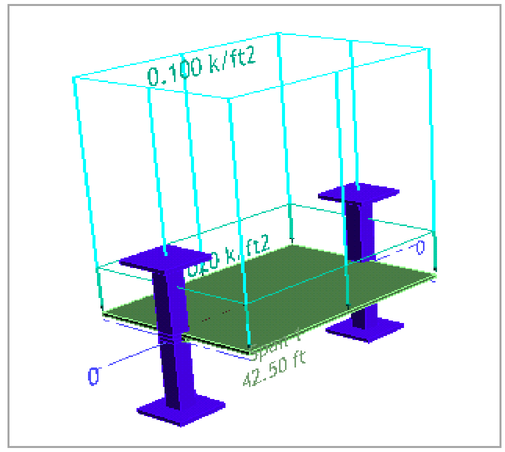


Figure 26 – Girder C Model

Girder C is a one span post tensioned girder due to the opening for the stairwells and elevator shafts. The girder did not have to deal with any forces caused by other spans allowing for a simple design. The final design for the girder consisted of 31 strands with a PT Force of 76.7 kips per a strand. This resulted in 77% of the dead load to be balanced by the post tension system. The tendon control point heights can be seen in figure 27.

						<— Tendon Control Point Height —>		
	Number of strands	PT Force per unit	PT Force	P/A	%DL balanced	Left	Center	Right
1	31	76.7	815.0	234	77	12.50	1.75	12.50

Figure 27 – Girder C Design Parameters

<u>Girder D Design</u>

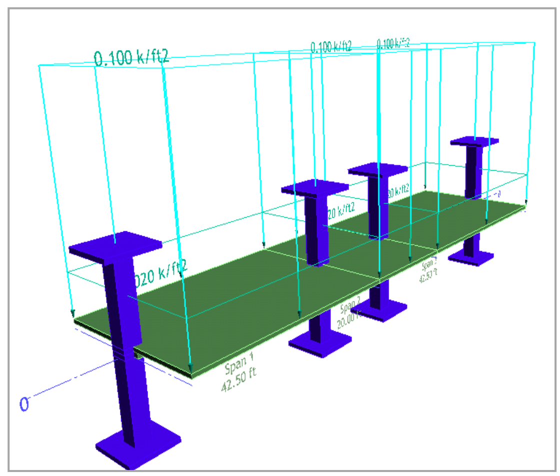


Figure 28 – Girder D Model

Girder Type D was a fairly uniform girder other than the span lengths between exterior and interior bays. The tributary area for each span was kept at 30' and effective flange width was included in the design of each span. This resulted in force of 500 kips required in the interior span and 750 kips for the exterior. Once again the tendon locations and strand information can be found below in figure 29.

						<— Tendon Control Point Height —>		
	Number of strands	PT Force per unit	PT Force	P/A	%DL balanced	Left	Center	Right
1	29	70.6	750.0	216	82	12.50	1.75	16.00
2	19	100.0	500.0	144	108	16.00	10.50	16.00
3	29	70.6	750.0	216	82	16.00	1.75	12.50

Figure 29 – Girder D Design Parameters

Design of Columns

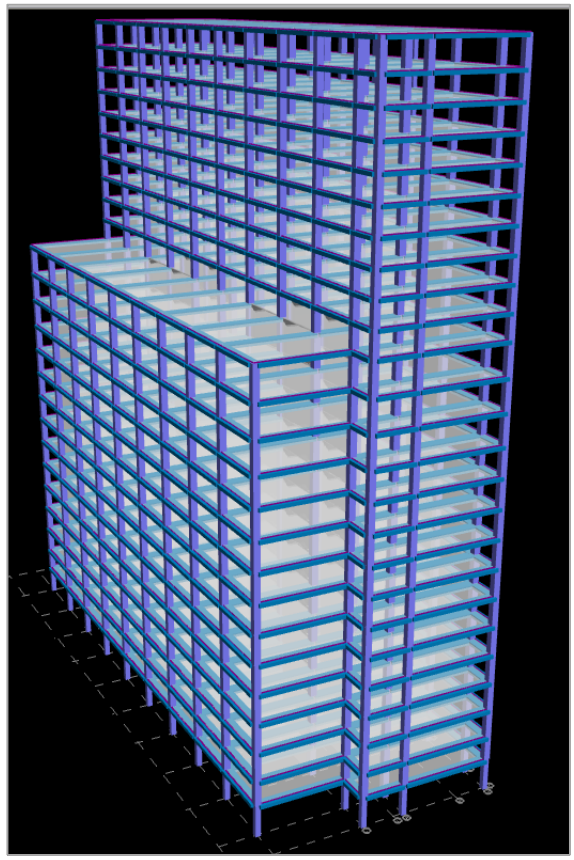


Figure 30 – RAM Model

After the slabs and girders were designed the columns could be analyzed for gravity loads. The designs for the columns were performed using a variety of programs from the RAM Structural System software package. The initial model, figure 30, was modeled in the RAM Structural Modeler portion of the program. The model was then analyzed in RAM Concrete to determine the gravity loads throughout the building. The final step was to use RAM Column to aid in the column design. The material properties of the columns were entered in the program and assigned to the columns. Once the columns were sized typical rebar patterns were assigned and the model was run to checked the columns in accordance with the newest code (ACI 318-02) provided by the program. This process was repeated several times to find the correct sizing and rebar combinations. The program provides a visual model showing which columns fail to meet required criteria by highlighting them in red. The columns can then be looked at in detail to see why they are failing or not meeting code. The columns sizes and rebar were changed until the entire structure was optimized for size while meeting code and strength requirements.

Columns were preliminarily sized at 36” by 36” which takes up 9 square feet of floor space. These sizes were pretty large and were looked at to try and reduce the footprint of the columns. The first design process showed that the loads from the building were quite substantial and that a 36” by 36” column at 6ksi concrete strength were not capable of supporting the loads on the lowest floors. They were then resized until they were able to meet the demands of the loading. This resulted in column sizes of 40” and higher throughout the bottom stories of the structure. To resolve this issue the concrete strength was increased to 10ksi for the first 14 stories of the structure then reduced to 7 ksi strength for the rest of the building. This resolved the issue and the typical size for the lower stories could stay at the original 36” by 36”. The large size of the 36” by 36” columns was not practical for all stories of the building. From a construction standpoint it did not make sense to have minute size changes ever floor. Sizing for the columns were stepped down at 6" intervals so formwork could be reused over several floors. The final design for the building has a typical size change every 6 levels, results in four different sizes of columns; 36"x36", 30"x30", 24"x24" and 18"x18". The majority of the columns in the building are reduced in size at the same time; however select columns do not follow this pattern due to increased/decreased loading. Figure 31 shows the RAM model used for the column designs. It is easy to see the size changes of each column due to the increase of stress represented by a color change. The orange columns have the largest stresses and are present typically every 6 stories. On the 15th story the concrete strength changes from 10ksi to 7ksi which is also visible by the stress changes.

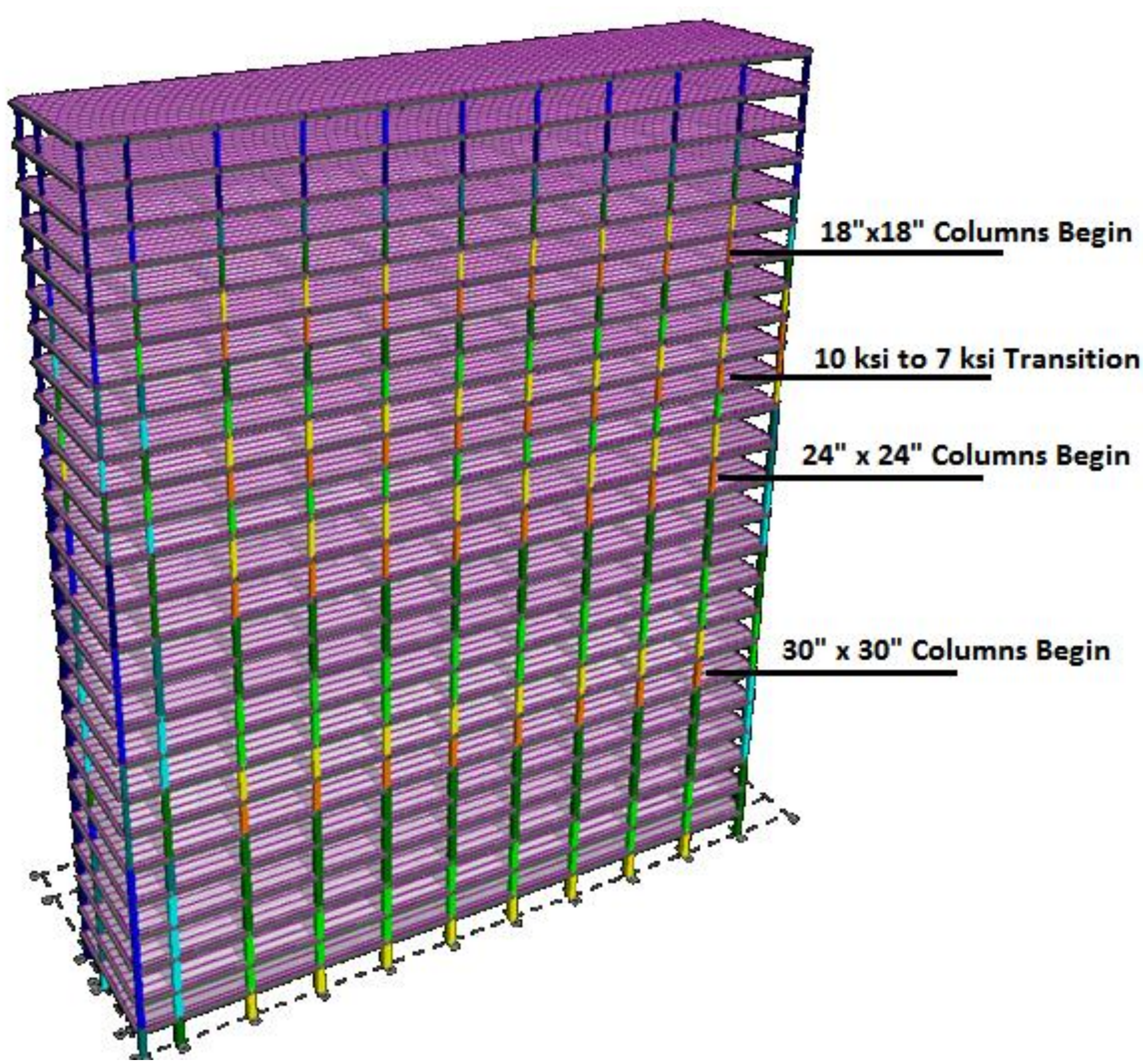


Figure 31 – RAM Concrete Column Design Model

Design of Proposed Lateral System

Design Process

Load Combinations provided by ASCE 7-10 for strength design are:

1. 1.4(D)
2. 1.2(D) + 1.6(L) + .5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)
4. 1.2D + 1.0W + L + .5(Lr or S or R)
5. 1.2D + 1.0E + L +.2S
6. .9D + 1.0W
7. .9D + 1.0E

For the analysis of the lateral system only load combinations that included lateral forces were explored. This would result in load combinations 4 and 5 being used for the general loading and combinations 6 and 7 for uplift.

The existing structure of Three PNC Plaza utilized a steel moment frame with concrete core walls throughout the middle to resist lateral forces. The same basic concept was used during design of the new lateral system. However, the existing core walls were located in 7 distinct sections as seen in figure 32. The original placement of these walls did not fare well with the new floor slab chosen. It would have been very challenging to accommodate all of the openings and small slab sections in between the cores. To resolve this problem the core walls were shifted into 2 larger sections resulting in only 2 openings in the slab seen in figure 33. These shear walls would follow the original design and continue from the lower levels up until the 14th floor. Once the 14th floor is reached the moment frame will take over for the remainder of the building similar to the original design.

Figure 32 – Existing Shear Walls

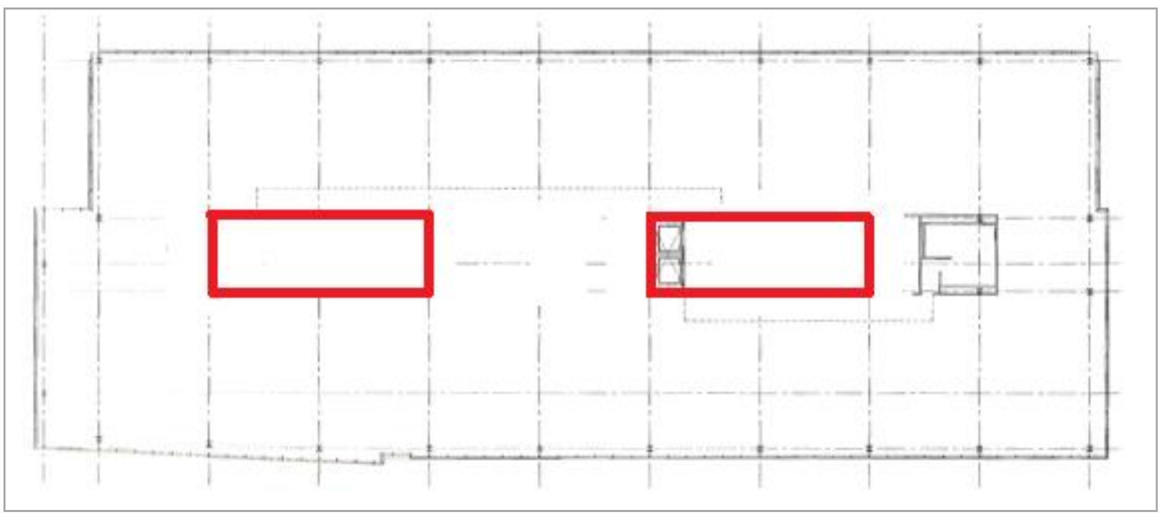


Figure 33 – Proposed Shear Walls

Loads placed on the building will travel through the structure laterally and vertically until they reach the ground. These loads will be resisted by the lateral system elements depending on the individual elements relative stiffness to the whole system. The elements resisting higher portions of the load will correspond with higher relative stiffness's. This design will resist the lateral loads resulting from seismic and wind forces by the transfer of load through the floor diaphragms to the 9 post tensioned girders, edge beams, building columns and the 2 core shear wall systems.

The code was vague on how exactly the post tensioned slab would be accounted for in the lateral system. The ETABS Model was modeled without the slab portion of the beams to be conservative due to not knowing the full interaction of the post-tensioned slab/beam system. This results in a more conservative design since the model is not taking into account the full portion of the beams. The lateral system used in the building will be both concrete moment frames and concrete shear walls. A Response Modification Coefficient for the building would fall under the dual system classification in table 12.2-1 of ASCE-7. This means the moment frame portion of the building needs to be capable of supporting 25% of the seismic loading. The building was modeled and analyzed for both the concrete moment frame without shear walls and the moment frame with the shear walls as seen in figures 34 and 35.

ETABS Model design assumptions

- Floor slabs were modeled as diaphragm elements with rigid properties at each level
- Joint/Points located on the ground level were restrained in all 6 degrees of freedom
- All structural elements were modeled without mass properties
- Shear walls were meshed into 24" by 24" areas
- Columns and Beams were modeled as line elements
- Moment of inertia of columns were reduced to 0.7I
- Rigid end offsets were modeled with a multiplier of 1
- Seismic Loads were applied at the center-of-mass

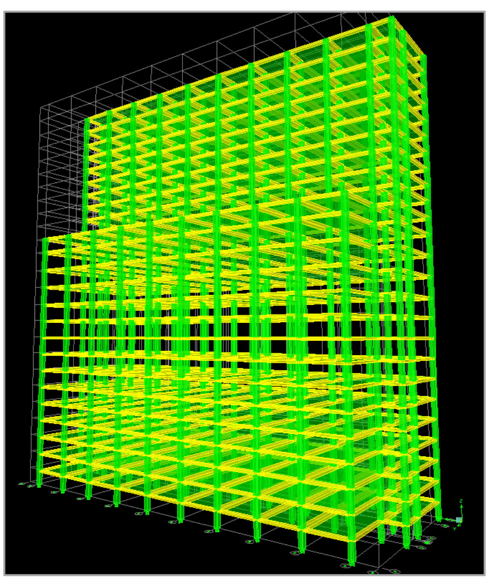


Figure 34 – Moment Frame Structure

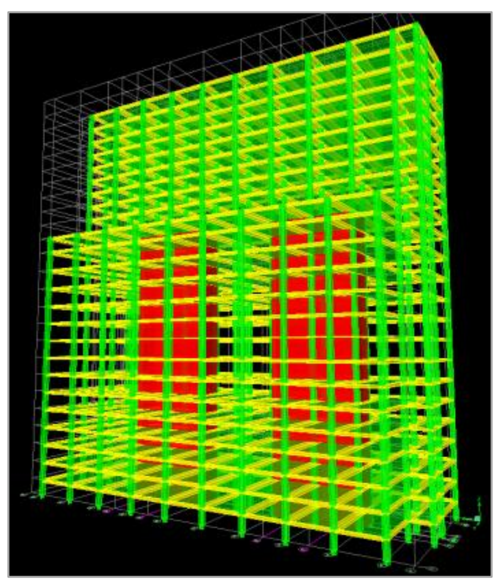


Figure 35 – Moment Frame and Shear Walls

Wind Loads

Wind Load Design Criteria		
Category		
Basic Wind Speed	V	120
Importance Factor	I	
Exposure Category	-	B
Directionality Factor	K_d	0.85
Topographic Factor	k_{zt}	1
Intensity of Turbulence	I_z	0.2238
Integral Length of Scale of Turbulence	L_z	574.945
Background Response Factor	Q	0.7766
Gust Effect Factor	G_f	0.8231
	GC_{pi}	+/- 0.18
Windward Pressure	C_p	0.8
Leeward Pressure	C_p	-0.5

East/West

Story	Height (ft)	kz or kh	qz	Windward (psf)	Windward (plf)	Windward (kips)	Leeward (psf)	Leeward (plf)	Leeward (kips)	Story Force (kips)	Moment (k-ft)
Building Portion A											
1mezz	12.50	0.57	17.86	20.43	2507.65	15.68	-19.16	-2351.79	-14.11	29.79	372.42
2.00	24.00	0.65	20.43	22.23	2729.22	16.42	-19.16	-2351.79	-14.70	31.12	746.86
2mezz	37.50	0.75	23.34	24.21	2971.40	19.24	-19.16	-2351.79	-15.87	35.11	1316.78
3.00	51.00	0.81	25.51	25.64	3146.94	20.65	-19.16	-2351.79	-15.87	36.52	1862.72
4.00	64.50	0.87	27.20	26.74	3282.15	21.70	-19.16	-2351.79	-15.87	37.57	2423.44
5.00	78.00	0.92	28.89	27.84	3417.56	22.61	-19.16	-2351.79	-15.87	38.49	3001.92
6.00	91.50	0.96	30.22	28.70	3522.74	23.42	-19.16	-2351.79	-15.87	39.30	3595.78
7.00	105.00	1.00	31.41	29.46	3616.33	24.09	-19.16	-2351.79	-15.87	39.97	4196.74
8.00	118.50	1.04	32.47	30.13	3699.01	24.69	-19.16	-2351.79	-15.87	40.56	4806.82
9.00	132.00	1.07	33.53	30.81	3781.94	25.25	-19.16	-2351.79	-15.87	41.12	5428.21
10.00	145.50	1.10	34.50	31.43	3857.84	25.78	-19.16	-2351.79	-15.87	41.66	6061.37
11.00	159.00	1.13	35.35	31.96	3923.44	27.24	-19.16	-2351.79	-16.46	43.71	6949.13
12.00	173.50	1.16	36.25	32.54	3994.06	28.70	-19.16	-2351.79	-17.05	45.75	7937.87
13.00	188.00	1.18	37.04	33.03	4054.39	29.68	-19.16	-2351.79	-17.34	47.03	8841.06
Building Portion B											
14.00	203.00	1.20	37.75	34.23	2139.39	21.35	-15.41	-962.85	-11.59	32.94	6687.25
15.00	214.50	1.22	38.33	34.58	2161.05	12.36	-15.41	-962.85	-5.54	17.90	3839.59
16.00	226.00	1.24	38.90	34.93	2182.83	12.49	-15.41	-962.85	-5.54	18.03	4073.67
17.00	237.50	1.26	39.48	35.28	2204.71	12.61	-15.41	-962.85	-5.54	18.15	4310.76
18.00	249.00	1.28	40.06	35.63	2226.67	12.74	-15.41	-962.85	-5.54	18.28	4550.88
19.00	260.50	1.29	40.57	35.93	2245.86	12.86	-15.41	-962.85	-5.54	18.39	4791.88
20.00	272.00	1.31	41.07	36.24	2264.86	12.97	-15.41	-962.85	-5.54	18.50	5033.28
21.00	283.50	1.33	41.58	36.54	2283.92	13.08	-15.41	-962.85	-5.54	18.61	5277.11
22.00	295.00	1.34	42.08	36.85	2303.04	13.19	-15.41	-962.85	-5.54	18.72	5523.56
23.00	306.50	1.36	42.55	37.13	2320.47	13.87	-15.41	-962.85	-5.78	19.65	6022.68
Roof Main	319.00	1.37	43.02	37.41	2338.01	20.70	-15.41	-962.85	-8.55	29.24	9327.67
Roof High	342.00	1.40	43.88	37.93	2370.44	13.44	-15.41	-962.85	-5.54	18.98	6491.15
									Sum=	795.11	123470.60

