



**Belmont Executive Center**  
**Building A**  
**Ashburn, VA**

**Final Thesis Report**  
**Nicholas L. Ziegler: Structural**  
**Advisor: Professor M. Kevin Parfitt**  
**April 7, 2010**

# Belmont Executive Center, Building A

19755 Belmont Executive Plaza  
 Ashburn, VA 20147



## Project Team

<b>Owner</b>	Toll Brothers Commercial
<b>General Contractor</b>	L.F. Jennings
<b>Architect</b>	The M Group Architects
<b>Civil Engineer</b>	Eastern States Engineering Inc.
<b>Structural Engineer</b>	Haynes Whaley and Associates Inc.
<b>MEP Engineer</b>	KTA Group Inc.
<b>Landscape Architect</b>	Urban Engineering & Associates

## Project Information

<b>Location</b>	Ashburn, VA
<b>Occupancy</b>	Business
<b>Total Size</b>	124,661 SF
<b>Stories</b>	5
<b>Dates of Construction</b>	9/21/06 - 7/19/07
<b>Contract Type</b>	Guaranteed Maximum Price

## Structural System

- Spread footings with perimeter grade beam
- First floor is a 5" slab on grade reinforced with #5 bars @ 15" o.c.
- Steel frame construction
- Floor system comprised of 6-1/4" of lightweight concrete on composite steel deck reinforced with welded wire fabric
- 4 diagonally braced lateral resisting frame
- Open web joists supporting roof



## Architecture

- Flexible multi-tenant office space
- Primarily face brick construction with Arristercraft CMU block accents
- EFCO series curtain walls in center of building and ends
- Exterior terrace located on the backside, accessible from the fifth floor
- Mechanical equipment shielded behind a metal roof screen

## Lighting/Electrical

- 277/480 volt, 3 phase, 4 wire switchboard with 4000 amp bus duct
- A 600kW, 480V, 3phase emergency diesel generator with a 900A output circuit breaker

## Mechanical/Plumbing Systems

- 2 14000 CFM self contained units per floor with a variable control system supplying heating and cooling to north and south zones of building
- Individual wall heaters supplement heating load
- 1 344 ton cooling tower with heat exchanger
- Individual water heater/chiller per floor

**Nicholas L. Ziegler: Structural Option**

[www.engr.psu.edu/ae/thesis/portfolios/2010/nlz5003/index.html](http://www.engr.psu.edu/ae/thesis/portfolios/2010/nlz5003/index.html)

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Professor Ali Memari

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Most importantly, a special thanks to friends and family who provided support throughout the project.

## Executive Summary

A redesign of the building structure was completed using post-tensioned concrete. To minimize the structural depth, a one-way post-tensioned slab was designed in the long direction of the building, supported by wide shallow post-tensioned girders in the short direction. Between the girders the centerline spacing was 30'-0", therefore post-tensioning for an 8" slab was designed using ADAPT PT. The slab was divided into 4 different zones, based on the number of spans and span distances, and individually designed. A hand calculation verified the amount of conventional reinforcement determined by the program. 60"x20" typical girders were then designed to support the slab. Only 3 individual girders were specifically designed.

1. A typical girder
2. A girder adjacent to two slab openings
3. A 5<sup>th</sup> floor transfer girder

The second of the three girders provided a unique case, due to the lack of an effective flange width, which helps decrease the precompression stress. To limit the precompression stress, the amount of tendons in each span had to be varied. Roof loads are transferred at the back of the building to the PT girders. It was necessary to see if the girder was capable of supporting the column's axial load. A design was achieved, but the girder depth had to be increased to 24" in the third span.

Concrete moment frames were then used to resist the lateral loads on the buildings. All nine frames in the short direction of the building were used as moment frames, while exterior perimeter beams were introduced in the long direction. An ETABS model of the lateral system was created, with reduced moment of inertias of 0.70 for columns and 0.40 for beams. Maximum deflections from ASCE 7-05 wind load case 1 were successfully limited to L/400. Story drifts created by the increased seismic loads were well under the allowable standard of  $0.02 \cdot h_{sx}$ .

Torsional capacity of the transfer girders was then determined sufficient enough to resist the ultimate torsional load without additional reinforcement. A column part of the lateral system was then designed, considering P- $\Delta$  effects. A 24"x24" column with a concrete strength of 5000 psi was capable of supporting the load with (12) #11's. The percent of steel exceeds the 2%, requiring some form of mechanical rebar splice.

Cost and schedule comparisons performed between the new and existing structure. Only the structure itself was investigated. The steel framing was both more cost efficient and quicker to erect. A total of 70 days was required for the concrete system, while the steel only required 50 days.

## Building Overview

Building A is a 125,000 SF office building, located in Ashburn, VA, just north of the Dulles International Airport. The building is part of the Belmont Executive Center, which when fully completed, will include office, retail, restaurant, daycare, and hotel spaces.

Completed in July of 2007, the building provides flexible, multi-tenant office space. Each of the five floors is subdivided into four areas for a maximum of twenty tenants.

The exterior facade of the building consists primarily of face brick, with portions of curtain wall glass. Spandrel steel angle beams, fastened to the structural columns, support the brick at mid-height on each floor. Light gauge C channel studs supported on each floor, provide backing for the brick.

Around the perimeter of the building, vertical brick column lines are spaced evenly. The central entrance is distinguished from the rest of the building by a large storefront curtain wall that extends the full height of the building. Storefront glazing is also present above level 3 at the corners of the building. Concrete masonry units provide architectural lintels and bases to the vertical brick columns. On the 5<sup>th</sup> floor, the façade steps back to provide an exterior terrace.