North/South

Story	Height (ft)	kz or kh	qz	Windward (psf)	Windward (plf)	Windward (kips)	Leeward (psf)	Leeward (plf)	Leeward (kips)	Story Force (kips)	Moment (k-ft)
Building Portion A											
1mezz	12.50	0.57	17.86	19.52	5796.52	36.34	-25.96	-7711.17	-46.27	82.61	1032.57
2.00	24.00	0.65	20.43	21.34	6338.93	38.06	-25.96	-7711.17	-48.19	86.25	2070.09
2mezz	37.50	0.75	23.34	23.33	6929.32	44.78	-25.96	-7711.17	-52.05	96.83	3631.15
3.00	51.00	0.81	25.51	24.79	7362.97	48.24	-25.96	-7711.17	-52.05	100.29	5114.63
4.00	64.50	0.87	27.20	25.93	7699.99	50.84	-25.96	-7711.17	-52.05	102.89	6636.27
5.00	78.00	0.92	28.89	27.05	8034.75	53.10	-25.96	-7711.17	-52.05	105.16	8202.10
6.00	91.50	0.96	30.22	27.94	8297.13	55.12	-25.96	-7711.17	-52.05	107.17	9806.10
7.00	105.00	1.00	31.41	28.72	8530.75	56.79	-25.96	-7711.17	-52.05	108.84	11428.67
8.00	118.50	1.04	32.47	29.42	8737.44	58.28	-25.96	-7711.17	-52.05	110.33	13074.17
9.00	132.00	1.07	33.53	30.11	8943.56	59.67	-25.96	-7711.17	-52.05	111.72	14747.54
10.00	145.50	1.10	34.50	30.75	9132.28	61.01	-25.96	-7711.17	-52.05	113.06	16449.70
11.00	159.00	1.13	35.35	31.30	9295.99	64.52	-25.96	-7711.17	-53.98	118.50	18841.12
12.00	173.50	1.16	36.25	31.89	9471.48	68.03	-25.96	-7711.17	-55.91	123.94	21503.25
13.00	188.00	1.18	37.04	32.40	9622.04	70.42	-25.96	-7711.17	-56.87	127.29	23929.89
Building Portion B											
14.00	203.00	1.20	37.75	32.89	9767.96	64.17	-25.96	-7711.17	-51.09	115.25	23396.16
15.00	214.50	1.22	38.33	33.26	9878.22	56.48	-25.96	-7711.17	-44.34	100.82	21626.32
16.00	226.00	1.24	38.90	33.63	9988.37	57.12	-25.96	-7711.17	-44.34	101.46	22928.98
17.00	237.50	1.26	39.48	34.00	10098.42	57.75	-25.96	-7711.17	-44.34	102.09	24246.07
18.00	249.00	1.28	40.06	34.37	10208.37	58.38	-25.96	-7711.17	-44.34	102.72	25577.59
19.00	260.50	1.29	40.57	34.70	10305.32	58.98	-25.96	-7711.17	-44.34	103.32	26913.84
20.00	272.00	1.31	41.07	35.02	10400.97	59.53	-25.96	-7711.17	-44.34	103.87	28252.59
21.00	283.50	1.33	41.58	35.34	10496.55	60.08	-25.96	-7711.17	-44.34	104.42	29602.95
22.00	295.00	1.34	42.08	35.66	10592.07	60.63	-25.96	-7711.17	-44.34	104.97	30965.85
23.00	306.50	1.36	42.55	35.96	10679.54	63.83	-25.96	-7711.17	-46.27	110.09	33743.44
Roof Main	319.00	1.37	43.02	36.26	10767.90	95.29	-25.96	-7711.17	-68.44	163.73	52228.48
Roof High	342.00	1.40	43.88	36.80	10930.35	61.92	-25.96	-7711.17	-44.34	106.25	36339.10
									Sum=	2813.86	512288.61

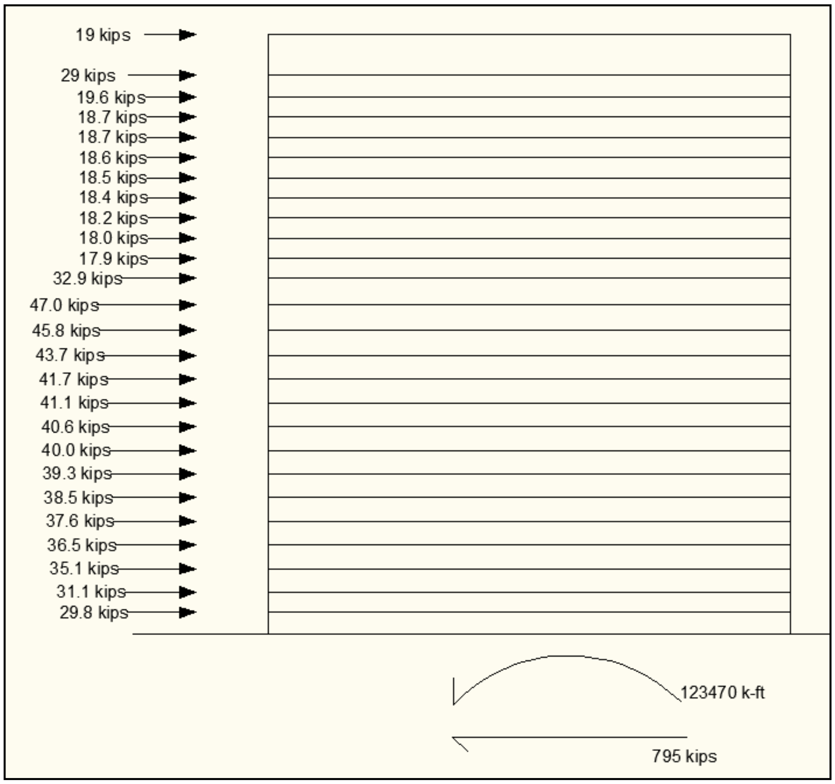


Figure 36 – East/West Wind Base Shear

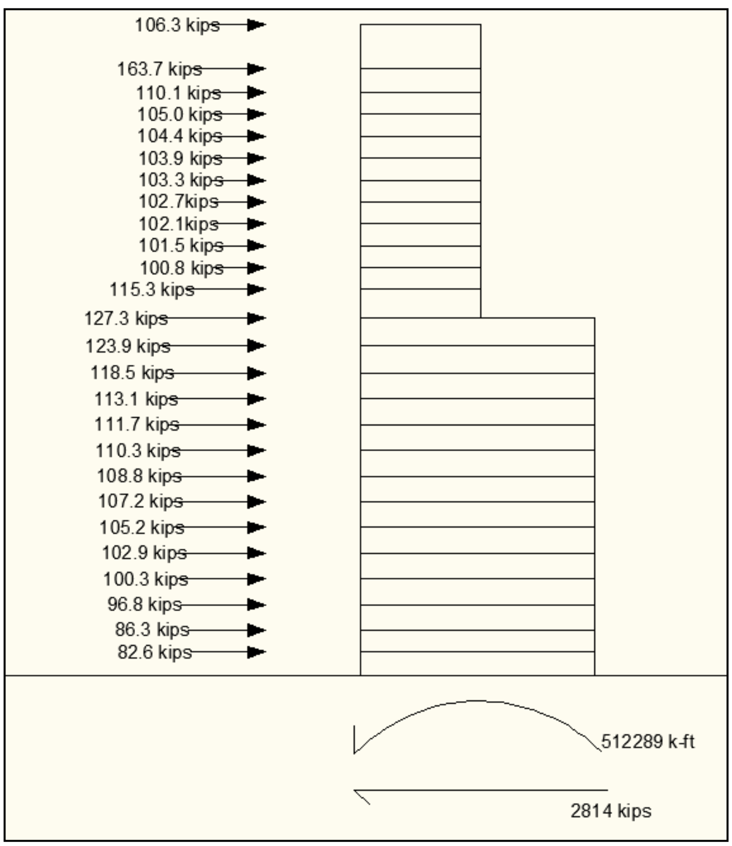


Figure 37 – North/South Wind Base Shear

Seismic Loads

Seismic calculations for the new concrete Three PNC Plaza structure were determined using the Equivalent Lateral Force Procedure according to ASCE 7-10 section 12.8. To aid in these calculations some of the seismic design parameters for Pittsburgh, PA were found from the United States Geological Survey website using the Ground Motion Parameter Application. The building would be classified as stated earlier under E.8 in table 12.2-1; Dual System with intermediate moment frames capable of resisting at least 25% of seismic forces along with ordinary reinforced concrete shear walls. The design of the lateral force resistance system resulted in the Response Modification Coefficient of 5.5 to be used. The output for the ETABS models put the period of vibration to be 3.2 seconds. This value was not able to be used during calculations due the code value of T_a*C_u=2.114 seconds for the fundamental period of vibration being lower. This means the modeled building is more flexible then the code limits permit resulting in the calculations taking into account a more rigid structure producing larger forces. The building weight was calculated to find the base shear force from the equation V=Cs (W). The minimum value of Cs was found to be 0.023 resulting in 2.3% of the building weight or 2390.3 kips as the base shear. After the base shear forces were calculated the vertical distribution of the seismic forces could be calculated as according to ASCE 7-10 section 12.8.3. The calculations relied heavily on Microsoft excel and can be seen below in the tables provided.

Seismic Design Criteria			
Seismic Use Group		III	
Site Class		D	Provided
Seismic Design Category		B	Table 11.6-1
Importance Factor	I_e	1.25	Table 11.5-1
Spectral Response Acceleration (Short)	S_s	0.201	USGS
Spectral Response Acceleration (1s)	S_1	0.118	USGS
Site Coefficient	F_a	1.6	Table 11.4-1
	F_v	2.328	table 11.4-2
Soil Modified Acceleration	S_{MS}	0.3216	Calculated
	S_{M1}	0.2747	Calculated
Design Spectral Response (Short)	S_{DS}	0.2144	Calculated
Design Spectral Response (1s)	S_{D1}	0.1831	Calculated
Response Modification Coefficient	R_x	5.5	Table 12.2-1
	R_y	5.5	Table 12.2-1
Approximate Period Parameter	C_t	0.02	Table 12.8-2
Building Height	h_n	319	
Approximate Period Parameter	x	0.75	Table 12.8-2
Approximate Fundamental Period	T_a	1.51	Table 12.8-8
Period Upper Limit Coefficient	C_u	1.4	Table 12.8-1
Long Period Transition Period	T_L	12	Figure 22-12

Floor Level	Floor Height (ft)	Total Height (ft)	Weight (kips)	$w*h^k$	C_{VX}	f_i (kips)	V_i (kips)	M_z (k-ft)	25% f_i (kips) (MF only Load)
Main Roof	23	319.0	3200	18765276	0.080	191	191	60831	48
23	12.5	306.5	3200	17669646	0.075	180	370	55035	45
22	11.5	295.0	3200	16681385	0.071	170	540	50007	42
21	11.5	283.5	3200	15712393	0.067	160	699	45266	40
20	11.5	272.0	3200	14763053	0.063	150	849	40806	38
19	11.5	260.5	3200	13833772	0.059	141	990	36621	35
18	11.5	249.0	3200	12924983	0.055	131	1121	32704	33
17	11.5	237.5	3200	12037151	0.051	122	1244	29051	31
16	11.5	226.0	3200	11170772	0.047	114	1357	25655	28
15	11.5	214.5	3200	10326379	0.044	105	1462	22509	26
14	11.5	203.0	4900	14553841	0.062	148	1610	30023	37
13	15	188.0	4900	12965929	0.055	132	1742	24771	33
12	14.5	173.5	4900	11490576	0.049	117	1859	20259	29
11	14.5	159.0	4900	10076246	0.043	102	1961	16281	26
10	13.5	145.5	4900	8816676	0.037	90	2051	13036	22
9	13.5	132.0	4900	7614824	0.032	77	2128	10214	19
8	13.5	118.5	4900	6473545	0.028	66	2194	7795	16
7	13.5	105.0	4900	5396174	0.023	55	2249	5758	14
6	13.5	91.5	4900	4386669	0.019	45	2293	4079	11
5	13.5	78.0	4900	3449836	0.015	35	2328	2734	9
4	13.5	64.5	4900	2591692	0.011	26	2355	1699	7
3	13.5	51.0	4900	1820075	0.008	18	2373	943	5
2mezz	13.5	37.5	4000	935356	0.004	10	2383	356	2
2	13.5	24.0	4900	585348	0.002	6	2388	143	1
1mezz	11.5	12.5	4000	179023	0.001	2	2390	23	0
Ground	12.5	0.0	4900	0	0.000	0	2390	0	0
		⍰	103700	235220621	1	2390		536598	

T=	1.510	s
k=	1.505	
V_b=	2390.3	kips

Drift and Displacement

The serviceability limits for story displacement and drift were calculated from the ETABS model. The code does not address drift due to wind, but is typically limited to L/400 as a standard practice. Seismic drift was calculated in accordance with ASCE 7-10. The results show that both the extreme load cases for each direction were under the limits for total deflection. Story drift values for wind in the N/S direction did come very close to limits at some levels and even exceed them. This was not addressed because the values were very close and should not be an issue. The tables below show the deflection values for the controlling load case in both X and Y directions. Figure 38 shows the maximum deflections experienced by the building by the wind load acting on the Y (North/South) direction of the building. Please note the model deflections are not to scale and are greatly exaggerated for a visual inspection.

Maximum Lateral Displacement in Y Direction (North South) due to wind load						
Floor Level	**Floor Height (ft)**	**Total Height (ft)**	**Displacement (in)**	**Story Drift (in)**	**Allowable Displacement (in) L/400**	**Allowable Story Drift (in)**
Main Roof	23	319.0	5.93	0.2	9.57	0.375
23	12.5	306.5	5.73	0.24	9.195	0.345
22	11.5	295.0	5.49	0.24	8.85	0.345
21	11.5	283.5	5.25	0.27	8.505	0.345
20	11.5	272.0	4.98	0.3	8.16	0.345
19	11.5	260.5	4.68	0.33	7.815	0.345
18	11.5	249.0	4.35	0.35	7.47	0.345
17	11.5	237.5	4.00	0.37	7.125	0.345
16	11.5	226.0	3.63	0.38	6.78	0.345
15	11.5	214.5	3.25	0.33	6.435	0.345
14	11.5	203.0	2.92	0.22	6.09	0.45
13	15	188.0	2.70	0.3	5.64	0.435
12	14.5	173.5	2.40	0.28	5.205	0.435
11	14.5	159.0	2.12	0.27	4.77	0.405
10	13.5	145.5	1.85	0.25	4.365	0.405
9	13.5	132.0	1.60	0.24	3.96	0.405
8	13.5	118.5	1.36	0.23	3.555	0.405
7	13.5	105.0	1.13	0.22	3.15	0.405
6	13.5	91.5	0.91	0.2	2.745	0.405
5	13.5	78.0	0.71	0.19	2.34	0.405
4	13.5	64.5	0.52	0.16	1.935	0.405
3	13.5	51.0	0.36	0.14	1.53	0.405
2mezz	13.5	37.5	0.22	0.11	1.125	0.405
2	13.5	24.0	0.11	0.076	0.72	0.345
1mezz	11.5	12.5	0.034	0.034	0.375	0.375
Ground	12.5	0.0	0.00	0	0	0

Maximum Lateral Displacement in X Direction (East West) due seismic load						
Floor Level	**Floor Height (ft)**	**Total Height (ft)**	**Displacement (in)**	**Story Drift (in)**	**Allowable Displacement (in) 0.015H**	**Allowable Story Drift (in) 0.015H**
Main Roof	23	319.0	2.56	0.09	53.592	2.25
23	12.5	306.5	2.47	0.13	51.492	2.07
22	11.5	295.0	2.34	0.16	49.56	2.07
21	11.5	283.5	2.18	0.19	47.628	2.07
20	11.5	272.0	1.99	0.23	45.696	2.07
19	11.5	260.5	1.76	0.26	43.764	2.07
18	11.5	249.0	1.50	0.28	41.832	2.07
17	11.5	237.5	1.22	0.28	39.9	2.07
16	11.5	226.0	0.94	0.28	37.968	2.07
15	11.5	214.5	0.66	0.2	36.036	2.07
14	11.5	203.0	0.46	0.04	34.104	2.7
13	15	188.0	0.42	0.05	31.584	2.61
12	14.5	173.5	0.37	0.04	29.148	2.61
11	14.5	159.0	0.33	0.05	26.712	2.43
10	13.5	145.5	0.28	0.04	24.444	2.43
9	13.5	132.0	0.24	0.03	22.176	2.43
8	13.5	118.5	0.21	0.04	19.908	2.43
7	13.5	105.0	0.17	0.03	17.64	2.43
6	13.5	91.5	0.14	0.03	15.372	2.43
5	13.5	78.0	0.11	0.03	13.104	2.43
4	13.5	64.5	0.08	0.03	10.836	2.43
3	13.5	51.0	0.05	-0.25	8.568	2.43
2mezz	13.5	37.5	0.30	0.1	6.3	2.43
2	13.5	24.0	0.20	0.19	4.032	2.07
1mezz	11.5	12.5	0.01	0.01	2.1	2.25
Ground	12.5	0.0	0	0	0	0

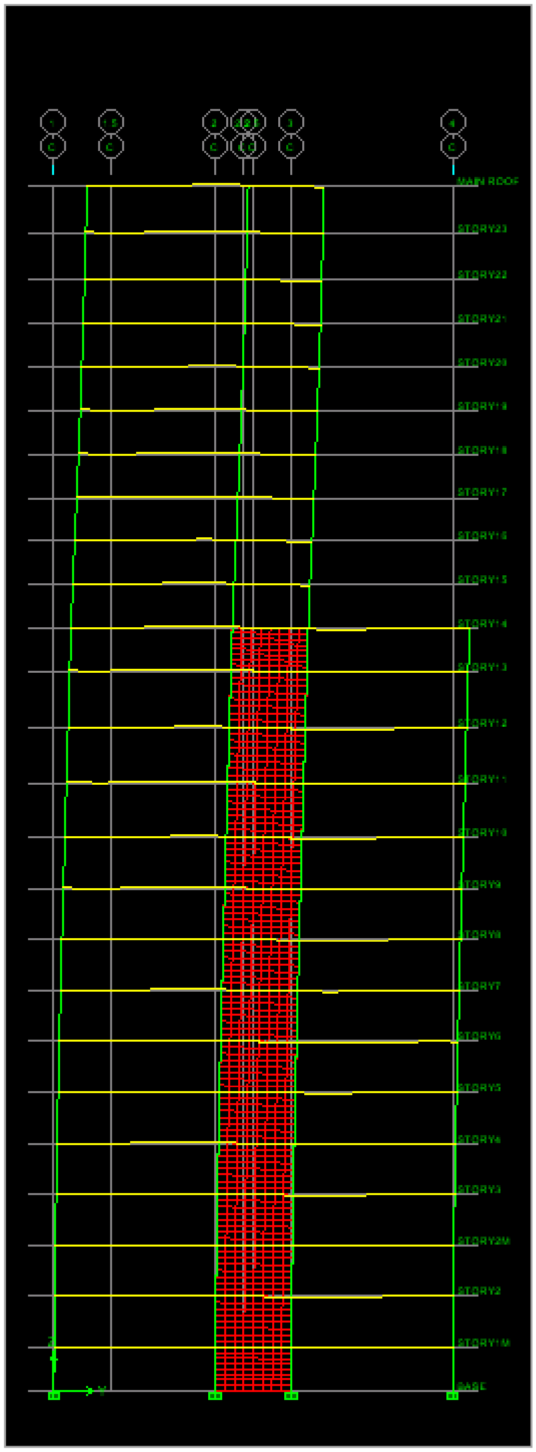


Figure 38 – Max Building Deflection Diagram (North/South)

Column Check for Gravity and Lateral

The column designs from the RAM outputs needed to be rechecked with account for the lateral loads. A sample calculation performed with excel can be found in the tables below for an internal column. The lateral forces the column will experience were found from the ETABS output and both axial and lateral forces were summarized below. Finally the forces were entered into the SP Column program to be analyzed. The lowest portion of the column was checked along with the column on floor level 15. Column 15 is a key point due the shear walls no longer being present to resist lateral loads resulting in much larger moments on the column. Both designs passed inspection along with the other levels resulting in the RAM rebar outputs being used. The rebar tables can be seen for this particular column in appendix E along with several other columns.