The building structure is lightweight concrete on composite metal deck. Composite beam action is achieved by the use of shear studs fastened to the supporting members. The slab is supported by W shape steel beams and columns.



Figure 1: Front of Building A

## Existing Structural System

### Foundation

The foundation system consists of square spread footings with a concrete strength of 3000 psi, located at the base of the steel columns. Footing size ranges from 19'-6" x 19'-6" x 3'-2" to 10'-6" x 10'-6" x 1'-6" square, depending on their location. Larger footings exist at the base of the lateral frames because of the increased axial force due to wind and seismic loads. In all other areas, the foundations are smaller. All larger foundations are shown in yellow in Figure 2 below.

The perimeter footings are connected by a strip footing that supports the first floor masonry facade. The strip footing is stepped in two locations to accommodate utility connections. No sub-grade levels exist in the building. The 1<sup>st</sup> floor is a 5" concrete slab on grade with a concrete strength of 4000 psi, reinforced with #3 rebar @ 15" o.c., running both directions. A 6" slab on grade, also with a strength of 4000 psi, is located to the right side of the building to support a 30 yard trash compactor, and is highlighted in purple in Figure 2. It is reinforced with #3 rebar @ 12" o.c. each way. The slab is supported by 4" granular material, on top of compacted soil with an allowable bearing capacity of 2.5 ksf.

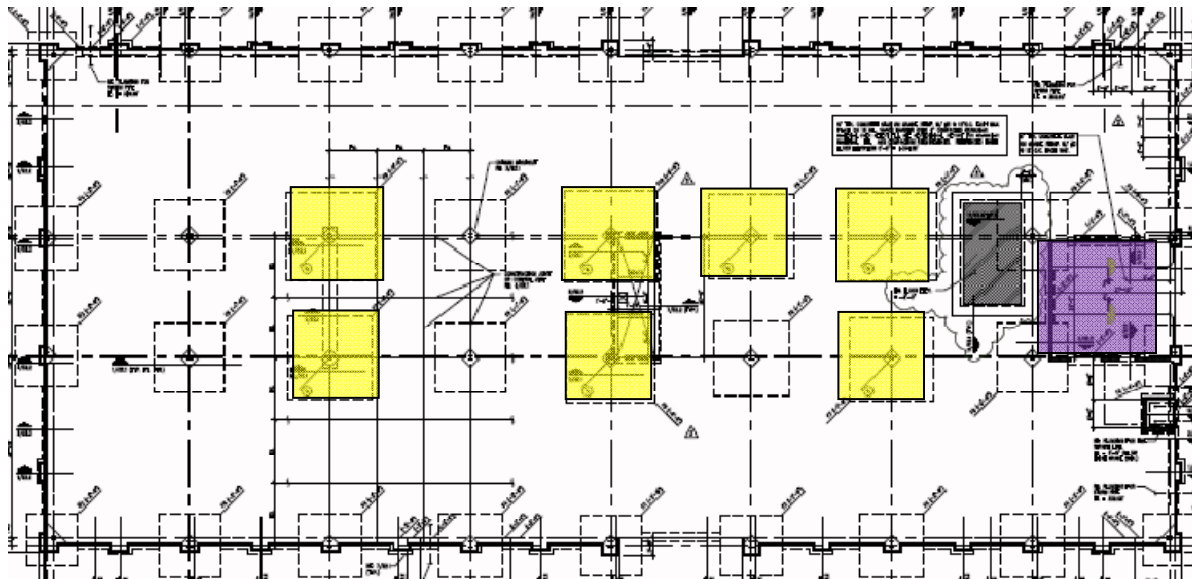


Figure 2: Foundation Plan



## Column System

Elevated slabs are supported by nine column lines in the north-south direction and four in the east-west direction. A majority of the columns are W-shape steel members, spliced 3'-0" above the third floor. In some instances, however, HSS square columns are used. At the front left and right corners of the building, they are used because of dimensions limitations of the slab. HSS columns are also used to transfer roof loads to the fifth floor beams at the exterior terrace. The location of the 5<sup>th</sup> floor HSS

columns is shown highlighted in red; in Figure 3. Typical bay sizes for each floor are 39'-0" x 30'-0", in the outer bays, and 26'-0" x 30'-0" in the interior bays. Because of the exterior architecture of the building, the perimeter columns are not aligned with the interior columns. The slight difference between the grid lines creates a skew in the girders spanning from the exterior to the interior. The skewed beams are shown highlighted in blue on Figure 4.