Column Check										
Floor Level	Tributary Area	Dead Load	Live Load (Reduced)	Superimposed Dead Load	Total Dead Load	Snow Load	1.4DL	1.2D + 1.6L	1.2DL + 1.6S + L	Total Load
Main Roof	937	93700	46381.5	18740	112440	16866	157416	209138.4	208295.1	209138.4
23	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	418276.8
22	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	627415.2
21	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	836553.6
20	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	1045692
19	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	1254830.4
18	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	1463968.8
17	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	1673107.2
16	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	1882245.6
15	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	2091384
14	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	2300522.4
13	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	2509660.8
12	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	2718799.2
11	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	2927937.6
10	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	3137076
9	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	3346214.4
8	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	3555352.8
7	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	3764491.2
6	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	3973629.6
5	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	4182768
4	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	4391906.4
3	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	4601044.8
2mezz	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	4810183.2
2	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	5019321.6
1mezz	937	93700	46381.5	18740	112440	0	157416	209138.4	181309.5	5228460

Interior Column Loading		
Floor Level	**Axial (Calculated)**	**Moment (ETABS)**
23	418.28	93
22	627.42	134
21	836.55	176
20	1045.69	222
19	1254.83	270
18	1463.97	320
17	1673.11	372
16	1882.25	458
15	2091.38	507
14	2300.52	134
13	2509.66	175
12	2718.80	179
11	2927.94	184
10	3137.08	192
9	3346.21	194
8	3555.35	194
7	3764.49	193
6	3973.63	189
5	4182.77	182
4	4391.91	173
3	4601.04	160
2mezz	4810.18	145
2	5019.32	111
1mezz	5228.46	178

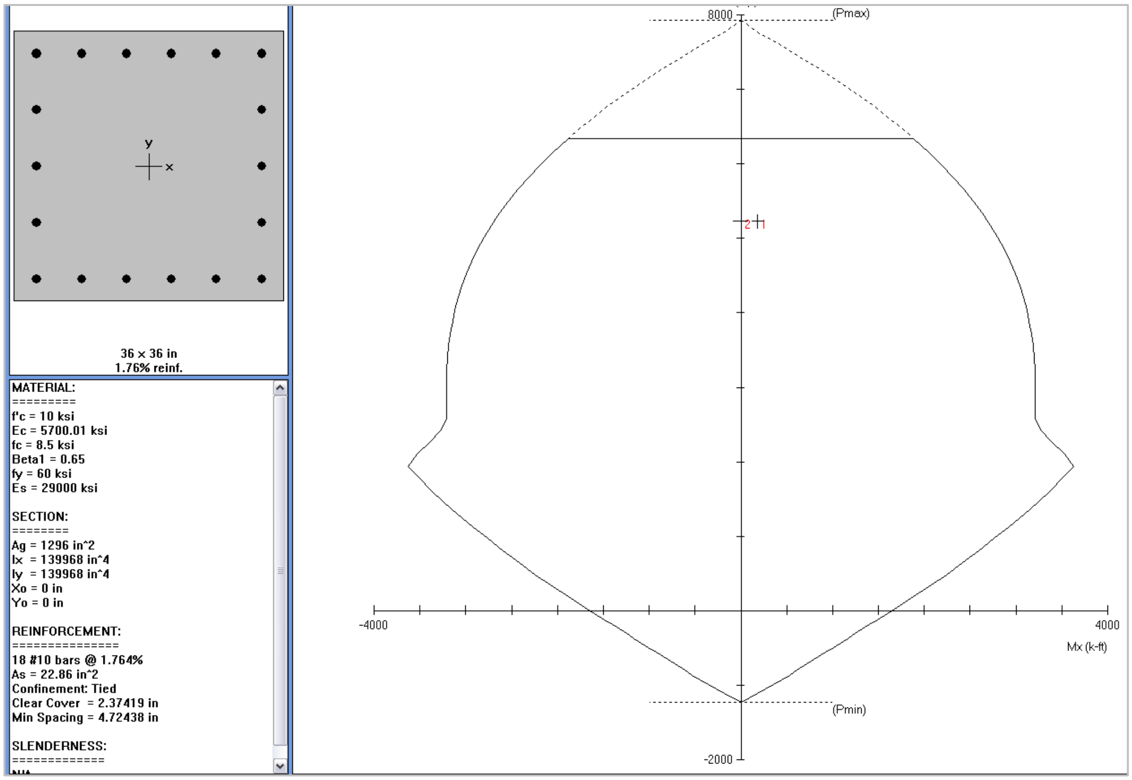


Figure 39 – First Floor Interior Column Check

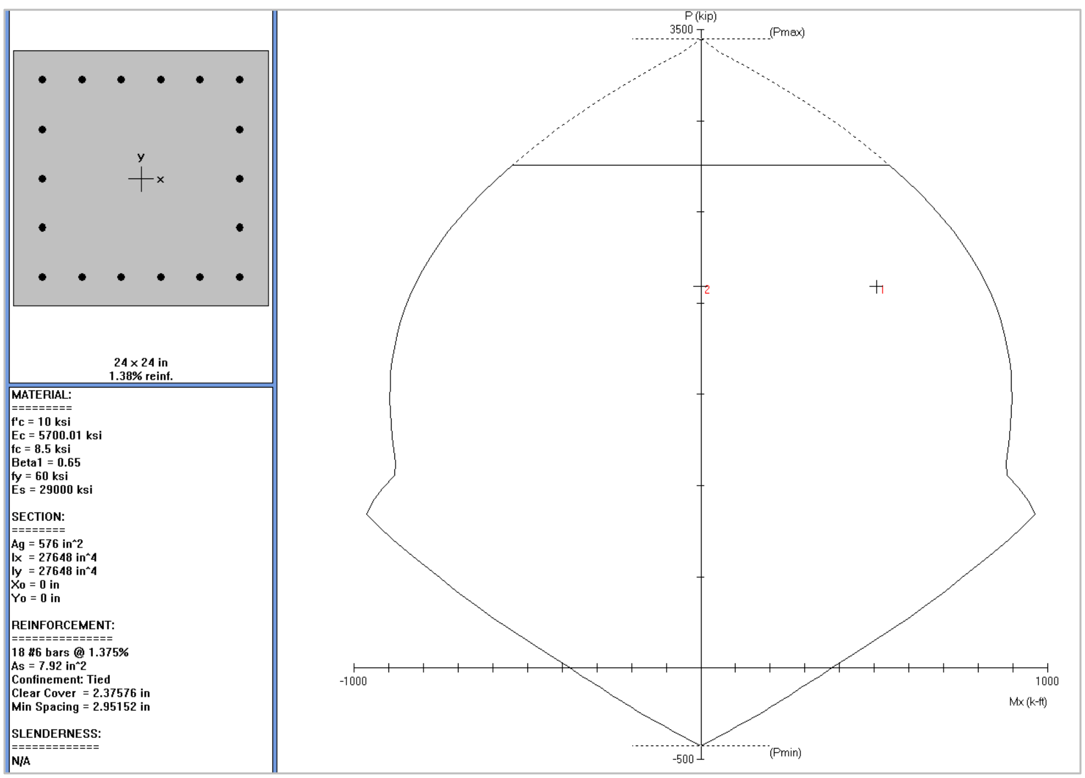


Figure 40 – 15th Floor Interior Column Check

Construction Management Breadth

The purpose of the construction management breadth was to see the comparison between the existing and proposed structure from a cost and sequencing stand point. The design needs to be both applicable in resisting loads and a feasible design in the real world. These comparisons will help better understand the relations between systems. The original detailed construction information was unattainable so cost and scheduling details will be produced for both systems respectively.

Cost Comparison

The cost of a system is a very important aspect of a design. Owners do not want to spend more money than needed and if an alternative solution to the building could be proposed at a lower cost they most likely would be interested. The cost for each structure was tabulated using RS Means data in particular using the online Cost Works application. The application was ran for the year 2007 (as far back as it would go) to get a more accurate cost for the time of construction. Detailed excel files for each cost can be found in appendix E while summarized date can be seen in the table below. Both analyses did not take into account the sublevels of the building under the assumption that it was already completed.

Cost Comparison			
	Bare Costs	O&P Costs	Total
Steel	$16,008,174	$3,610,772	$19,618,946
Concrete	$16,599,099	$5,062,030	$21,661,129

Schedule Comparison

Schedules were comprised by the use of RS Means Cost Works data and graphically with Microsoft Project. An arbitrary date was selected for the start of the schedule, because one was not known otherwise. Several assumptions and guidelines followed include:

Concrete:

- 300 Cubic Yards daily maximum concrete pours
- 40 Hour work weeks
- Sublevel parking garage already completed
- Tendons man not be stressed until 3 days after concrete poured
- Construction separated into 3 zones; A, B and C

Steel

- Erect 35 pieces of steel a day
- Place floor decking after two stories are erected
- Shear studs and welded wire fabric can lag behind decking

As stated earlier the construction of the concrete structure was broken into 3 separate zones. This allowed for the construction speed to increase greatly by allowing one section being framed then moving onto frame another section while the previous is being poured. The total time per 2 floors for the concrete structure came out to be 32 days. To complete all floors it would take approximately 400 days. A typical schedule was made for the duration of 2 floors that can be seen in the appendix E. This process would be repeated throughout the rest of the building. The steel schedule was also made for a 2 story interval as seen in appendix. It came out to be around 43 days to complete a section of 2 resulting in a construction time of 537 days.

<u>Conclusion</u>

The proposed system and the steel system were very close in bare material costs. When over head and production were factored into the equation the concrete structure did become a more expensive system. Due to the limitations of the system from a cost stand point the original structure makes more sense. These estimates were based only of RS Mean's data and may not account for all aspects of the cost associated with the respective systems. From the scheduling point of view the concrete structure could be erected quicker with repetitive form work and sequencing of slab sections. Further increase in scheduling could be achieved with a two-day concrete cycle construction style.

Architectural Breadth

Design Process

The redesigning of the new structural system impacts the existing architectural layout of the building. It was important to see to what effect these changes will make to Three PNC Plaza. The existing floor plans along with the vertical circulation of the building are key aspects that were investigated. The major impact to these areas will be the relocation of vertical stairs/elevators. Preliminary Floor Plans for the current building were available for several floors. The floors that will be investigated are a typical office level, a hotel/condo level and the 2nd floor. Columns will be kept in their original locations to not interfere with architecture layouts.

Typical Condo/Hotel Plan (Level 15 Floor Plan)

The existing layout can be seen in figure 41. The major change from the existing layout to the new layout results from the vertical circulation shafts, due to having to condense the elevators and stairwells into two distinct locations. The major area affected by the relocation of elements would be the hotel portion. Due to not being able to maintain all 7 elevators that serve the floor to left side of the building the service elevators were moved to the right side. This movement resulted in the loss of one hotel room space. Overall the new system provided very little leeway from an architectural stand point and the existing system produces a more desirable layout. The Condo units remained unaltered along with a majority of the closet and circulation spaces. The stairwell locations had minimal movement so no code violations would arise.

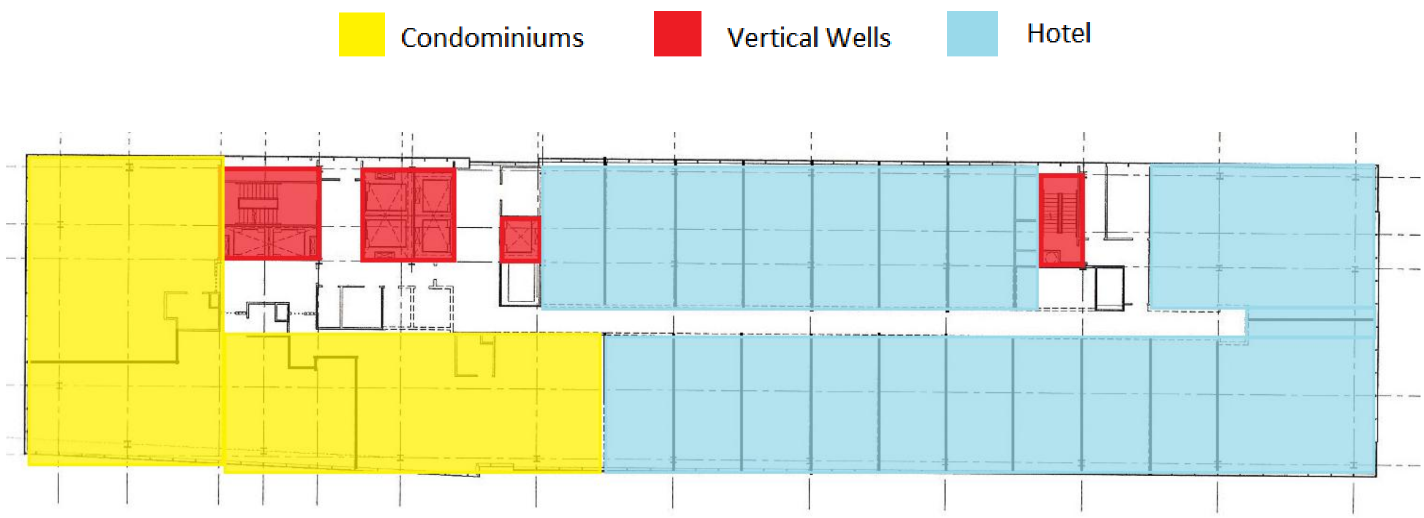


Figure 41 - Floor 15 Existing Plan

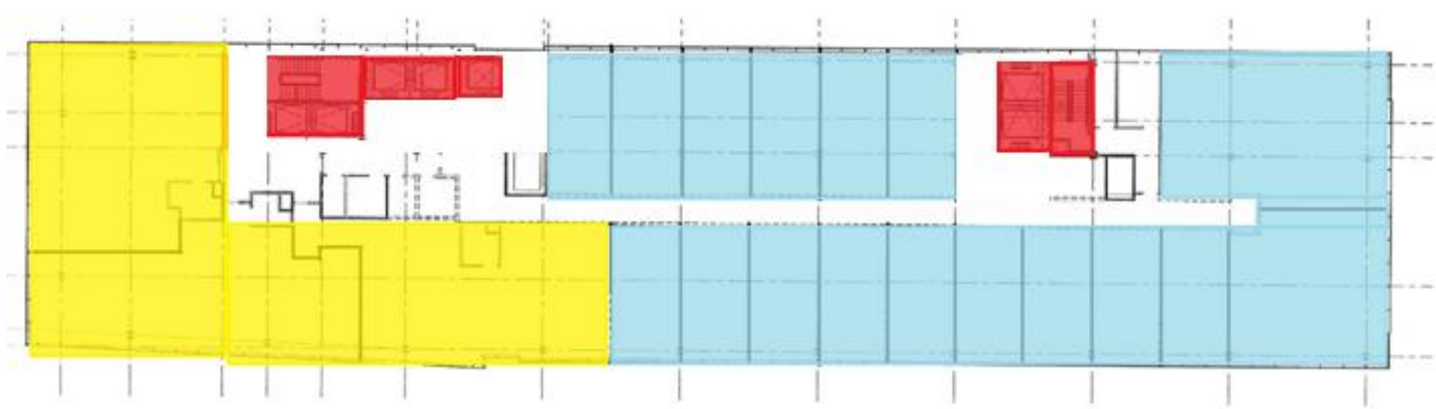


Figure 42 – Floor 15 Proposed Plan

Typical Office Plan (Level 5 Plans)

The office layout for the existing plans provided a very large area for the tenants to occupy and do what they wanted with. The only architectural features for these floors were the vertical circulation wells and storage/wash room spaces. The elevators and stairwells were shifted into larger clusters as required by the structural redesign. This resulted in the bathroom and storage spaces having minimal relocations to the center of the floor plan. Circulation paths can be seen in the existing layout highlighted in yellow. This path was essentially mirrored along the long axis of the building for the redesign due to elevator openings being shifted.

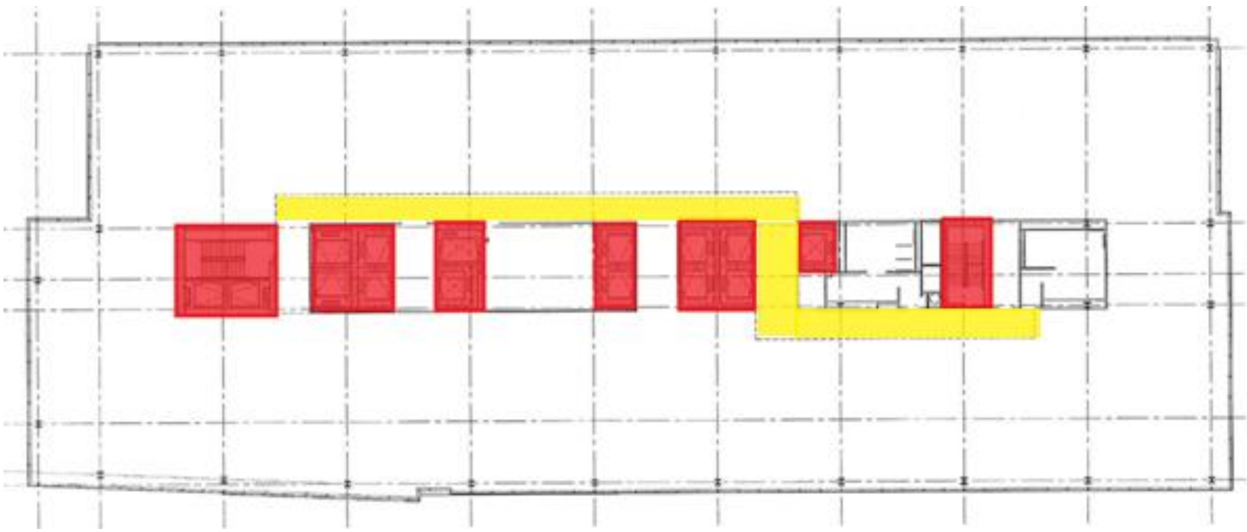


Figure 43 – Floor 5 Existing Plan

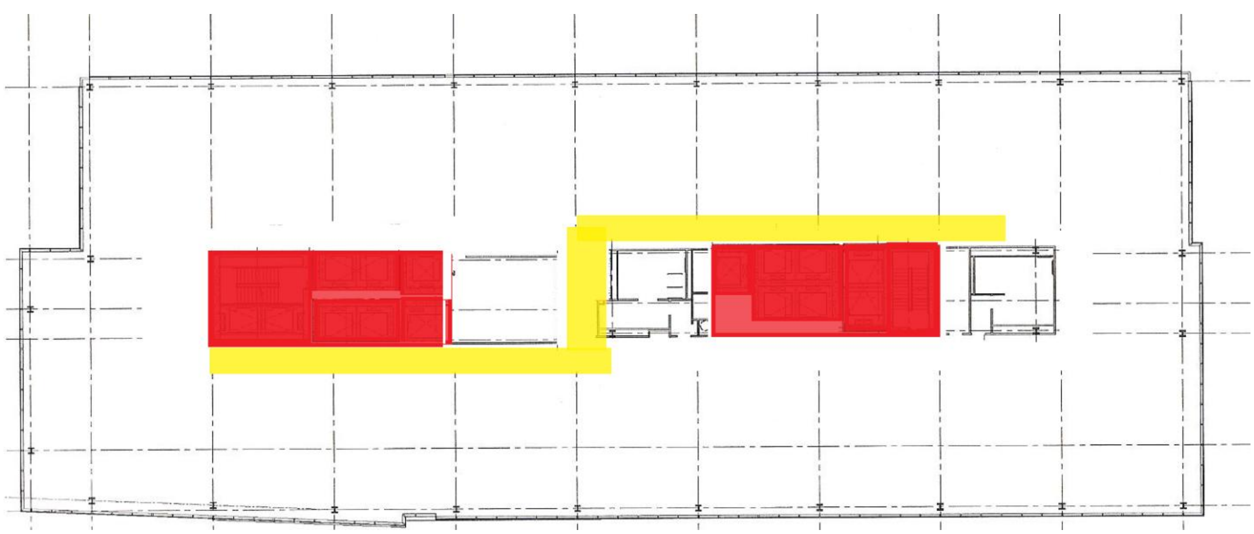


Figure 44 – Floor 5 Proposed Plan

Floor Plan Level 2

The provided plans for the second floor level had more information provided. The major aspects of the floor plan remain unchanged as seen from the figures below. However the ballroom area would still have to have a transfer girder as in the original plans to maintain the column free layout. This could be challenging due to the building weight increasing almost 2 times the existing structures weight. The new layout actually provides the ballroom and restaurant areas with some extra square footage while taking away space from the storage spaces located throughout the core of the building.