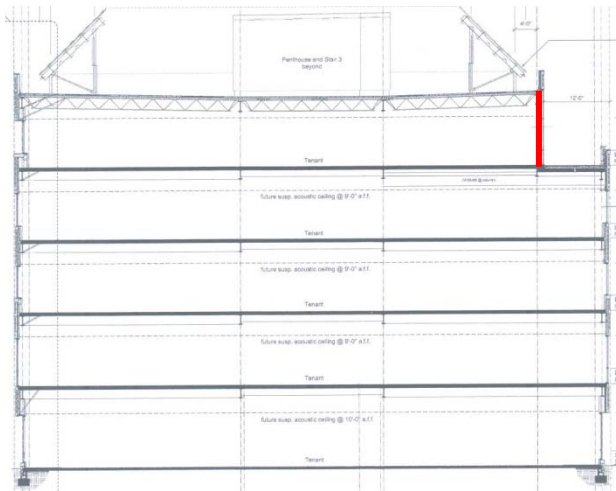


Figure 3: 5th Floor HSS Column Location

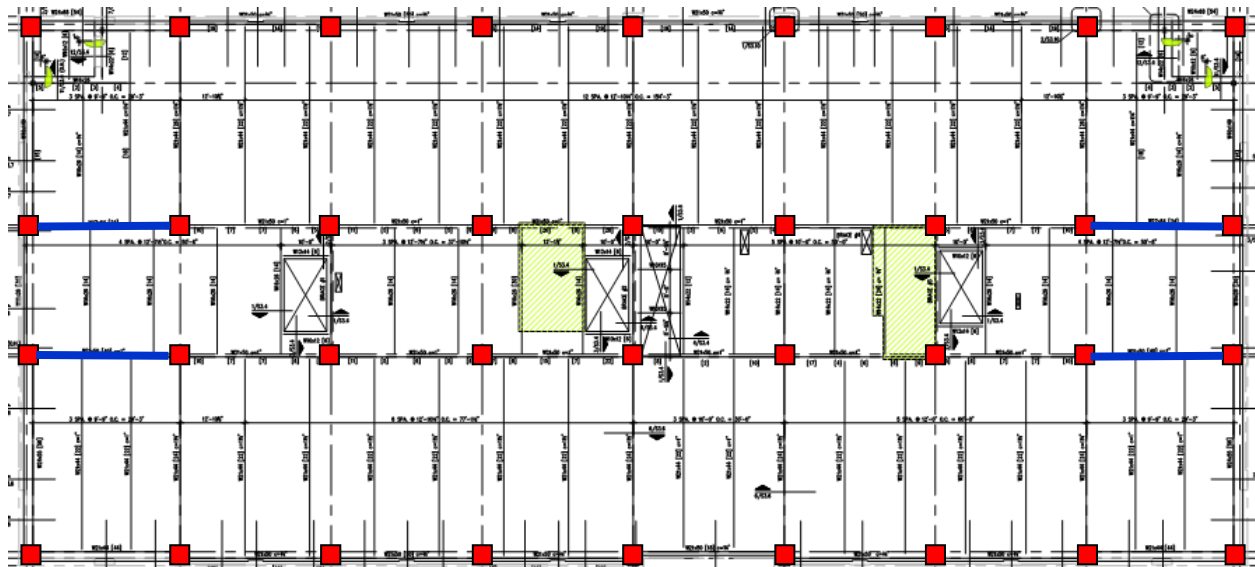


Figure 4: Column Locations

## Floor System

The floor system in Building A consists of 3500 psi, 6 ¼” of light weight concrete, on 3” composite metal deck. Mesh reinforcement is provided by 6”x6”-W1.4 x W1.4 welded wire fabric. Supporting the decks are W-shape beams spaced at roughly 10’-0”.

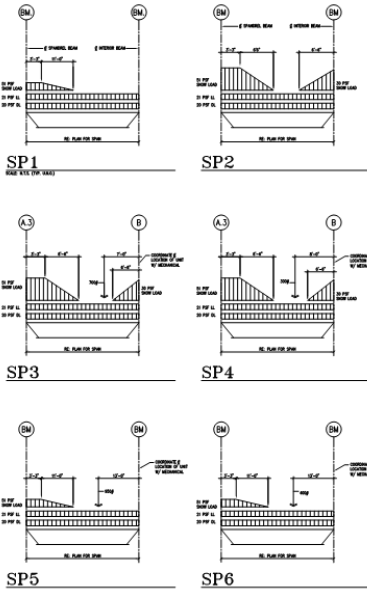


Figure 5: Snow Drift Loadings

In the interior bays and the exterior bays the beams are typically W21x44 and W16x26, respectively. Composite action is achieved through the use of ¾” diameter, 5 ¼” long shear studs, welded to the supporting beams. To limit deflections, most beams also have an upward camber ranging from ¾” to 1”. Support for the beams is provided by W21x50 girders, running in the long direction of the building. Mechanical equipment in the penthouse is supported by a typical concrete floor. W16x26 beams span between girders to support the slab.

## Roof System

The support members of the main roof system consist of K-series joists, spanning over the three bays in the short direction of the building, spaced at 6’-0”. Because of the penthouse and the perimeter parapet, joists in the outer bays were designed specifically by the joist manufacture for snow drifting. There were 6 unique drift conditions for different locations; these are shown in Figure 5.

Regular K series joists, ranging from 22K5 – 18K3, support the roof in the middle bay. The penthouse roof is supported by 20K3 joists @ 6’-0”. In all bays, three rows of bridging prevent lateral torsional buckling.

The metal roof screen, that shields the penthouse from view, is supported by a combination of K series joists and W shape beams. Approximately every 30’-0” around the perimeter of the roof, there are W shape beams angled at 45° from the roof plane. Between these beams, there are four K series joists, running parallel to the building façade. L2”x2”x½” steel angle provides bracing every 6’-0”. Figure 6 shows the W shape beams highlighted in orange, and the K series joists can be seen running parallel to them. Also, Figure 7 shows a cross section of the roof screen, with the W shape beams highlighted in blue.

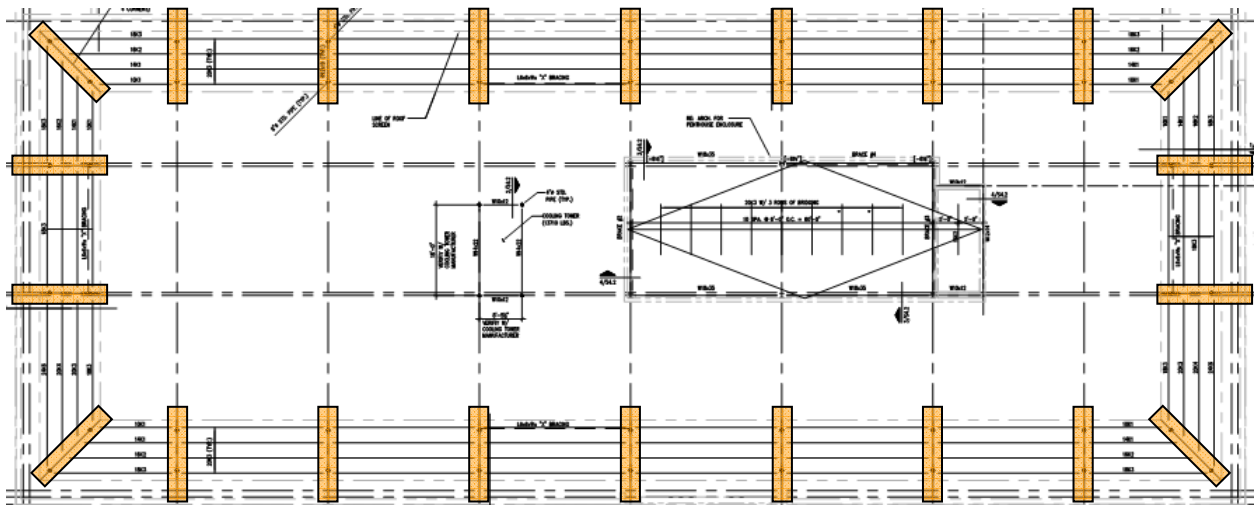


Figure 6: Roof Screen Framing

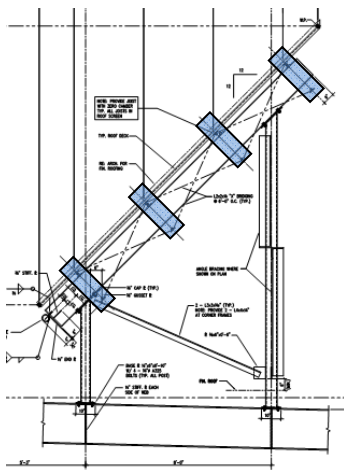


Figure 7: Roof Screen Section

### Lateral System

Lateral loads on Building A are resisted by four braced frames. Three of the frames are located in the north-south direction of the building, to resist the higher wind loads from the broader side. These three frames lie along the column lines, adjacent to the stairwell openings. Much lower loads exist in the other direction; therefore only one braced frame is present. This frame is located along the perimeter of the central mechanical/restroom walls. All frames are braced with hollow structural steel members ranging in size from 8" x 8" x 1/4" at the first floor to 4" x 4" x 1/4" at the fifth floor. Elevations and plan locations for the frames can be seen in Figure 8 and Figure 9.