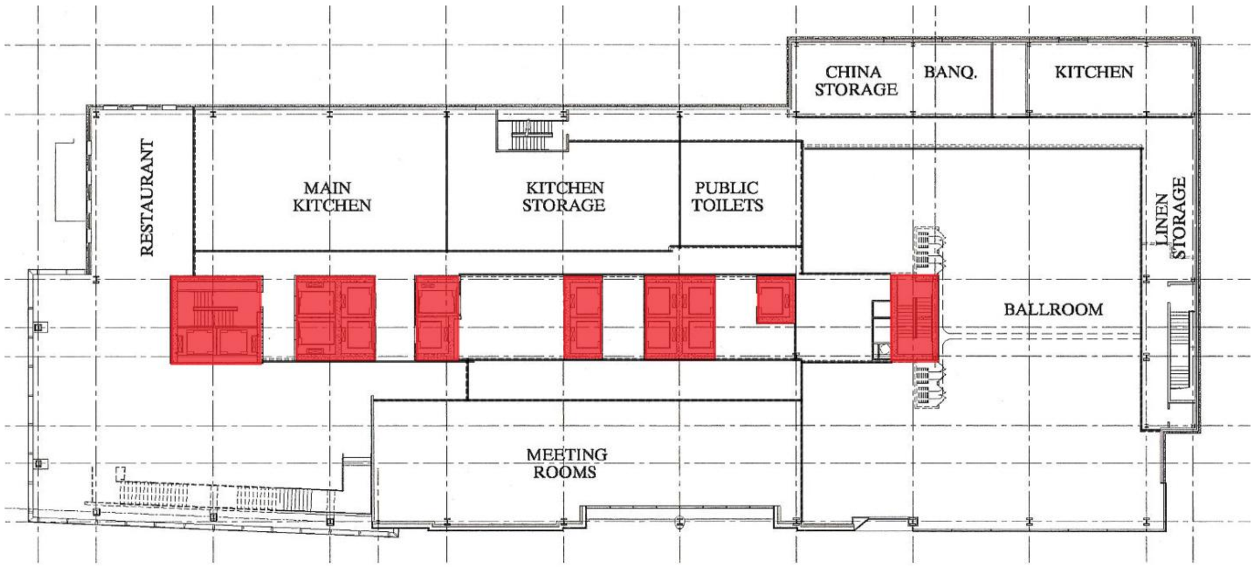


Figure 45 – Floor 2 Existing Plan

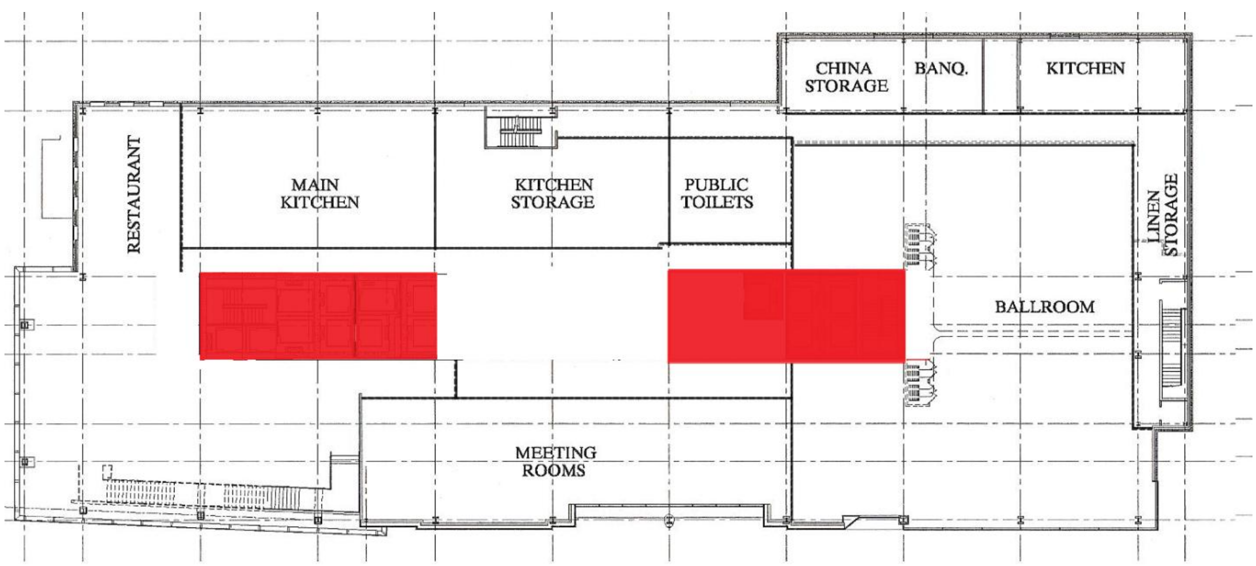


Figure 46 – Floor 2 Proposed Plan

Architectural Breadth Conclusion

The floor plan of the building in the first 14 floors did not sustain a major impact from the vertical corridor changes. However, from the new layouts provided a large impact could be seen in the hotel/condo portion of the building. Losing hotel units would be a major drawback to the design and new architectural layouts may be required. Also, the top floor condos take up a larger portion of the floor plan and could see negative affects when further investigated. From an architectural stand point the original steel structure provided more flexibility with the structural system allowing a more practical floor plan to utilize the space to its fullest.

Final Conclusion and Recommendations

Post-tensioning was one of the wide variety of slab systems that could have been utilized during the redesign of the structure. Some of the key reasons for choosing the system were:

- Significant portions of load can be resist by tendons resulting in simplification of ordinary reinforcement
- Reduction of dead load and member depths due to decreased amount of concrete required resulting in lower building weights
- Increased deflection and crack control

The existing exterior bay layout of 42.5'-0" by 30'-0" was preserved during the redesign without much of an issue. Due to the large bays columns were required to support large tributary areas. This resulted in fairly large column sizes to be used at the lower level of the building. The same basic lateral force resisting system was utilized in the redesign only with concrete moment frames instead of steel and both performed well. The redesign required movement of the interior core walls which did have negative effects form a circulation/architectural stand point.

Based upon all of the information from thesis work the original assumption that the existing structure would be the more efficient system was proven. The concrete structure did not adapt to the mix use nature of the building as well as the steel structure can. Different loadings throughout the levels of the structure were more readily handled by the steel framed building. From a cost stand point it seems that the steel structure came in lower than the concrete structure. Due to the minimal gains of the concrete structure vs. the steel, this redesign would not be suggested.

APPENDIX

APPENDIX A: Wind

Risk Category: III
Basic Wind Speed: $V = 120$ mph
Directionality Factor: $K_d = 0.85$
Exposure Category: B, urban
Topographic Factor: $K_{zt} = 1.0$

Gust Factor:

Cannot assume rigid, cannot use Approximate Natural Frequency due to $319 > 300$

Section 26.9.5:

$$g_Q = 3.4 \qquad g_v = 3.4$$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}, \text{ where: } n_1 = \frac{1}{T_a} \qquad T_a = \text{natural period}$$

Sec 12.8.2.1: $T_a = C_t h_n^x \qquad C_t = 0.02 \qquad x = 0.75$

$T_a = 0.02(319)^{0.75}$ $\quad h_n = 319$ main roof height
$= 1.5096$

$n_1 = \frac{1}{1.5096} = 0.6624$, less than 1.0, Flexible assumption was correct

Sec 27:

$$g_R = \sqrt{2\ln(3600 \cdot 0.6624)} + \frac{0.577}{\sqrt{2\ln(3600 \cdot 0.6624)}} = \underline{4.09}$$

$I_z = c\left(\frac{33}{\bar{z}}\right)^{1/6}$ where $\bar{z} = 0.6(\text{height})$ $\quad 0.6(319) = 191.4$
$c = 0.30$

$$= 0.30\left(\frac{33}{191.4}\right)^{1/6} = \underline{0.2238}$$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$ where $\ell = 320$
$\bar{\epsilon} = 1/3.0$

$$= 320\left(\frac{191.4}{33}\right)^{1/3} = \underline{574.945}$$

$$Q = \frac{1}{\sqrt{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \frac{1}{\sqrt{1 + 0.63\left(\frac{297+319}{574.945}\right)^{0.63}}} = \underline{0.7766}$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} \quad , \quad N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \quad , \quad \bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} \left(\frac{88}{66}\right) V$$

Solve $\bar{V}_{\bar{z}}$: $\bar{\alpha} = 1/4.0$ $\quad \bar{b} = 0.45$

$$\bar{V}_{\bar{z}} = 0.45 \left(\frac{191.4}{33}\right)^{1/4} \left(\frac{88}{66}\right) \cdot 120 = 111.735$$

Solve N_1 : $\frac{0.6624(574.945)}{111.735} = 3.408$

Solve R_n : $\frac{7.47(3.408)}{(1 + 10.3(3.408))^{5/3}} = 0.06455$

R_L : $\eta = 15.4 n_1 L / \bar{V}_z = 15.4(0.6624)(122.75) / 111.735 = 11.2066$

$$R_L = \frac{1}{11.2066} - \frac{1}{2(11.2066)^2}\left(1 - e^{-2(11.2066)}\right) = \underline{0.08525}$$

R_B : $\eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.6624)(297) / 111.735 = 8.0993$

$$R_B = \frac{1}{8.0993} - \frac{1}{2(8.0993)^2}\left(1 - e^{-2(8.0993)}\right) = \underline{0.1158}$$

R_h : $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.6624)(319) / 111.735 = 8.6992$

$$R_h = \frac{1}{8.6992} - \frac{1}{2(8.6992)^2}\left(1 - e^{-2(8.6992)}\right) = \underline{0.1083}$$

* $\beta = 1.5\%$ for steel and concrete buildings

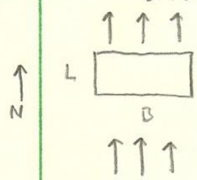
$$R = \sqrt{\frac{1}{0.015}(0.06455)(0.1083)(0.1158)(0.53 + 0.47(0.08525))}$$

$$= \underline{0.1754}$$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) = 0.8231$$

Enclosure Classification : Enclosed , $GC_{pi} = \pm 0.18$

$L/B = \frac{122.75}{297} = 0.41$ Windward Wall 0.8
Leeward Wall -0.5



$p = q G_f C_p - q_i (GC_{pi})$ $K_z = 1.3728$

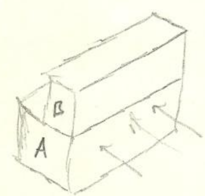
$q = 0.00256(1.3728)(1.0)(.85)(120)^2$

$= 43.0159$

windward $p = 28.325 - 28.325(-.18) = 33.424$ psf

leeward $p = -14.1625 - 5.0985 = -19.261$ psf

* These values are off from spread sheet values due to taking into account shape of buildings



Spread Sheet seperates building into two parts to get a more accurate analysis.

APPENDIX B: Seismic

SESMIC DESIGN

Site Class D (Provided) Seismic Category B

$S_{MS} = F_a S_s$ $S_{M1} = F_v S_1$

$S_s = 0.201$ $S_1 = 0.118$ } (USGS)

$F_a = 1.6$ $F_v = 2.328$

$S_{MS} = 0.3216$ $S_{DS} = \frac{2}{3} S_{MS} = 0.2144$

$S_{M1} = 0.2747$ $S_{D1} = \frac{2}{3} S_{M1} = 0.1831$

$I_e = 1.25$ (11.5-1)

$R = 5.5$ N/S

5 E/W

Equivalent Lateral Force Procedure

$V = C_s W$

$W = 103700$ Kips

$$C_s = \min \begin{cases} S_{DS}/(R/I) \\ S_{DS}/(T(R/I)) \\ S_{D1} \cdot T_L / (T^2 (R/I)) \end{cases}$$

$0.2144/(5.5/1.25) = 0.049$

$0.2144/(2.114(5.5/1.25)) = 0.0230$ * controls

$0.118(12) / [(2.114)^2 (5.5/1.25)] = 0.072$

$$T = \begin{cases} C_u \cdot T_a = 1.4(1.51) = 2.114 \\ T_b \approx 3.3 \quad \text{(ETABS)} \end{cases}$$

C_s cannot be less than:

$0.044 S_{DS} I_e \geq 0.01$

$\hookrightarrow = 0.011$

$C_u = 1.4$ (12.8-1)

$T_a = C_t h_n^x = 0.02(319)^{.75} = 1.51$

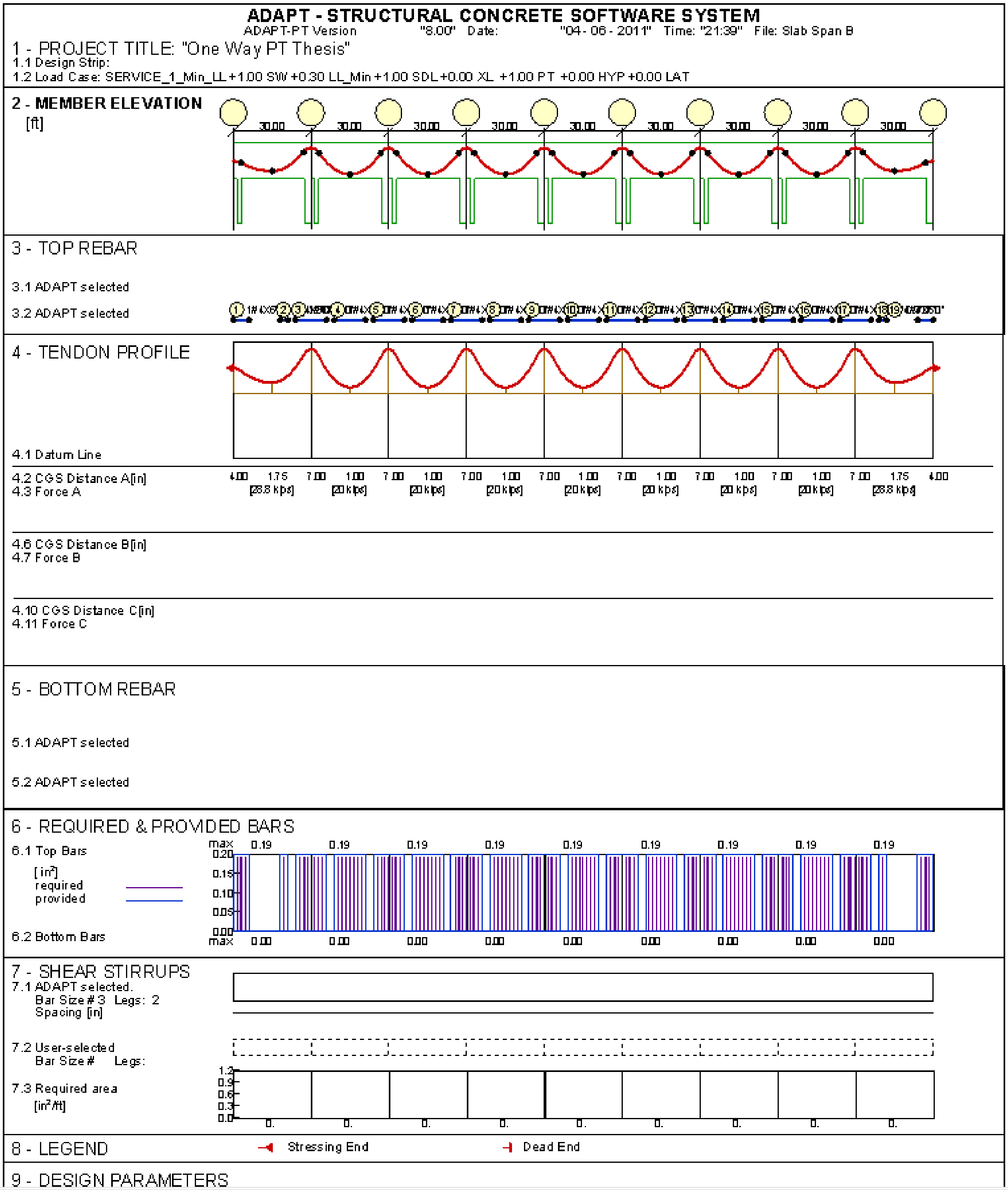
$T_L = 12$ Figure 22-12 (ASCE 7-10)

$0.011 < 0.0230$ ∴ use 2nd value

$V = 0.0230(103700) = 2390.3$ K N/S

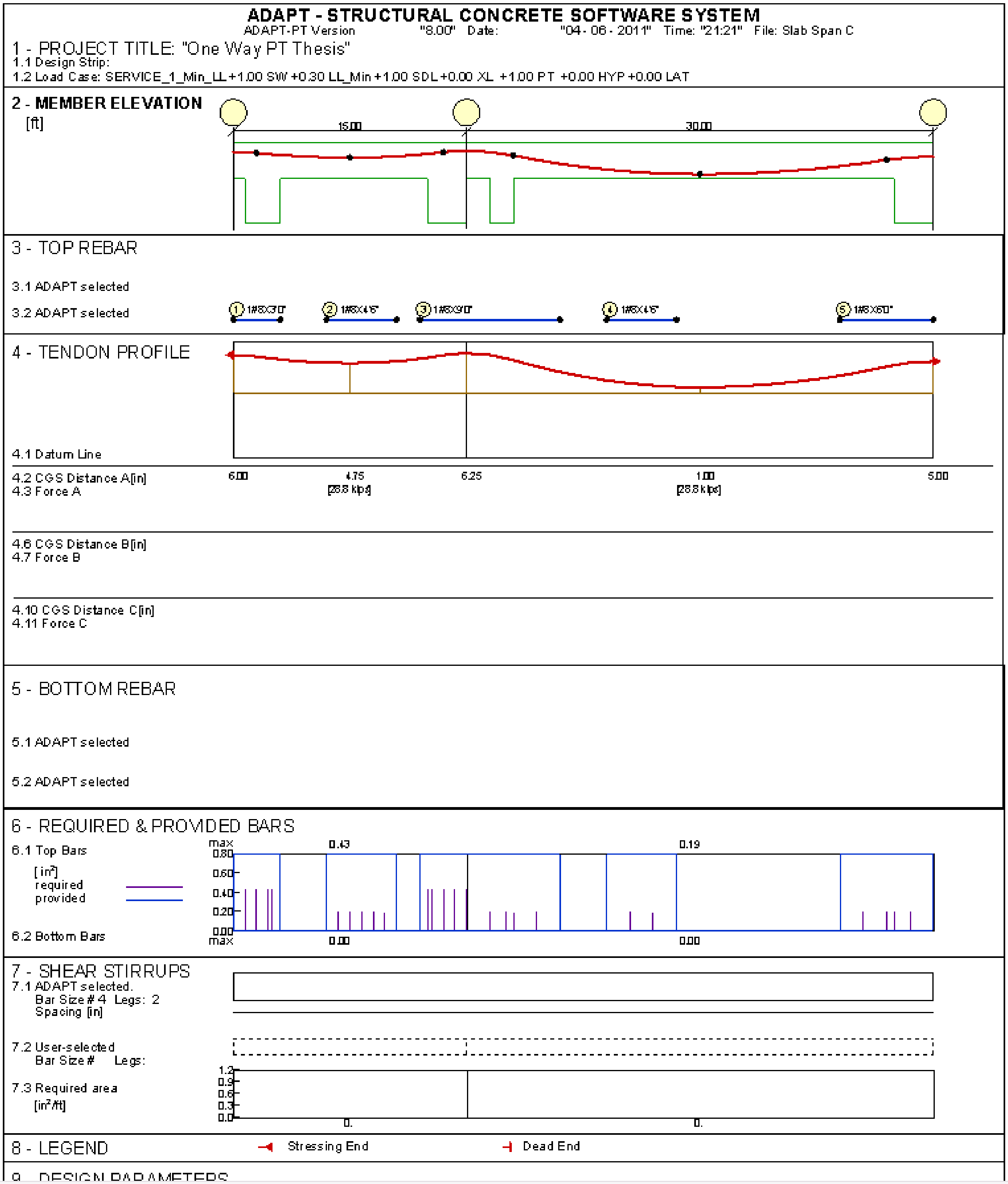
$0.0254(103700)$ 2629.3 K E/W

APPENDIX C: Post-Tensioning

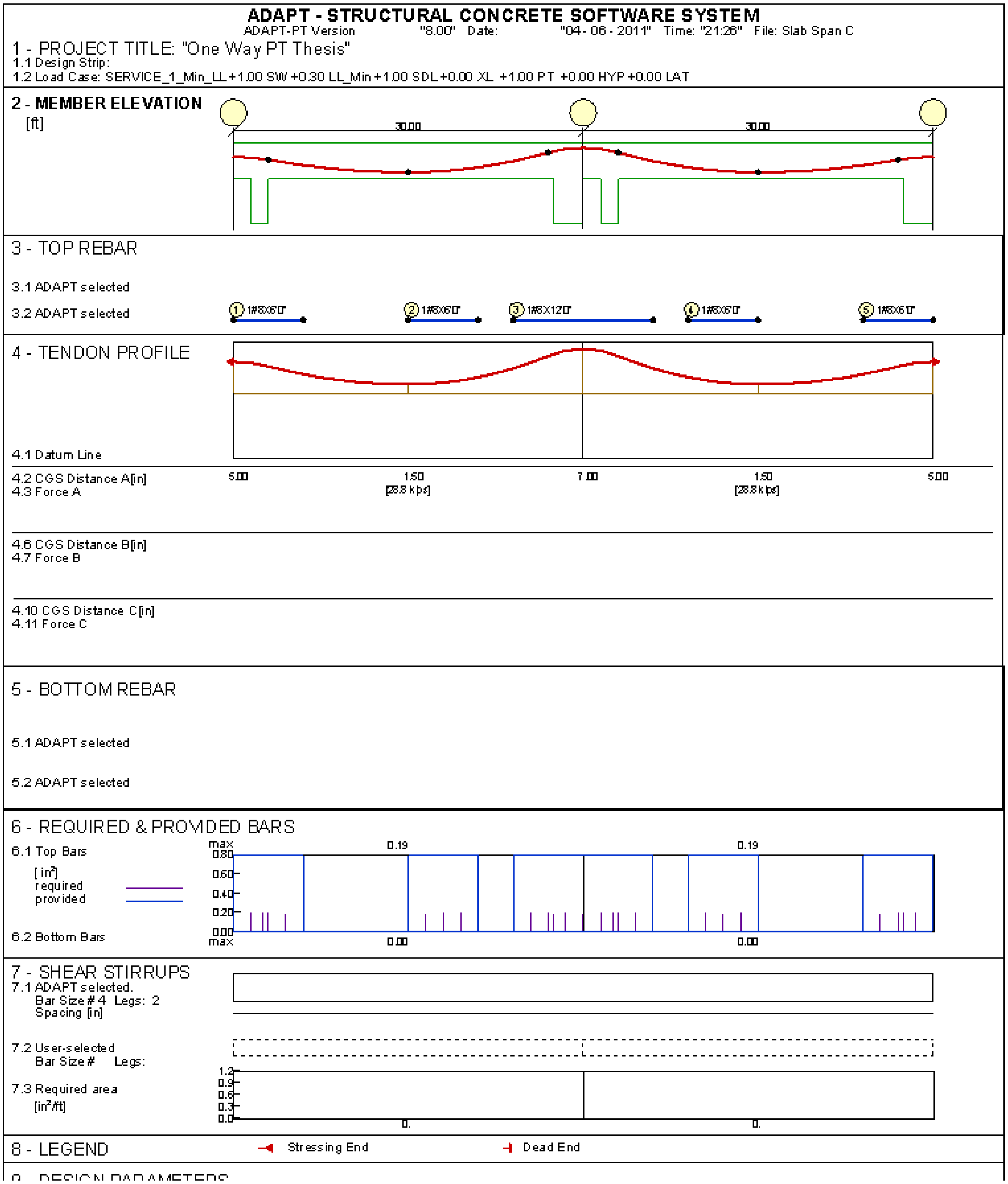


Span A Design

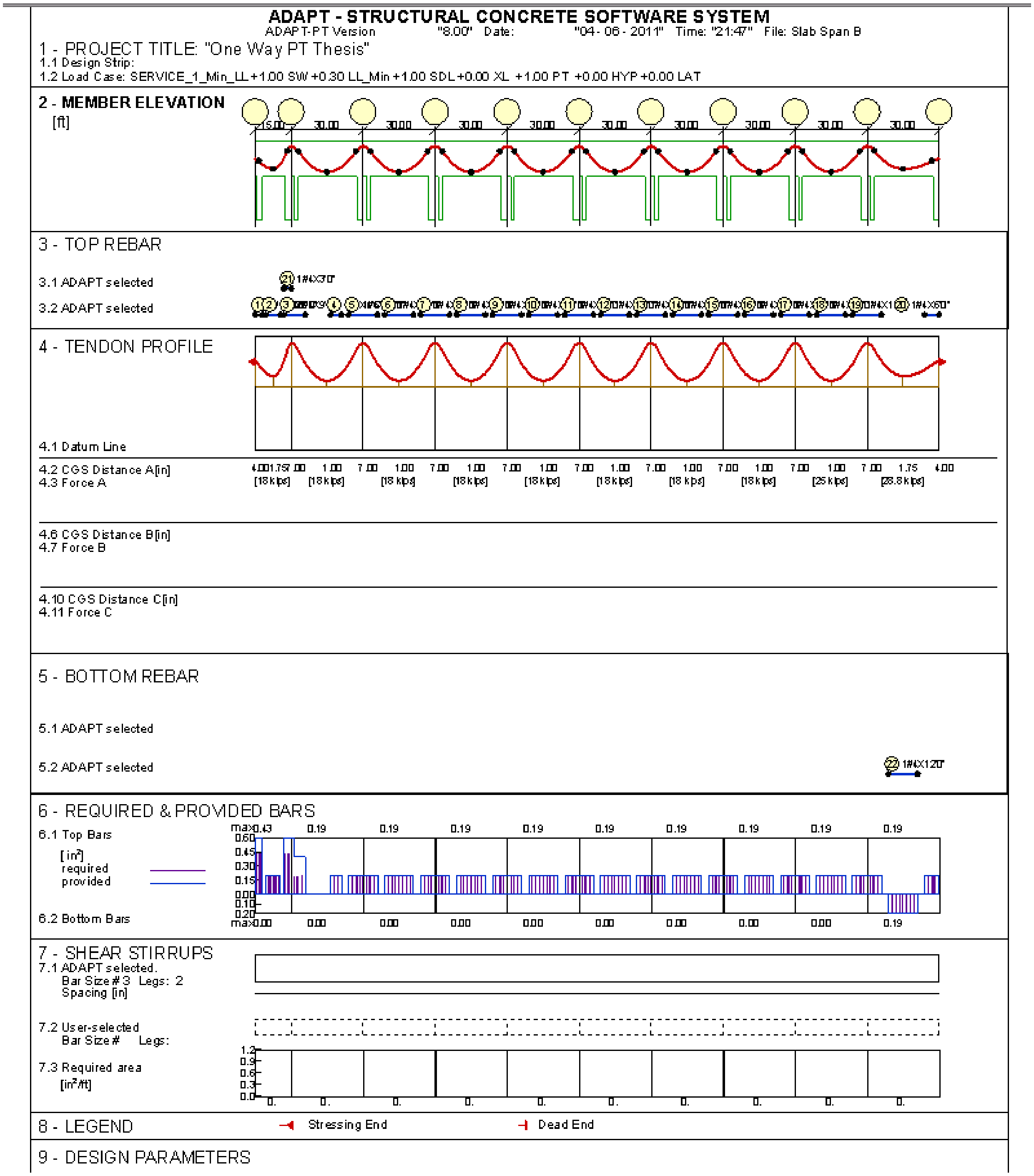
Slab B Design



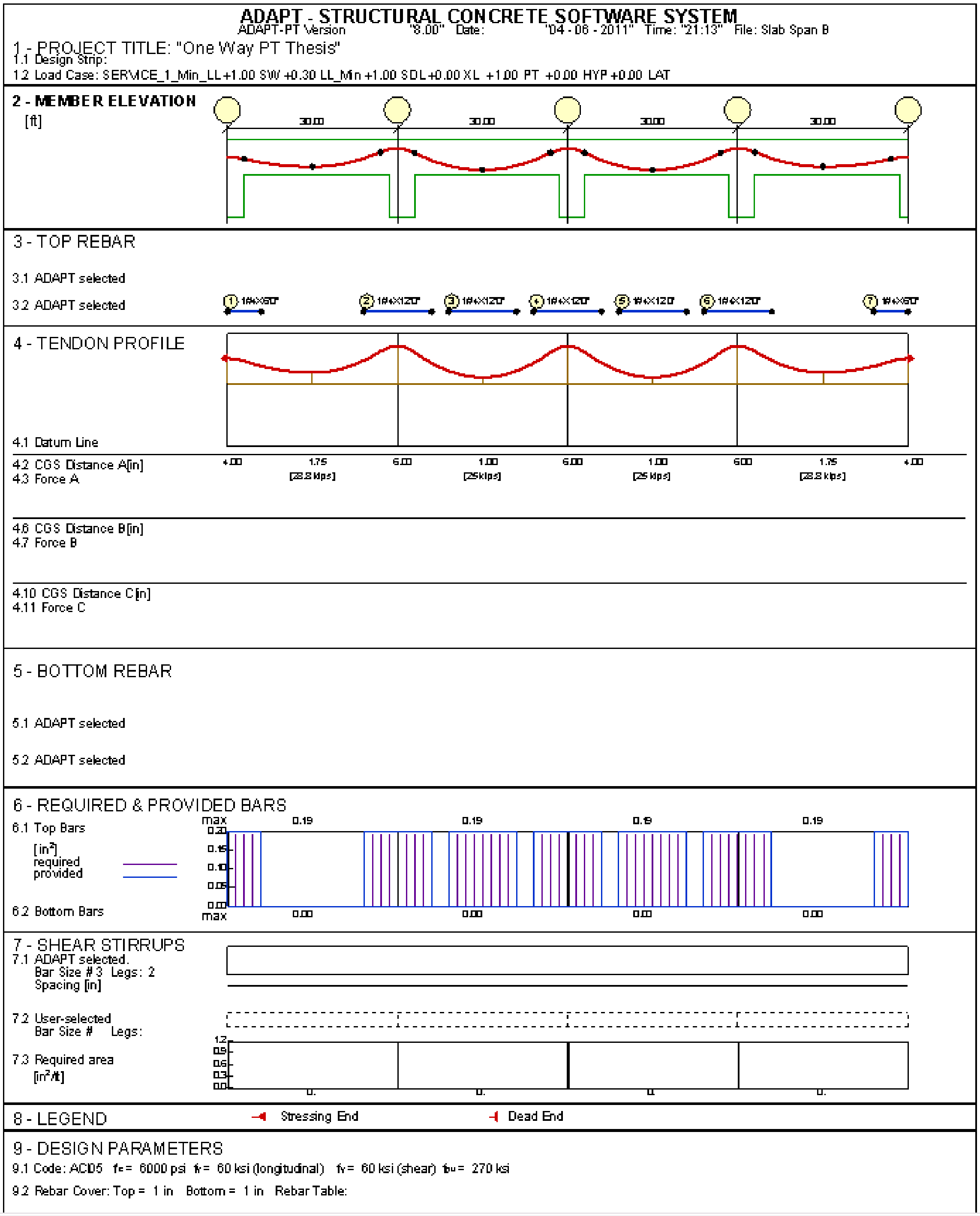
Slab C Design



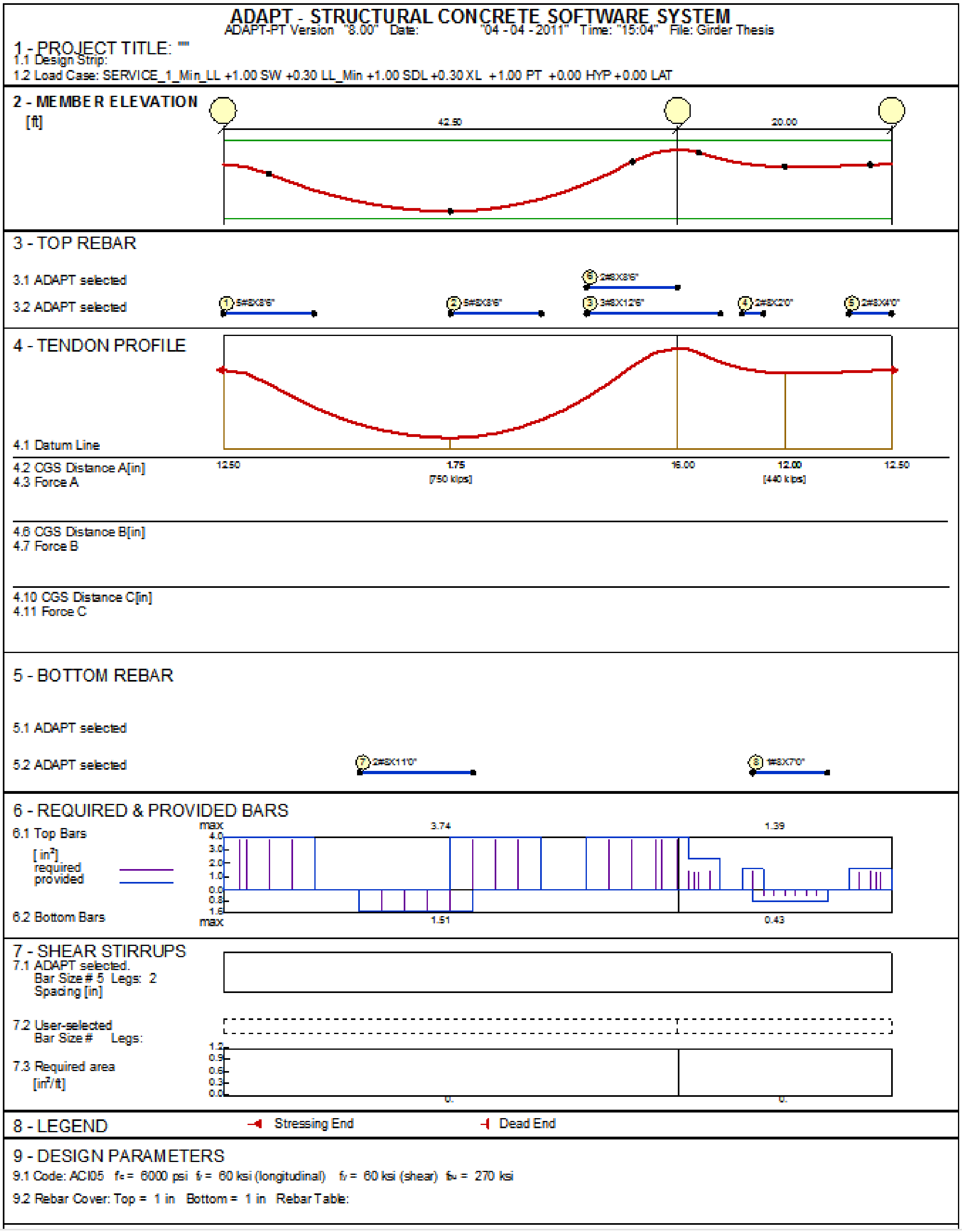
Slab D Design



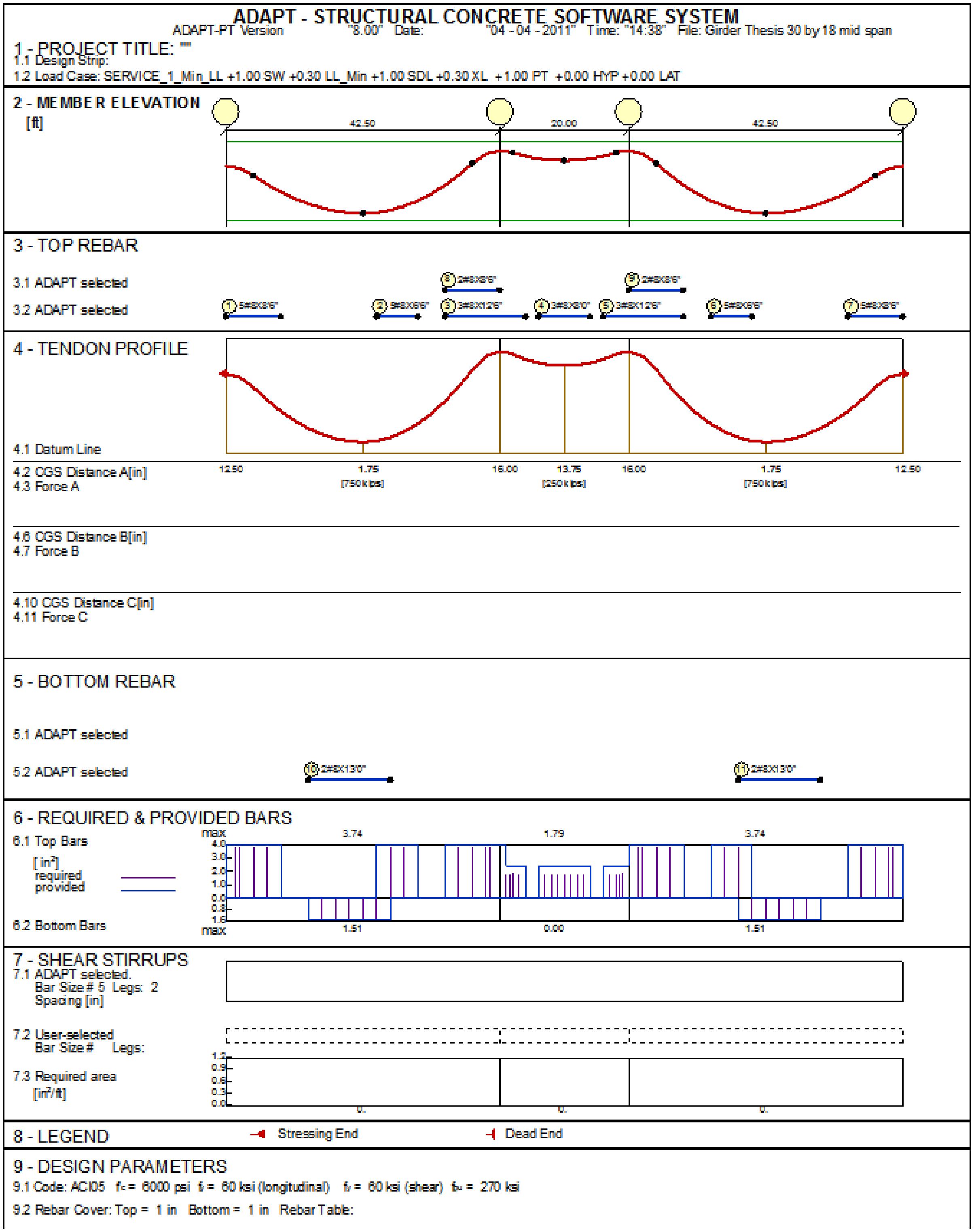
Slab E Summary



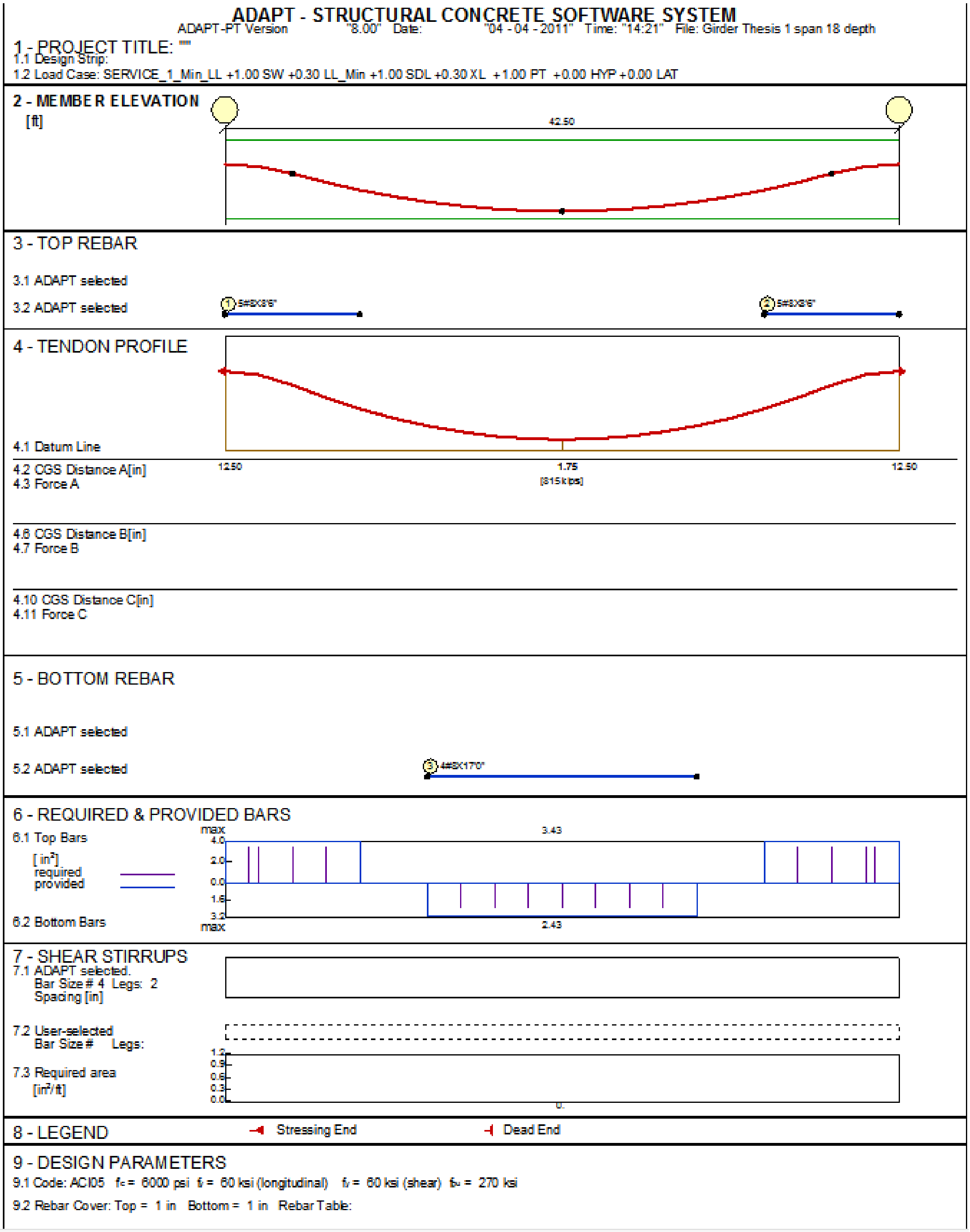
Girder A Summary



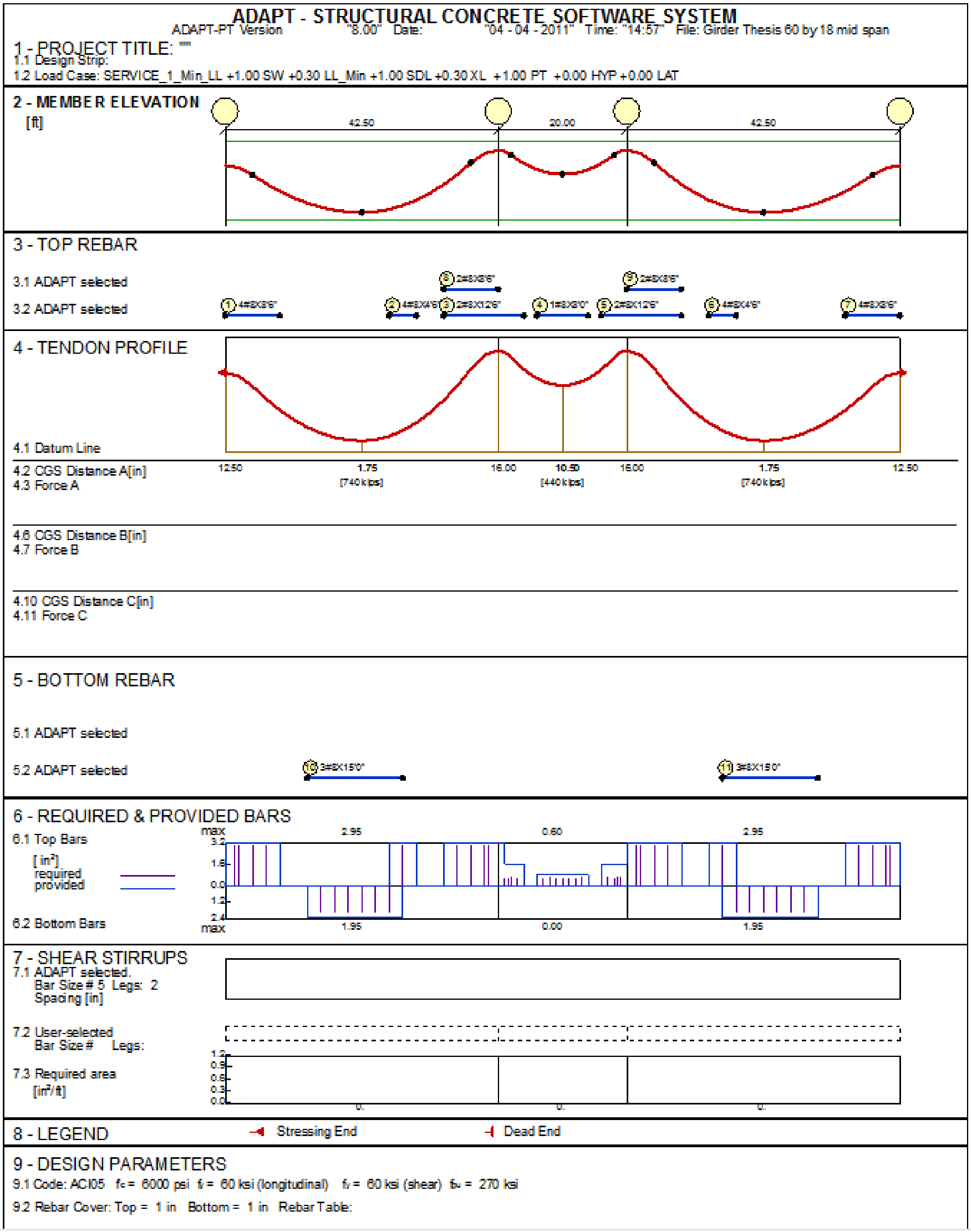
Girder B Summary



Girder C Summary



Girder D Summary



PT Girder Hand Calculation

PT GIRDER

$\frac{1}{2}''\ \phi$ w/ $A = .153\ in^2$ 270 Ksi strands

Thesis Design

Preliminary Thickness $h = L/30$ $\frac{42.5(12)}{30} = 17''$ rounded up to 18" to be conservative

Loading

$DL = 126\,psf\,(30) = 3.8\ klf$ Slab
$SW = .6\ klf$ Beam
total $= 4.4\ klf$

$LL = 100\,psf$ $100\left(0.25 + \frac{15}{\sqrt{4(1275)}}\right) = 46\,psf \approx 65\,psf$ conservative

$LL = 65(30) = 1950$

$A = bh = 60(18) = 1080\ in^2$

$S = \frac{bh^2}{6} = 3240\ in^3$

$1.2(4.4) + 1.6(1.95) = 8.4\ klf$

Target Loading Balance (Trying 75% can vary but for calculations will use this)

$0.75(8.4) = 6.3\ k/ft$

Tendon Conduit	Location (Typical)
anchor ext	4.0"
top int support	7.0"
bot int support	1.0"
end span	1.75"

$P = \frac{w_b L^2}{8} = \frac{6.3\left(42.5 - \frac{36}{12}\right)^2}{8} = 1228.7^K$

* Assume 15 ksi losses in Strand

$f_{se} = .75 f_{pu} = .75(270000) - 15000 = 187.5\ ksi$

$P_{eff} = f_{se} A = (187.5)(.153) = 28.69^K$

$\frac{1229}{28.7} = 42.8 \approx \boxed{43\ \text{tendons}}$ Ext.

P_{actual} Exterior $= 43(28.7) = 1234.1^K$ * Design from ADAPT-PT used 38 strands w/ a PT force of 1000 K

↳ I feel this is pretty close for a rough hand calc...

Going to use ADAPT design

Interior $= \frac{6.3\left(20 - \frac{36}{12}\right)^2}{8} = 228^K$

$\frac{228^K}{28.7} \approx \boxed{8\ \text{tendons}}$ Int. → ADAPT used 10 tendons once again pretty similar for rough hand calculations.

APPENDIX D: Shear Wall Design

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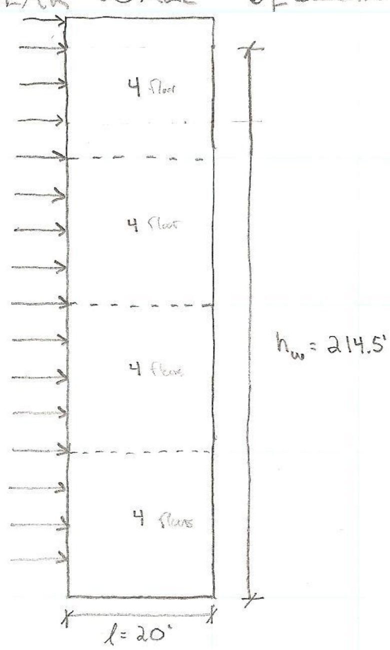
SHEAR WALL DESIGN



$t = 18"$

V_i = Excel File

* WALL WAS BROKEN INTO 4 PARTS TO SIMPLIFY DESIGN

SIMPLIFIED METHOD

$V_c \leq 2\sqrt{F_c'}\, t\, d \qquad d = 0.8 l_w$

$V_c = 2\sqrt{5000}\,(18)\,0.8(20 \cdot 12) = 488752 \approx 489^k$

METHOD 1

298.8