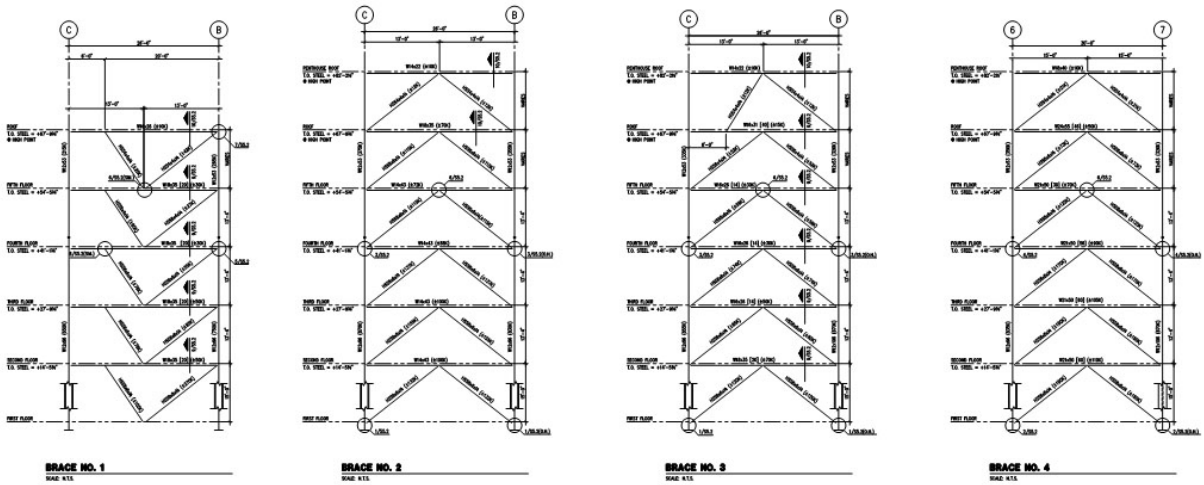


Figure 8: Lateral Frame Elevations

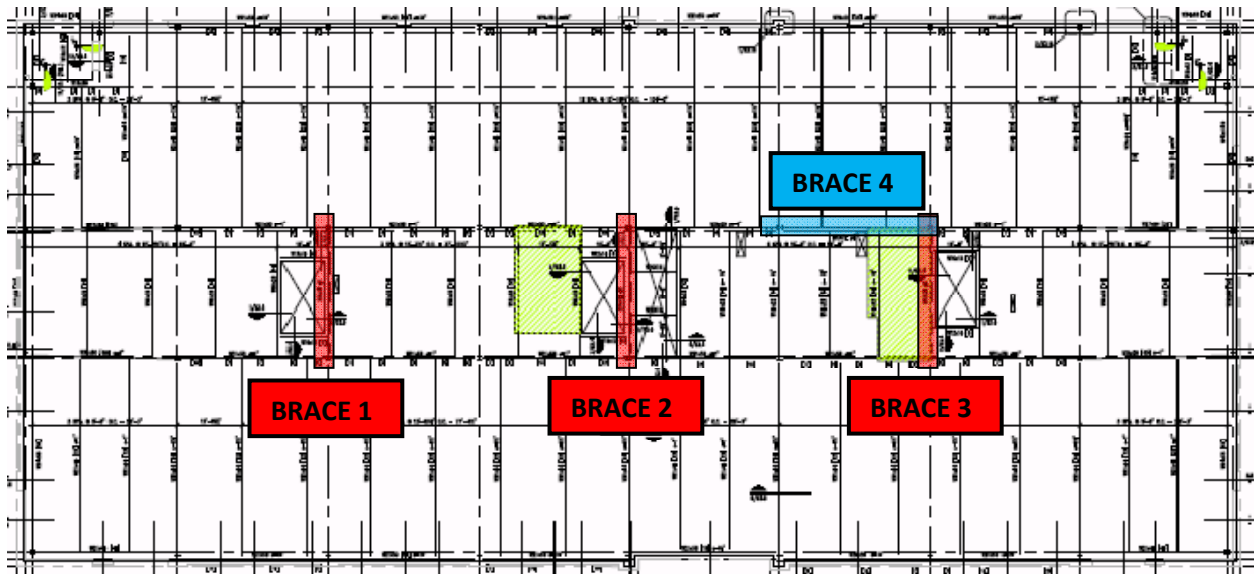


Figure 9: Lateral Frame Locations

## Gravity Loads

### Snow Load

Snow loads were calculated in accordance with ASCE 7-05, Chapter 7. As mentioned earlier, special snow drift conditions were considered for the K series roof joists. Snow drifting against the parapet and the penthouse was considered. One calculation determined that an additional load of 50 - 49 pounds per foot should be applied where drifting occurs. This matches the loading from the structural drawings of 51 psf. The flat roof snow load of 21 psf also matched the load listed in the notes. Detailed snow load calculations can be found in the appendix.

### Dead Loads

Superimposed dead load:  $SDL = 15 \text{ lb/ft}^2$

Exterior façade:  $24 \text{ lb/ft}^2$

Brick:  $DL = 52 \text{ lb/ft}^2$

Glazing:  $DL = 3 \text{ lb/ft}^2$

8" CMU block ungrouted:  $DL = 48 \text{ lb/ft}^2$

Ballasted single ply roof:  $DL = 11 \text{ lb/ft}$

Normal weight concrete:  $DL = 150 \text{ lb/ft}^3$

### Live Load

Mechanical room:  $LL = 125 \text{ psf}$  – nonreducible

All other spaces:  $LL = 100 \text{ psf}$

## Lateral Loads

### Wind Loads

Building wind loads were calculated in accordance to ASCE 7-05, Chapter 6. Analytical method number two was used to determine loads in both directions. For the purpose of this report, a few assumptions were made to simplify the calculations. One, the roof screen height was included into the total building height, and two, the building horizontal projections were assumed to be rectangular. Individual wind effects on the building parapets and roof screen were not taken into consideration. All variables and coefficients used in the calculations are located in the appendix. Wind loads are summarized in Table 1.

Wind Forces			
Y-Direction		X-Direction	
Level	Force (kips)	Level	Force (kips)
RF	100	RF	37.5
5	56.1	5	20.8
4	54.1	4	19.9
3	51.6	3	18.8
2	51.5	2	18.5
All loads are applied at the floor slab.			

Table 1: Wind Loads

### Seismic Loads

Seismic loads were determined using Chapters 11 and 12 from ASCE 7-05. Values for the short period and one second period response accelerations were determined by the USGS computer program and verified with the USGS seismic maps.

Because of the building height, soil class, and response accelerations, the building fell into seismic design category A. Therefore, the building needed only to be designed in accordance to §11.7 of ASCE 7-05.

An average weight per square foot of the building was determined in a typical exterior bay, and multiplied by the total building area. The total building weight (structure+façade) was divided by 5 to determine an approximate weight per floor. Table 2 summarizes the seismic loads at each floor.

Seismic Forces	
Y/X Direction	
Level	Force (kips)
RF	42.5
5	42.5
4	42.5
3	42.5
2	42.5

Table 2: Seismic Loads

## **Proposal**

### **Structural Depth**

#### **Problem Statement**

Building A's current structural system consists of lightweight concrete on composite metal deck, supported by W shape steel beams and columns. The system is very efficient at supporting loads over the long spans that exist in the outer bays, with W21x50 girders being the deepest members in the system. Combination of the member depth and the floor thickness of 6 ¼", creates the greatest floor depth equal to 27". Lateral forces are resisted by steel braced frames and limit both wind and seismic deflections/displacements to acceptable standards. The building location borders the Washington D.C. metro area, where post-tensioned concrete is a common building method. Post-tensioned concrete design will be applied to create a structure, equivalent in performance and floor depth.

#### **Problem Solution**

To limit the structure depth, a post-tensioned system design was proposed. The long spans in the short direction of the building, coupled with the evenly spaced column lines other direction, make a one-way post-tensioned slab, supported by wide-shallow post-tensioned girders, a probable solution. The switch from steel to concrete also required the need for a new lateral system design. Construction of shear walls is relatively expensive, so the proposed PT girders were used as concrete moment frames. Support of the girders and slab was provided by square concrete columns.

### **Architectural Breadth**

The existing slab layout had to be modified to accommodate the wide post-tensioned girders. Because maximum tenant space was desired, it was necessary to examine the impacts of the structural changes. Areas around the three stairwells were investigated to determine if detrimental effects on the office layout occurred.