$V_c \leq 3.3\sqrt{F_c'}\, t \cdot d + \dfrac{N_u d}{4 l_w} \Rightarrow 3.3\sqrt{5000}\,(18)(0.8 \cdot 20 \cdot 12) + \dfrac{(93.36)(16)(192)}{4(240)} = 1105^k$

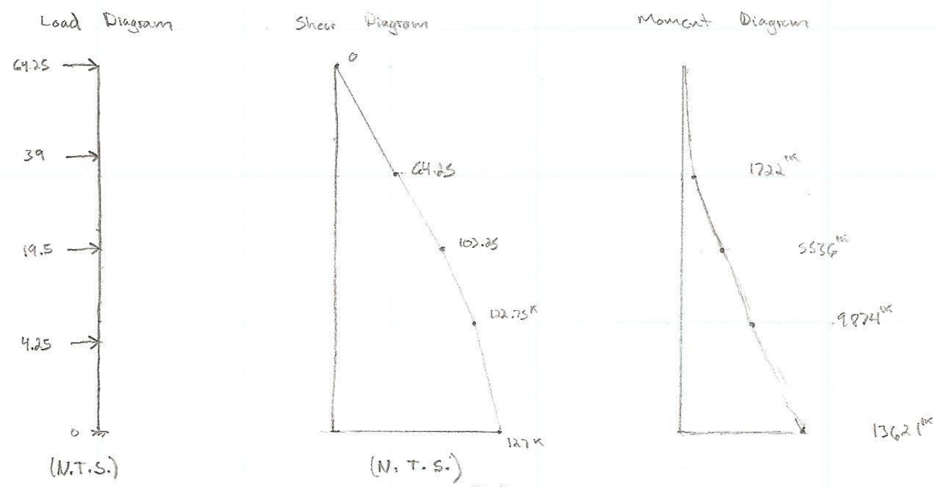
$D = 126\,psf\,(15)(20) = 37800$

$L = 100\,psf\,(15)(20) = 30000$

$N_u = 1.2(37.8) + 1.6(30) = 93.36\ ^k/_{Floor}$

SHEAR WALL CONT.

METHOD 2 $$V_c \le \left[0.6\sqrt{f'_c} + \frac{l_w\left(1.25\sqrt{f'_c} + 0.2\frac{N_u d}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right] h\, d$$



58.9 62.24 30.9

$$V_c \le \left[0.6\sqrt{5000} + \frac{(240)\left(1.25\sqrt{5000} + 0.2\,\frac{93.36(1000)(12)}{240(20)}\right)}{\frac{13621(12)}{127} - \frac{240}{2}}\right] 18(192)$$

1167

$= 2537^k$

USE $V_c \le 489^k$

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SHEAR WALL CONT.

Horizontal Reinforcement :

$$\tfrac{1}{2}\phi V_c = \tfrac{1}{2}(0.75)(489) = 183.4^K$$

$V_u = 127^K < 183.4^K$ $\therefore$ USE MIN HORIZONTAL STEEL

$\rho_t \geq 0.0025$

$\rho_t = \frac{A_v}{S}$

S_{min} | $l_w/5$, $3h$, $18"$ — 18" controls

$A_v = S\rho_t = 18(0.0025) = 0.045 in^2$ * Seems Pretty Small

Use #3 Bar w/ 0.11 A_s Final design (2) #3 @ 18" oc

Vertical Reinforcement :

$\rho_l = \frac{A_v}{S_h} > 0.0025$ $\cancel{+0.5(2.5-\frac{h_w}{l_w})(\rho_t - 0.0025)}$

use (2) #3 @ 18"

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SHEAR WALL CONT.

FLEXURAL

$d = 192 \qquad jd = 0.9(192) = 172.8$

$M_u = \phi M_n = \phi A_s f_y (jd) \Rightarrow \qquad 13621(12) = 0.9(A_s)(60)(172.8)$

$A_s = 17.5 \text{ in}^2$

C=T

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{17.5(60000)}{0.85(5000)(18)} = 13.7"$

New $jd = d - \frac{a}{2} = 192 - \frac{13.7}{2} = 185.15$

New A_s w/ New $jd \Rightarrow \qquad 13621(12) = 0.9(A_s)(60)(185.15)$

$A_s = 16.3 \text{ in}^2$

Try (16) #9 w/ $A_s = 16 \text{ in}^2$

Tension Controlled

$d_t = (20)(12) - 3 = 237$

C=T $\quad a = \frac{16(60000)}{0.85(5000)(18)} = 12.5"$

$c = \frac{a}{\beta_1} = \frac{12.5}{0.85} = 14.7$

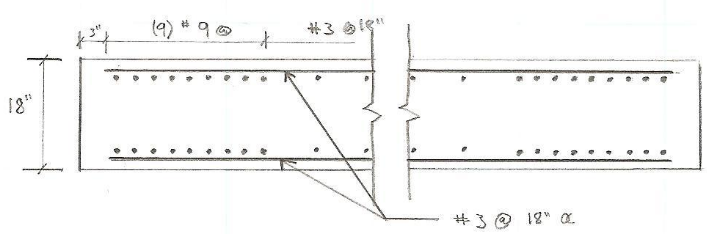
$\varepsilon_t = \varepsilon_a \left(\frac{d_t - c}{c}\right) = 0.003\left(\frac{237 - 14.7}{14.7}\right) = 0.045$

$0.045 \not> 0.005$

Increase f'_c to 6000

$\varepsilon_t = 0.055 > 0.005 \therefore$ Tension Controlled

* This increase will just increase the max V_u for other calcs



APPENDIX E: Construction Management Breadth

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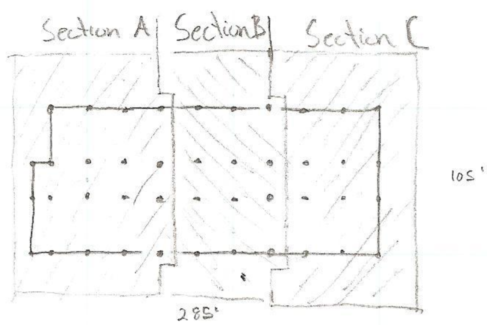
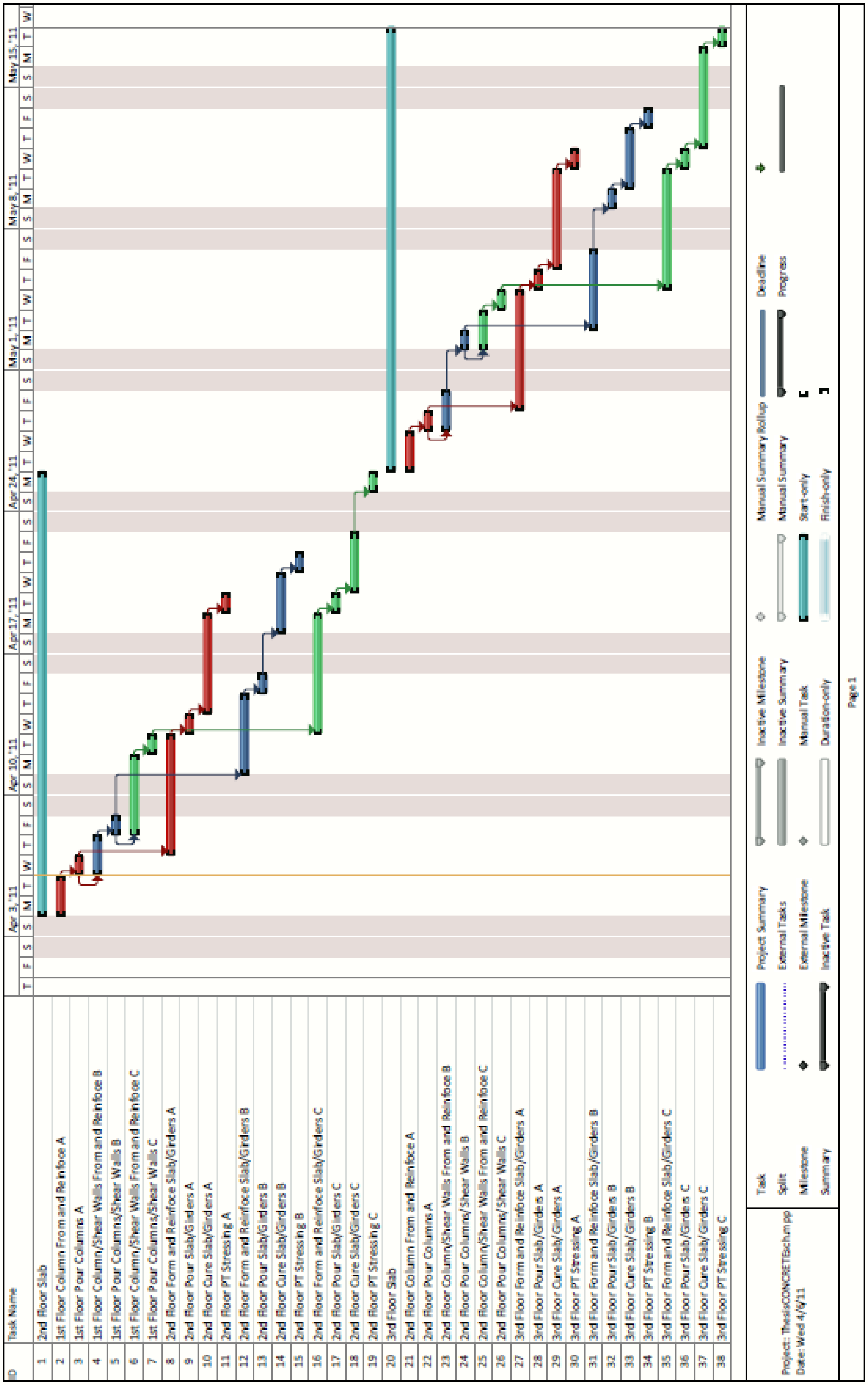
Cubic Yards Concrete Floors G– = 785 ≈ 3 days