### **Construction Breadth**

To determine which system would be more cost and schedule efficient, both a cost estimate and schedule for each superstructure type were created. Only the structure itself was considered. Costs associated with foundations were neglected.

## Design of Gravity System

### New Column Layout

The existing supported slabs in Building A, have a very complex profile and do not share common edges. To avoid changes to the exterior architecture, it was required to locate all the columns within the boundaries of the smallest slab. Also, due to load transfer, it was desired to have the columns concentrically located atop each other from floor to floor. Assuming 24"x24" columns, the perimeter columns had to be brought inwards to achieve these goals. The new column layout is shown in Figure 10. Smaller, 12"x24" columns were introduced next to the slab openings. These were required to support normal reinforced concrete beams, which support the CMU shaft walls.

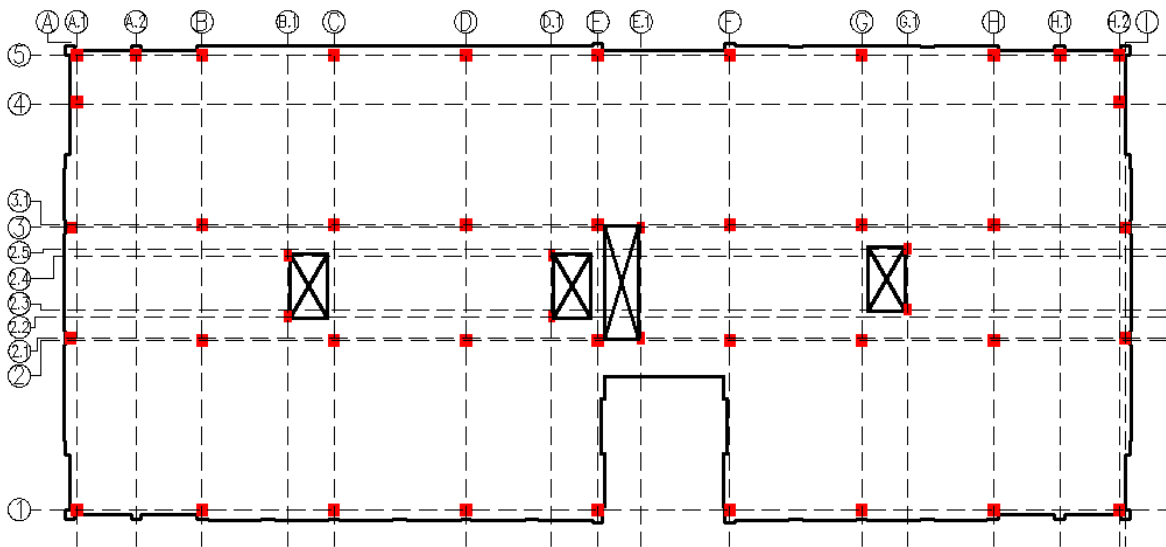


Figure 10: New 2<sup>nd</sup> Floor Column Layout

### Preliminary Slab/Girder Sizes

“Long-span Concrete Floor Systems”, by David Anthony Fanella and existing building plans were referenced to determine a preliminary slab thickness, girder sizes, and column sizes. Both sources suggested that, with a 30'-0" centerline girder spacing, a 100 psf live load, and a 15 psf superimposed dead load, an 8" slab would suffice. Further, 60" wide by 20" deep girders were shown to be adequate with the longest span being 39'-0". To support the slab and girders, 24" square columns were chosen. The sizes of the columns do not change as the building height increases, due to the need for extra lateral reinforcement at the top floor.

To accommodate the wide girders, the stair openings were moved to either the left or the right 7-1/4", depending on which side of the column lines, the openings were on. It was also necessary to move the elevator shaft 3" to the right. Architectural impacts due to the opening movements are investigated in the architectural breadth.



In the middle bay, it was not possible to have a 60" wide continuous cross section. This was because of the opening limitations. However, since the span length was much less, such a large size was not required. The minimum width was limited to the boundaries of the slab openings; the beams are 36" between the stairwell and elevator openings. Girders adjacent to one stairwell were limited to 48" wide. A girder width of 28" was provided at the narrowest section of the exterior girders. The outside edge was truncated at the slab edge. Because a 60" wide beam was said to work with a full tributary width, it was assumed that half the cross section would be capable of supporting half the load. Similar reasoning was applied to the girders around the atrium opening. The wide shallow girders are shown in Figure 11. The exterior edge beam, perpendicular to the girders, was later found to be required to support the exterior façade and to resist lateral loads in the long direction.

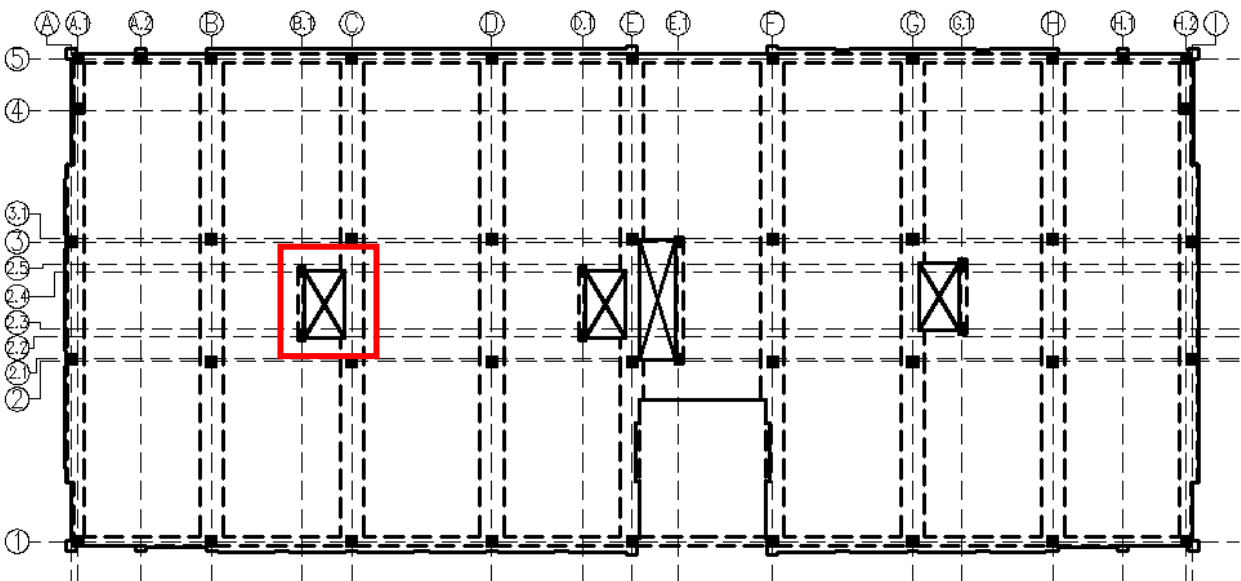


Figure 11: Girder/Beam Plan

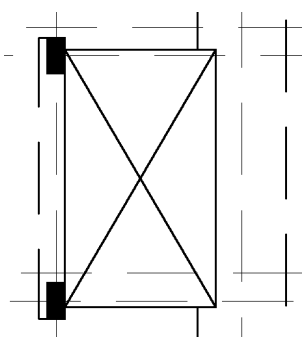


Figure 12: Enlarged Floor Plan

As mentioned earlier, it was required to have a normal reinforced concrete beam, next to edge of the openings because of the CMU shaft walls. Introducing a beam here also eliminated the large cantilever from the girder. One of these beams can be seen in Figure 12, along the opening perimeter.

## Design of Post-Tensioned Slab

### Slab Zones

With these preliminary sizes/span distances, the slab was subdivided into five design areas and given a numerical designation. These zones are shown in Figure 13, with the number designations below it in Table 3. Each zone had a different number of spans, with different span distances.

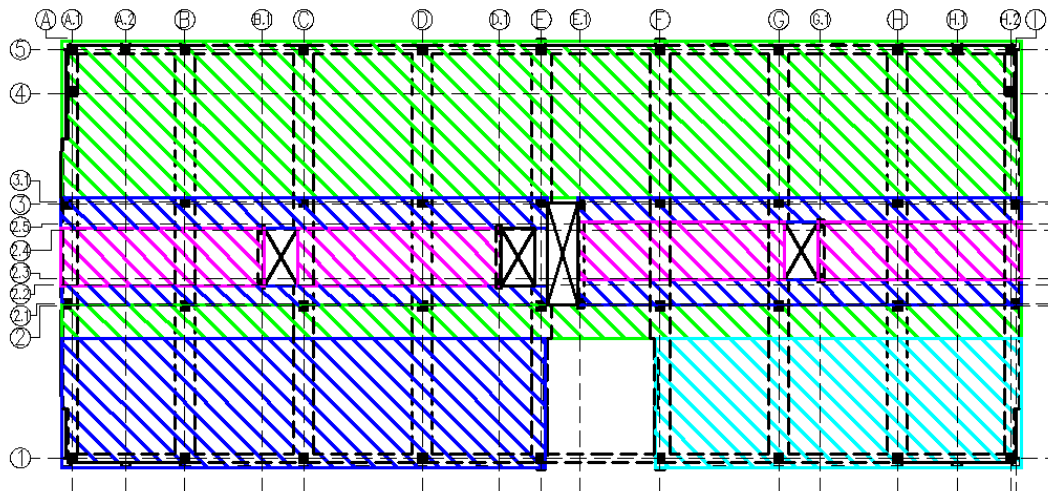


Figure 13: Slab Zones

Zone Designation	
Zone Color	Zone Number
Green	1
Magenta	2
Blue	3
Cyan	4

Table 3: Zone Designations

### Design Goals/Parameters

Design of each zone was completed using ADAPT PT v8, according to ACI 318-05/IBC 2006. A concrete strength of 5000 psi was used for the slab design and 4000 psi for the columns. Tension and compression stresses were bounded by prestress class U limits, from ACI 318-08, Chapter 18. The stress limits are summarized in Table 4, and further design parameters are summarized in Table 5.

Post-Tensioning Stress Limits: Class U	
Immediately after transfer	
Tension	Compression
$6\sqrt{f'_{ci}}$	$.60 f'_{ci}$
At service loads	
Tension	Compression
$6\sqrt{f'_c}$	$.45 f'_c$

Table 4: Class U Stress Limits

Post-Tensioned Slab Design Parameters	
Tendon Height @ Slab Edge	4.0" *
Tendon Height @ Mid Span	1.0" *
Tendon Height @ Supports	7.0" *
Size of Conventional Reinforcement	#4 Bars
Balanced Dead Load	50%-100% <sup>£</sup>
Precompression Stress	≤ 300psi <sup>¥</sup>
Location of Tendon Inflection Point	0.1 x L <sup>Ⓜ</sup>
* Measured from the bottom of the slab	
<sup>£</sup> To avoid extreme upward camber	
<sup>¥</sup> To avoid cracking due to creep	
<sup>Ⓜ</sup> To model a more accurate tendon profile	

Table 5: PT Slab Design Parameters

A 1'-0" wide strip was modeled for each zone, to determine the required amount of post-tensioning force, number of tendons and the amount of conventional reinforcement. Live load reduction was performed for loads less than or equal to 100 psf following ASCE 7-05 §4.8.1-§4.8.5. A skip live load factor of 1 was used to determine the maximum moments due to differential loading conditions. Design and deflection summaries for each zone can be found in the appendix.