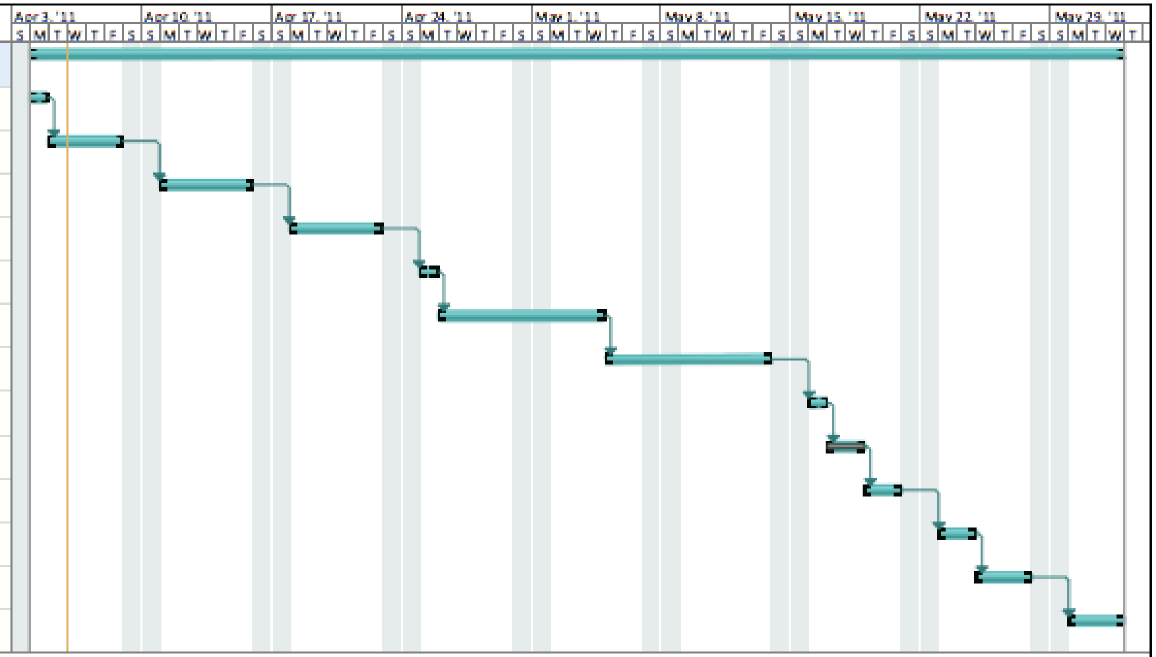
Cubic Yards Concrete Floors = 467 ≈ 2 days

- 300 cubic yards daily Max concrete pours
- 40 Hour Work week M – F
- Portions of Sublevel Parking Structure assumed already constructed
  ↳ Schedule is from ground Floor and up.
- Tendons May not be stressed untill 2 days after pour

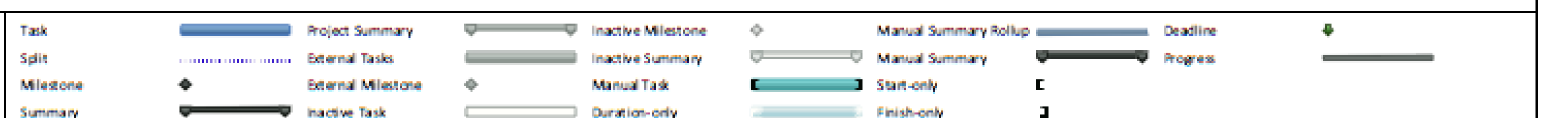



ID
Task Name
1 2nd Floor Slab
2 1st Floor Column From and Reinfoce A
3 1st Floor Pour Columns A
4 1st Floor Column/Shear Walls From and Reinfoce B
5 1st Floor Pour Columns/Shear Walls B
6 1st Floor Column/Shear Walls From and Reinfoce C
7 1st Floor Pour Columns/Shear Walls C
8 2nd Floor Form and Reinforce Slab/Girders A
9 2nd Floor Pour Slab/Girders A
10 2nd Floor Cure Slab/Girders A
11 2nd Floor PT Stressing A
12 2nd Floor Form and Reinforce Slab/Girders B
13 2nd Floor Pour Slab/Girders B
14 2nd Floor Cure Slab/Girders B
15 2nd Floor PT Stressing B
16 2nd Floor Form and Reinforce Slab/Girders C
17 2nd Floor Pour Slab/Girders C
18 2nd Floor Cure Slab/Girders C
19 2nd Floor PT Stressing C
20 3rd Floor Slab
21 2nd Floor Column From and Reinfoce A
22 2nd Floor Pour Columns A
23 2nd Floor Column/Shear Walls From and Reinfoce B
24 2nd Floor Pour Columns/Shear Walls B
25 2nd Floor Column/Shear Walls From and Reinfoce C
26 2nd Floor Pour Columns/Shear Walls C
27 3rd Floor Form and Reinfoce Slab/Girders A
28 3rd Floor Pour Slab/Girders A
29 3rd Floor Cure Slab/Girders A
30 3rd Floor PT Stressing A
31 3rd Floor Form and Reinfoce Slab/Girders B
32 3rd Floor Pour Slab/Girders B
33 3rd Floor Cure Slab/Girders B
34 3rd Floor PT Stressing B
35 3rd Floor Form and Reinfoce Slab/Girders C
36 3rd Floor Pour Slab/Girders C
37 3rd Floor Cure Slab/Girders C
38 3rd Floor PT Stressing C
Apr 3, '11
Apr 10, '11
Apr 17, '11
Apr 24, '11
May 1, '11
May 8, '11
May 15, '11
Project: ThesisCONCRETEsch.mpp
Date: Wed 4/6/11
Task
Split
Milestone
Summary
Project Summary
External Tasks
External Milestone
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
Deadline
Progress
Page 1

ID	Task Name	Duration	Start	Finish
1	Steel Structure (Ground - 2nd Floor)	43 days	Mon 4/4/11	Thu 6/2/11
2	Shakeout	1 day	Mon 4/4/11	Mon 4/4/11
3	Erect Columns (Ground - 2nd Floors)	4 days	Tue 4/5/11	Fri 4/8/11
4	Erect Beams (1st Floor)	5 days	Mon 4/11/11	Fri 4/15/11
5	Erect Beams (2nd Floor)	5 days	Mon 4/18/11	Fri 4/22/11
6	Surveyors plumb-up structure	1 day	Mon 4/25/11	Mon 4/25/11
7	Place Metal Decking (1st Floor)	7 days	Tue 4/26/11	Wed 5/4/11
8	Place Metal Decking (2nd Floor)	7 days	Thu 5/5/11	Fri 5/13/11
9	Detailing- Shear Studs	1 day	Mon 5/16/11	Mon 5/16/11
10	Detailing - Final Bolts/Welds	2 days	Tue 5/17/11	Wed 5/18/11
11	Edge Forms	2 days	Thu 5/19/11	Fri 5/20/11
12	WWF	2 days	Mon 5/23/11	Tue 5/24/11
13	Place Concrete	3 days	Wed 5/25/11	Fri 5/27/11
14	Finish Concrete	3 days	Mon 5/30/11	Wed 6/1/11



Project: Project1
Date: Wed 4/6/11



Page 1

Concrete Cost Analysis

Data Release : Year 2007

Unit Cost Estimate

Quantity	LineNumber	Description	Unit	Material	Labor	Equipment	Total	Ext. Total	Ext. Total O&P
2200	033105350413	Structural concrete, ready mix, normal weight, 10000 psi, includes local aggregate, sand, portland cement and water, excludes all additives and treatments	C.Y.	$290.70	$ -	$ -	$ 290.70	$ 639,540.00	$ 703,494.00
700	033105350562	Structural concrete, ready mix, normal weight, high early, 8000 PSI, includes local aggregate, sand, portland cement and water, excludes all additives and treatments	C.Y.	$215.12	$ -	$ -	$ 215.12	$ 150,584.00	$ 166,187.00
16445	033105350560	Structural concrete, ready mix, normal weight, high early, 5000 psi, includes local aggregate, sand, portland cement and water, excludes all additives and treatments	C.Y.	$116.28	$ -	$ -	$ 116.28	$ 1,912,224.60	$ 2,103,479.95
60000	033529300250	Concrete finishing, floors, monolithic, machine trowel finish	S.F.	$ -	$ 0.49	$ -	$ 0.49	$ 29,400.00	$ 43,200.00
6800	033105700050	Structural concrete, placing, beam, small, elevated, pumped, includes vibrating, excludes material	C.Y.	$ -	$ 31.35	$ 14.29	$ 45.64	$ 310,352.00	$ 436,356.00
2363	033105701000	Structural concrete, placing, column, square or round, pumped, 36" thick, includes vibrating, excludes material	C.Y.	$ -	$ 13.49	$ 6.12	$ 19.61	$ 46,338.43	$ 65,313.32
500	033105700800	Structural concrete, placing, column, square or round, pumped, 24" thick, includes vibrating, excludes material	C.Y.	$ -	$ 20.43	$ 9.32	$ 29.75	$ 14,875.00	$ 20,820.00
16445	033105703500	Structural concrete, placing, high rise, pumped, more than 5 stories, includes vibrating, excludes material, add per story	C.Y.	$ -	$ 0.90	$ 0.41	$ 1.31	$ 21,542.95	$ 30,094.35
500	032110600100	Reinforcing steel, in place, beams and girders, #3 to #7, A615, grade 60, incl labor for accessories, excl material for accessories	Ton	$855.62	$867.08	$ -	$ 1,722.70	$ 861,350.00	$ 1,180,255.00
365	032110600200	Reinforcing steel, in place, columns, #3 to #7, A615, grade 60, incl labor for accessories, excl material for accessories	Ton	$855.62	$924.88	$ -	$ 1,780.50	$ 649,882.50	$ 899,947.65
40	032110600250	Reinforcing steel, in place, columns, #8 to #18, A615, grade 60, incl labor for accessories, excl material for accessories	Ton	$855.62	$604.33	$ -	$ 1,459.95	$ 58,398.00	$ 77,394.40
1100000	032305502250	Prestressing steel, ungrouted single strand, 100' slab, 35 kip, post-tensioned in field	Lb.	$ 0.47	$ 0.96	$ 0.02	$ 1.45	$ 1,595,000.00	$ 2,321,000.00
1000000	032305500350	Prestressing steel, grouted strand, 100' span, 300 kip, post-tensioned in field	Lb.	$ 1.63	$ 0.80	$ 0.03	$ 2.46	$ 2,460,000.00	$ 3,110,000.00
130000	031113202000	C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 1 use, includes shoring, erecting, bracing, stripping and cleaning	SFCA	$ 4.32	$ 5.60	$ -	$ 9.92	$ 1,289,600.00	$ 1,753,700.00
75000	031113256500	C.I.P. concrete forms, column, square, plywood, 24" x 24", 1 use, includes erecting, bracing, stripping and cleaning	SFCA	$ 2.61	$ 5.74	$ -	$ 8.35	$ 626,250.00	$ 885,750.00
120110	031113257000	C.I.P. concrete forms, column, square, plywood, 36" x 36", 1 use, includes erecting, bracing, stripping and cleaning	SFCA	$ 1.93	$ 5.45	$ -	$ 7.38	$ 886,411.80	$ 1,274,367.10
627000	031113351000	C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 1 use, includes shoring, erecting, bracing, stripping and cleaning	S.F.	$ 4.47	$ 3.58	$ -	$ 8.05	$ 5,047,350.00	$ 6,589,770.00
Total								$16599099.28	$21661128.77

Steel Cost Analysis

Data Release : Year 2007 **Unit Cost Estimate**

Quantity		Description	Unit	Material	Labor	Equipment	Total	Ext. Total	Ext. Total O&P
4500	051223771000	Structural steel project, offices, hospitals, etc, 100-ton project, Over 15 stories, A992 steel, shop fabricated, incl shop primer, bolted connections	Ton	$1,971.35	$ 452.25	$ 148.59	$ 2,572.19	############	$ 14,012,955.00
566186	053113505400	Metal decking, steel, non-cellular, composite, galvanized, 2" D, 18 ga	S.F.	$ 1.91	$ 0.49	$ 0.03	$ 2.43	$ 1,375,831.98	$ 1,715,543.58
566186	033529300250	Concrete finishing, floors, monolithic, machine trowel finish	S.F.	$ -	$ 0.49	$ -	$ 0.49	$ 277,431.14	$ 407,653.92
4587	032205500200	Welded wire fabric, sheets, 6 x 6 - W2.1 x W2.1 (8 x 8) 30 lb. per C.S.F., A185	C.S.F.	$ 14.72	$ 22.60	$ -	$ 37.32	$ 171,186.84	$ 243,065.13
6200	033116100760	Structural concrete, ready mix, lightweight, 110 #/C.F., includes lightweight aggregate, sand, portland cement and water, excludes all additives and treatments	C.Y.	$ 129.85	$ -	$ -	$ 129.85	$ 805,070.00	$ 883,128.00
940000	078116100400	Cementitious Fireproofing, sprayed mineral fiber or cementious for fireproofing, beams, 1 hour rated, 1-3/8" thick, excl. tamping or canvas protection	S.F.	$ 0.46	$ 0.52	$ 0.09	$ 1.07	$ 1,005,800.00	$ 1,306,600.00
350000	078116100800	Cementitious Fireproofing, sprayed mineral fiber or cementious for fireproofing, columns, 4 hour rated, 2-3/16" thick, excl. tamping or canvas protection	S.F.	$ 0.98	$ 1.11	$ 0.19	$ 2.28	$ 798,000.00	$ 1,050,000.00
Total								**$16008174.96**	**$19618945.63**

APPENDIX F: RAM Model Outputs

Concrete Column Design Summary

RAM Concrete Column v14.03.01.00
Database: ThesisRAM2222
Building Code: IBC
04/02/11 22:33:01
Concrete Code: ACI 318-08
Academic License. Not For Commercial Use.

Location: 6 - B

No.	Level	Section	f'c	Longitudinal	Rho %	Ld/Cap	Transverse	Ld/Cap
92	13th	24x24	10.00	18-#6 (5 x 4)	1.37	0.38	#3@ 12.0" 0'-0"-15'-0"	0.16
92	12th	24x24	10.00	18-#6 (5 x 4)	1.37	0.17	#3@ 12.0" 0'-0"-14'-6"	0.10
92	11th	24x24	10.00	18-#6 (5 x 4)	1.37	0.24	#3@ 12.0" 0'-0"-14'-6"	0.09
92	10th	24x24	10.00	18-#6 (5 x 4)	1.37	0.32	#3@ 12.0" 0'-0"-13'-6"	0.09
92	9th	24x24	10.00	18-#6 (5 x 4)	1.37	0.40	#3@ 12.0" 0'-0"-13'-6"	0.08
92	8th	24x24	10.00	18-#6 (5 x 4)	1.37	0.48	#3@ 12.0" 0'-0"-13'-6"	0.07
92	7th	24x24	10.00	18-#6 (5 x 4)	1.37	0.56	#3@ 12.0" 0'-0"-13'-6"	0.07
92	6th	24x24	10.00	18-#6 (5 x 4)	1.37	0.64	#3@ 12.0" 0'-0"-13'-6"	0.06
92	5th	24x24	10.00	18-#6 (5 x 4)	1.37	0.72	#3@ 12.0" 0'-0"-13'-6"	0.06
92	4th	24x24	10.00	18-#6 (5 x 4)	1.37	0.81	#3@ 12.0" 0'-0"-13'-6"	0.06
92	3rd	24x24	10.00	18-#6 (5 x 4)	1.37	0.89	#3@ 12.0" 0'-0"-13'-6"	0.05
92	2nd Mezz	30x30	10.00	18-#7 (5 x 4)	1.20	0.63	#3@ 12.0" 0'-0"-13'-6"	0.06
92	2nd	30x30	10.00	18-#7 (5 x 4)	1.20	0.68	#3@ 12.0" 0'-0"-11'-6"	0.06
92	Ground Mezz	30x30	10.00	18-#7 (5 x 4)	1.20	0.73	#3@ 12.0" 0'-0"-12'-6"	0.04

Concrete Column Design Summary

RAM Concrete Column v14.03.01.00
Database: ThesisRAM2222
Building Code: IBC
04/02/11 22:33:01
Concrete Code: ACI 318-08
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Location: 6 - C

No.	Level	Section	f'c	Longitudinal	Rho %	Ld/Cap	Transverse	Ld/Cap
6	Roof	18x18	7.00	18-#4 (5 x 4)	1.11	0.25	#3@ 6.0" 0'-0"-12'-6"	0.07
105	23rd	18x18	7.00	18-#4 (5 x 4)	1.11	0.20	#3@ 6.0" 0'-0"-12'-6"	0.07
105	22nd	18x18	7.00	18-#4 (5 x 4)	1.11	0.32	#3@ 6.0" 0'-0"-11'-6"	0.07
105	21st	18x18	7.00	18-#4 (5 x 4)	1.11	0.44	#3@ 6.0" 0'-0"-11'-6"	0.06
105	20th	18x18	7.00	18-#4 (5 x 4)	1.11	0.56	#3@ 6.0" 0'-0"-11'-6"	0.06
105	19th	18x18	7.00	18-#4 (5 x 4)	1.11	0.67	#3@ 6.0" 0'-0"-11'-6"	0.04
105	18th	24x24	7.00	18-#6 (5 x 4)	1.37	0.44	#3@ 12.0" 0'-0"-11'-6"	0.07
105	17th	24x24	7.00	18-#6 (5 x 4)	1.37	0.50	#3@ 12.0" 0'-0"-11'-6"	0.05
105	16th	24x24	7.00	18-#6 (5 x 4)	1.37	0.57	#3@ 12.0" 0'-0"-11'-6"	0.05
105	15th	24x24	7.00	18-#6 (5 x 4)	1.37	0.63	#3@ 12.0" 0'-0"-11'-6"	0.04
105	14th	24x24	10.00	18-#6 (5 x 4)	1.37	0.51	#3@ 12.0" 0'-0"-11'-6"	0.02
102	13th	24x24	10.00	18-#6 (5 x 4)	1.37	0.60	#3@ 12.0" 0'-0"-15'-0"	0.04
102	12th	24x24	10.00	18-#6 (5 x 4)	1.37	0.69	#3@ 12.0" 0'-0"-14'-6"	0.03
102	11th	30x30	10.00	18-#7 (5 x 4)	1.20	0.51	#3@ 12.0" 0'-0"-14'-6"	0.04
102	10th	30x30	10.00	18-#7 (5 x 4)	1.20	0.57	#3@ 12.0" 0'-0"-13'-6"	0.04
102	9th	30x30	10.00	18-#7 (5 x 4)	1.20	0.63	#3@ 12.0" 0'-0"-13'-6"	0.03
102	8th	30x30	10.00	18-#7 (5 x 4)	1.20	0.69	#3@ 12.0" 0'-0"-13'-6"	0.03
102	7th	30x30	10.00	18-#7 (5 x 4)	1.20	0.75	#3@ 12.0" 0'-0"-13'-6"	0.03
102	6th	30x30	10.00	18-#7 (5 x 4)	1.20	0.81	#3@ 12.0" 0'-0"-13'-6"	0.03
102	5th	36x36	10.00	18-#8 (5 x 4)	1.10	0.61	#3@ 15.0" 0'-0"-13'-6"	0.03
102	4th	36x36	10.00	18-#8 (5 x 4)	1.10	0.66	#3@ 15.0" 0'-0"-13'-6"	0.03
102	3rd	36x36	10.00	18-#8 (5 x 4)	1.10	0.70	#3@ 15.0" 0'-0"-13'-6"	0.03
102	2nd Mezz	36x36	10.00	18-#8 (5 x 4)	1.10	0.74	#3@ 15.0" 0'-0"-13'-6"	0.03
102	2nd	36x36	10.00	18-#8 (5 x 4)	1.10	0.79	#3@ 15.0" 0'-0"-11'-6"	0.03
102	Ground Mezz	36x36	10.00	18-#8 (5 x 4)	1.10	0.83	#3@ 15.0" 0'-0"-12'-6"	0.02

Concrete Column Design Summary

RAM Concrete Column v14.03.01.00
Database: ThesisRAM2222
Building Code: IBC
04/02/11 22:33:01
Concrete Code: ACI 318-08

Location: 6 - F

No.	Level	Section	f'c	Longitudinal	Rho %	Ld/Cap	Transverse	Ld/Cap
30	Roof	18x18	7.00	18-#4 (5 x 4)	1.11	0.43	#3@ 6.0" 0'-0"-12'-6"	0.15
77	23rd	18x18	7.00	18-#4 (5 x 4)	1.11	0.31	#3@ 6.0" 0'-0"-12'-6"	0.13
77	22nd	18x18	7.00	18-#4 (5 x 4)	1.11	0.51	#3@ 6.0" 0'-0"-11'-6"	0.13
77	21st	18x18	7.00	18-#4 (5 x 4)	1.11	0.70	#3@ 6.0" 0'-0"-11'-6"	0.11
77	20th	18x18	7.00	18-#4 (5 x 4)	1.11	0.90	#3@ 6.0" 0'-0"-11'-6"	0.10
77	19th	18x18	7.00	18-#6 (5 x 4)	2.44	0.99	#3@ 12.0" 0'-0"-11'-6"	0.09
77	18th	24x24	7.00	18-#6 (5 x 4)	1.37	0.71	#3@ 12.0" 0'-0"-11'-6"	0.12
77	17th	24x24	7.00	18-#6 (5 x 4)	1.37	0.82	#3@ 12.0" 0'-0"-11'-6"	0.09
77	16th	24x24	7.00	18-#6 (5 x 4)	1.37	0.93	#3@ 12.0" 0'-0"-11'-6"	0.08
77	15th	24x24	7.00	18-#8 (5 x 4)	2.47	0.96	#3@ 15.0" 0'-0"-11'-6"	0.08
77	14th	24x24	10.00	18-#6 (5 x 4)	1.37	0.84	#3@ 12.0" 0'-0"-11'-6"	0.07
77	13th	24x24	10.00	18-#6 (5 x 4)	1.37	0.92	#3@ 12.0" 0'-0"-15'-0"	0.05
77	12th	24x24	10.00	18-#7 (5 x 4)	1.88	0.97	#3@ 12.0" 0'-0"-14'-6"	0.04
77	11th	30x30	10.00	18-#7 (5 x 4)	1.20	0.70	#3@ 12.0" 0'-0"-14'-6"	0.05
77	10th	30x30	10.00	18-#7 (5 x 4)	1.20	0.75	#3@ 12.0" 0'-0"-13'-6"	0.05
77	9th	30x30	10.00	18-#7 (5 x 4)	1.20	0.81	#3@ 12.0" 0'-0"-13'-6"	0.04
77	8th	30x30	10.00	18-#7 (5 x 4)	1.20	0.86	#3@ 12.0" 0'-0"-13'-6"	0.04
77	7th	30x30	10.00	18-#7 (5 x 4)	1.20	0.92	#3@ 12.0" 0'-0"-13'-6"	0.04
77	6th	30x30	10.00	18-#7 (5 x 4)	1.20	0.97	#3@ 12.0" 0'-0"-13'-6"	0.04
77	5th	36x36	10.00	18-#8 (5 x 4)	1.10	0.72	#3@ 15.0" 0'-0"-13'-6"	0.05
77	4th	36x36	10.00	18-#8 (5 x 4)	1.10	0.75	#3@ 15.0" 0'-0"-13'-6"	0.04
77	3rd	36x36	10.00	18-#8 (5 x 4)	1.10	0.79	#3@ 15.0" 0'-0"-13'-6"	0.04
77	2nd Mezz	36x36	10.00	18-#8 (5 x 4)	1.10	0.83	#3@ 15.0" 0'-0"-13'-6"	0.04
77	2nd	36x36	10.00	18-#8 (5 x 4)	1.10	0.87	#3@ 15.0" 0'-0"-11'-6"	0.05
77	Ground Mezz	36x36	10.00	18-#8 (5 x 4)	1.10	0.91	#3@ 15.0" 0'-0"-12'-6"	0.03

Concrete Column Design Summary

RAM Concrete Column v14.03.01.00
Database: ThesisRAM2222
Building Code: IBC
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04/02/11 22:33:01
Concrete Code: ACI 318-08

Location: 6 - D

No.	Level	Section	f'c	Longitudinal	Rho %	Ld/Cap	Transverse	Ld/Cap
18	Roof	18x18	7.00	18-#4 (5 x 4)	1.11	0.29	#3@ 6.0" 0'-0"-12'-6"	0.09
112	23rd	18x18	7.00	18-#4 (5 x 4)	1.11	0.33	#3@ 6.0" 0'-0"-12'-6"	0.08
112	22nd	18x18	7.00	18-#4 (5 x 4)	1.11	0.56	#3@ 6.0" 0'-0"-11'-6"	0.08
112	21st	18x18	7.00	18-#4 (5 x 4)	1.11	0.78	#3@ 6.0" 0'-0"-11'-6"	0.07
112	20th	18x18	7.00	18-#5 (5 x 4)	1.72	0.96	#3@ 9.0" 0'-0"-11'-6"	0.07
112	19th	24x24	7.00	18-#6 (5 x 4)	1.37	0.68	#3@ 12.0" 0'-0"-11'-6"	0.04
112	18th	24x24	7.00	18-#6 (5 x 4)	1.37	0.80	#3@ 12.0" 0'-0"-11'-6"	0.08
112	17th	24x24	7.00	18-#6 (5 x 4)	1.37	0.93	#3@ 12.0" 0'-0"-11'-6"	0.06
112	16th	24x24	7.00	18-#8 (5 x 4)	2.47	0.97	#3@ 15.0" 0'-0"-11'-6"	0.05
112	15th	24x24	7.00	18-#10 (5 x 4)	3.97	0.98	#3@ 18.0" 0'-0"-11'-6"	0.05
112	14th	24x24	10.00	18-#6 (5 x 4)	1.37	0.95	#3@ 12.0" 0'-0"-11'-6"	0.05
109	13th	24x24	10.00	18-#8 (5 x 4)	2.47	0.98	#3@ 15.0" 0'-0"-15'-0"	0.02
109	12th	24x24	10.00	18-#10 (5 x 4)	3.97	0.99	#3@ 18.0" 0'-0"-14'-6"	0.02
109	11th	30x30	10.00	18-#7 (5 x 4)	1.20	0.80	#3@ 12.0" 0'-0"-14'-6"	0.04
109	10th	30x30	10.00	18-#7 (5 x 4)	1.20	0.86	#3@ 12.0" 0'-0"-13'-6"	0.03
109	9th	30x30	10.00	18-#7 (5 x 4)	1.20	0.92	#3@ 12.0" 0'-0"-13'-6"	0.03
109	8th	30x30	10.00	18-#7 (5 x 4)	1.20	0.98	#3@ 12.0" 0'-0"-13'-6"	0.03
109	7th	30x30	10.00	18-#9 (5 x 4)	2.00	0.99	#3@ 18.0" 0'-0"-13'-6"	0.03
109	6th	30x30	10.00	18-#11 (5 x 4)	3.12	0.99	#3@ 18.0" 0'-0"-13'-6"	0.02
109	5th	36x36	10.00	18-#8 (5 x 4)	1.10	0.81	#3@ 15.0" 0'-0"-13'-6"	0.03
109	4th	36x36	10.00	18-#8 (5 x 4)	1.10	0.85	#3@ 15.0" 0'-0"-13'-6"	0.02
109	3rd	36x36	10.00	18-#8 (5 x 4)	1.10	0.90	#3@ 15.0" 0'-0"-13'-6"	0.03
109	2nd Mezz	36x36	10.00	18-#8 (5 x 4)	1.10	0.94	#3@ 15.0" 0'-0"-13'-6"	0.02
109	2nd	36x36	10.00	18-#8 (5 x 4)	1.10	0.98	#3@ 15.0" 0'-0"-11'-6"	0.03
109	Ground Mezz	36x36	10.00	18-#10 (5 x 4)	1.76	0.99	#3@ 18.0" 0'-0"-12'-6"	0.02

APPENDIX G: ETABS Outputs

Modes

Modal Participation Factors

Edit View

Modal Participation Factors

Mode	Period	UX	UY	UZ	RX	RY	RZ	ModalMass
1	3.370985	0.218583	15.124689	0.000000	-40425.8178	664.636109	1037.134326	1.000000
2	2.653829	7.500386	-0.949728	0.000000	2731.568689	22654.138812	13325.913225	1.000000
3	2.346877	9.310471	0.441568	0.000000	-1278.627335	27978.294784	-8377.28936	1.000000
4	1.205619	0.047093	-7.425176	0.000000	6807.104602	294.817167	-4046.83159	1.000000
5	1.090516	-0.296534	3.075303	0.000000	-2944.611610	-135.619930	-9281.25959	1.000000
6	0.936632	-11.095550	-0.078150	0.000000	84.434927	-18808.08672	1701.128730	1.000000
7	0.589022	-4.921200	-0.623002	0.000000	359.757332	-6861.78947	-861.739314	1.000000
8	0.571632	-0.940655	3.649850	0.000000	-2245.270315	-1308.092080	1876.487677	1.000000
9	0.532495	0.334053	1.709298	0.000000	-1161.280018	479.061947	-3892.85887	1.000000
10	0.417027	0.203891	4.608590	0.000000	-1634.709603	165.247830	2827.328100	1.000000
11	0.385231	0.413662	-3.515587	0.000000	1267.117258	305.421080	3965.081656	1.000000
12	0.338819	-2.513265	-0.111740	0.000000	42.209756	-1564.586193	1020.140745	1.000000

OK

Wind E/W

DISPLACEMENTS AND DRIFTS AT POINT OBJECT 90

File

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
MAIN ROOF	0.732536	-0.093856	0.000179	0.000019
STORY23	0.705745	-0.090932	0.000230	0.000020
STORY22	0.671218	-0.087926	0.000283	0.000020
STORY21	0.632182	-0.085158	0.000339	0.000020
STORY20	0.585432	-0.082389	0.000396	0.000020
STORY19	0.530821	-0.079616	0.000450	0.000020
STORY18	0.468742	-0.076819	0.000496	0.000021
STORY17	0.400239	-0.073940	0.000525	0.000022
STORY16	0.327835	-0.070852	0.000509	0.000026
STORY15	0.257631	-0.067318	0.000384	0.000031
STORY14	0.204674	-0.062987	0.000128	0.000040
STORY13	0.186963	-0.057506	0.000105	0.000033
STORY12	0.167985	-0.051644	0.000106	0.000032
STORY11	0.149527	-0.046118	0.000105	0.000031
STORY10	0.131277	-0.040737	0.000103	0.000030
STORY9	0.114596	-0.035860	0.000100	0.000029
STORY8	0.098323	-0.031103	0.000097	0.000029
STORY7	0.082603	-0.026480	0.000093	0.000028
STORY6	0.067595	-0.022015	0.000087	0.000026
STORY5	0.053474	-0.017739	0.000081	0.000025
STORY4	0.040432	-0.013692	0.000073	0.000023
STORY3	0.028681	-0.009927	0.000063	0.000021
STORY2M	0.018447	-0.006513	0.000052	0.000018
STORY2	0.010004	-0.003555	0.000040	0.000015
STORY1M	0.003564	-0.001180	0.000026	0.000009

Seismic E/W

DISPLACEMENTS AND DRIFTS AT POINT OBJEC...

File

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
MAIN ROOF	2.689815	-0.355831	0.000695	0.000070
STORY23	2.585630	-0.345307	0.000926	0.000073
STORY22	2.446678	-0.334411	0.001169	0.000073
STORY21	2.285352	-0.324329	0.001417	0.000073
STORY20	2.089737	-0.314209	0.001659	0.000074
STORY19	1.860759	-0.304055	0.001878	0.000074
STORY18	1.601650	-0.293780	0.002051	0.000077
STORY17	1.318563	-0.283140	0.002137	0.000084
STORY16	1.023723	-0.271589	0.002029	0.000098
STORY15	0.743744	-0.258100	0.001468	0.000124
STORY14	0.541212	-0.241030	0.000392	0.000157
STORY13	0.487058	-0.219391	0.000294	0.000127
STORY12	0.434187	-0.196494	0.000294	0.000124
STORY11	0.383010	-0.174996	0.000287	0.000120
STORY10	0.333013	-0.154147	0.000279	0.000116
STORY9	0.287857	-0.135327	0.000268	0.000113
STORY8	0.244373	-0.117042	0.000256	0.000109
STORY7	0.202947	-0.099353	0.000241	0.000105
STORY6	0.163986	-0.082346	0.000223	0.000100
STORY5	0.127911	-0.066137	0.000202	0.000094
STORY4	0.095166	-0.050872	0.000179	0.000087
STORY3	0.066221	-0.036743	0.000152	0.000079
STORY2M	0.041567	-0.024007	0.000122	0.000068
STORY2	0.021779	-0.013038	0.000089	0.000054
STORY1M	0.007309	-0.004298	0.000053	0.000031

Wind N/S

DISPLACEMENTS AND DRIFTS AT POINT OBJECT 71

File

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
MAIN ROOF	0.087850	5.930072	0.000008	0.001365
STORY23	0.086674	5.725365	0.000011	0.001564
STORY22	0.085019	5.490738	0.000015	0.001756
STORY21	0.083001	5.248392	0.000018	0.001959
STORY20	0.080459	4.978059	0.000022	0.002166
STORY19	0.077383	4.679220	0.000026	0.002367
STORY18	0.073785	4.352571	0.000030	0.002553
STORY17	0.069698	4.000314	0.000033	0.002696
STORY16	0.065210	3.628317	0.000034	0.002718
STORY15	0.060577	3.253251	0.000029	0.002384
STORY14	0.056561	2.924250	0.000020	0.001661
STORY13	0.053741	2.695090	0.000021	0.001623
STORY12	0.050022	2.402917	0.000021	0.001603
STORY11	0.046350	2.123942	0.000022	0.001574
STORY10	0.042495	1.850092	0.000023	0.001532
STORY9	0.038769	1.601882	0.000024	0.001483
STORY8	0.034859	1.361682	0.000025	0.001420
STORY7	0.030775	1.131631	0.000026	0.001343
STORY6	0.026540	0.914035	0.000027	0.001251
STORY5	0.022191	0.711435	0.000027	0.001141
STORY4	0.017784	0.526631	0.000027	0.001012
STORY3	0.013398	0.362703	0.000026	0.000862
STORY2M	0.009145	0.223107	0.000024	0.000686
STORY2	0.005194	0.111968	0.000021	0.000482
STORY1M	0.001748	0.033919	0.000013	0.000246

Seismic N/S

DISPLACEMENTS AND DRIFTS AT POINT OBJECT 71

File

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
MAIN ROOF	0.055942	4.075979	0.000007	0.000997
STORY23	0.054964	3.926374	0.000009	0.001164
STORY22	0.053580	3.751778	0.000012	0.001323
STORY21	0.051898	3.569237	0.000015	0.001484
STORY20	0.049796	3.364455	0.000018	0.001640
STORY19	0.047291	3.138157	0.000021	0.001783
STORY18	0.044412	2.892123	0.000023	0.001905
STORY17	0.041211	2.629284	0.000025	0.001986
STORY16	0.037775	2.355248	0.000025	0.001969
STORY15	0.034315	2.083514	0.000021	0.001682
STORY14	0.031410	1.851460	0.000014	0.001108
STORY13	0.029505	1.698599	0.000014	0.001072
STORY12	0.027075	1.505697	0.000013	0.001052
STORY11	0.024733	1.322620	0.000014	0.001026
STORY10	0.022335	1.144170	0.000014	0.000991
STORY9	0.020080	0.983651	0.000014	0.000951
STORY8	0.017778	0.829623	0.000014	0.000902
STORY7	0.015443	0.683499	0.000014	0.000844
STORY6	0.013095	0.546759	0.000014	0.000777
STORY5	0.010758	0.420961	0.000014	0.000699
STORY4	0.008463	0.307747	0.000014	0.000610
STORY3	0.006252	0.208858	0.000013	0.000511
STORY2M	0.004178	0.126151	0.000011	0.000397
STORY2	0.002318	0.061765	0.000010	0.000270
STORY1M	0.000755	0.017960	0.000005	0.000130

Center of Rigidity/Mass

Center Mass Rigidity

Edit View

Center Mass Rigidity

Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR
MAIN ROOF	D1	3.2080	3.2080	1834.389	381.390	3.2080	3.2080	1834.389	381.390	1818.005	455.900
STORY23	D1	12.3006	12.3006	1871.184	379.302	15.5087	15.5087	1863.572	379.734	1818.178	462.657
STORY22	D1	12.2430	12.2430	1871.442	379.111	27.7517	27.7517	1867.044	379.459	1818.284	469.629
STORY21	D1	12.1854	12.1854	1871.702	378.917	39.9370	39.9370	1868.465	379.294	1818.372	477.319
STORY20	D1	12.1854	12.1854	1871.702	378.917	52.1224	52.1224	1869.222	379.206	1818.469	487.096
STORY19	D1	12.1854	12.1854	1871.702	378.917	64.3078	64.3078	1869.692	379.151	1818.576	499.896
STORY18	D1	12.1854	12.1854	1871.702	378.917	76.4931	76.4931	1870.012	379.114	1818.689	517.022
STORY17	D1	12.1854	12.1854	1871.702	378.917	88.6785	88.6785	1870.244	379.087	1818.778	540.266
STORY16	D1	12.1854	12.1854	1871.702	378.917	100.8638	100.8638	1870.421	379.066	1818.734	571.272
STORY15	D1	12.1854	12.1854	1871.702	378.917	113.0492	113.0492	1870.559	379.050	1818.241	607.224
STORY14	D1	17.7433	17.7433	1908.969	612.621	130.7925	130.7925	1875.769	410.736	1816.802	628.415
STORY13	D1	18.2070	18.2070	1908.965	619.801	148.9996	148.9996	1879.826	436.283	1815.619	628.810
STORY12	D1	18.4323	18.4323	1908.299	619.747	167.4319	167.4319	1882.960	456.480	1814.338	628.923
STORY11	D1	18.3948	18.3948	1908.409	619.756	185.8267	185.8267	1885.480	472.643	1813.013	629.022
STORY10	D1	18.3197	18.3197	1908.630	619.774	204.1464	204.1464	1887.557	485.846	1811.679	629.105
STORY9	D1	18.2446	18.2446	1908.853	619.792	222.3909	222.3909	1889.304	496.835	1810.420	629.179
STORY8	D1	18.2446	18.2446	1908.853	619.792	240.6355	240.6355	1890.786	506.157	1809.208	629.249
STORY7	D1	18.2446	18.2446	1908.853	619.792	258.8801	258.8801	1892.060	514.166	1808.066	629.315
STORY6	D1	18.2446	18.2446	1908.853	619.792	277.1247	277.1247	1893.165	521.120	1807.016	629.377
STORY5	D1	18.2446	18.2446	1908.853	619.792	295.3693	295.3693	1894.134	527.214	1806.083	629.435
STORY4	D1	18.2446	18.2446	1908.853	619.792	313.6139	313.6139	1894.990	532.600	1805.304	629.486
STORY3	D1	18.2446	18.2446	1908.853	619.792	331.8584	331.8584	1895.753	537.394	1804.748	629.526
STORY2M	D1	18.2446	18.2446	1908.853	619.792	350.1030	350.1030	1896.435	541.688	1804.557	629.549
STORY2	D1	18.2446	18.2446	1908.853	619.792	368.3476	368.3476	1897.050	545.556	1805.184	629.492
STORY1M	D1	18.0944	18.0944	1909.305	619.829	386.4420	386.4420	1897.624	549.034	1811.230	629.552