### Design of Zone 1

Zone 1 consists of 8 spans. The interior bays have a span length of 30'-0", and the exterior bays have a span length of 28'-6". Because larger moments exist in the exterior spans, more post-tensioning force was required. The force was specified at  $F_e = 28.8$  kips/ft, as to not exceed the maximum precompression stress of 300psi. Two tendons per foot, capable of supplying 26.6kips each, were required.

In the interior bays, the moments were lower. As a result, only one tendon per foot was necessary to provide a force of  $F_e = 20.0$  kips/ft. Having either more or less tendons across several spans is common in practice. To achieve this, one tendon from the outer bay is anchored at roughly  $\frac{1}{4}L$  away from the column support in the adjacent interior bay. The anchor is placed at this distance in order to decrease reinforcement congestion at the columns. Figure 14 displays the aforementioned tendon layout. Jacking from both sides is required.

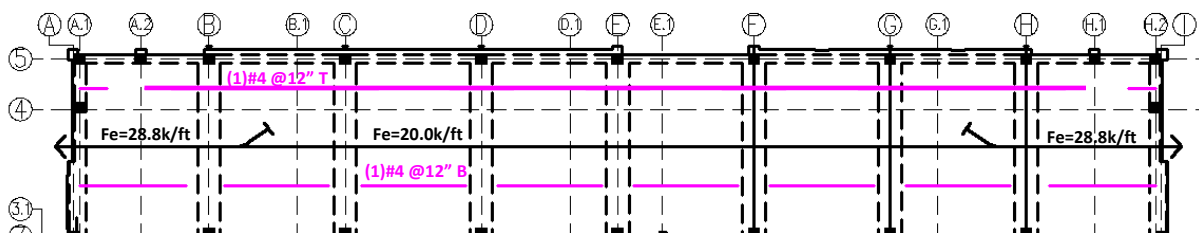


Figure 14: Zone 1 Post-Tensioning Diagram

In each span, a continuous (1) #4 conventional reinforcing bar was needed at the top of the slab due to negative moment at the column faces. Additionally, (1) #4 reinforcing bar was required at the bottom of the slab to provide flexural strength. The specified amounts of rebar are shown above in Figure 14, highlighted in magenta.

Deflections were not a critical issue in the design. The maximum service deflection was equal to 0.58" at the first interior supports. No upward camber existed in the spans.

For zone 1, a hand calculation was performed to verify the amount of conventional reinforcement determined by ADAPT. The unfactored dead load, live load, and secondary moments from the program were used with the strength design load combinations from ASCE 7-05. Load combination 2 ( $1.2D+1.6L+1.0M_{sec}$ ) controlled, and a maximum moment of 23.8 ft-kips occurred at the first interior support. Following the PCA post-tensioned design guide and ACI 318-08 chapter 18, (1) #4 in addition to the PT tendon, provided a factored flexural strength of 33.1 ft-kips. Results of the hand calculations confirmed the ADAPT output. Detailed hand calculations can be found in the appendix.

### Design of Zone 2

In this zone, there are four individual areas with two spans each. Originally, it was thought that a beam would not be required around the perimeter of the slab openings. But, the slab supports a CMU wall around the shaft openings, creating an additional load of 640 lb/ft: ( $48\text{lb/ft}^2 \times 13.33\text{ft}$ ). A quick calculation determined that the slab itself was not sufficient. A normal reinforced beam was introduced to solve this problem.

The critical beam location lies next to the elevator opening. This was because the longest span existed here. Figure 15 shows an enlarged plan of the elevator opening. The normal reinforced beam, of topic, can be seen to the right of the larger opening, labeled as Beam 1.

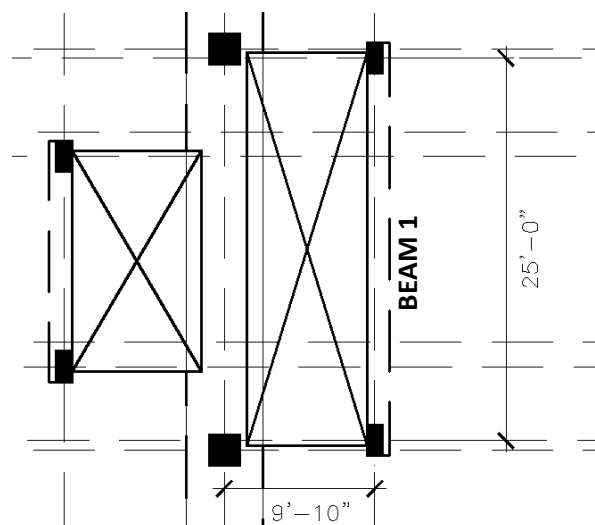


Figure 15: Central Building Plan Enlargement

Design of the beam was performed to determine the minimum beam depth to limit deflections, the required amount of flexural reinforcement, and the amount of shear reinforcement. According to Table 9.4(a) from the ACI code, the minimum depth for a simply supported, one-way beam is limited to  $L/16$ . With a span length of 25'-0" the minimum beam depth was 18.75". As a result, an 18"x20" was selected as the trial size. Since the entire slab would be poured monolithically, a concrete strength of 5000 psi was used. To support the CMU shaft wall and the slab, (5) #9's were required at the bottom of the cross section. Two legs of #3 bars @8" were also required to increase the shear strength of the beam.

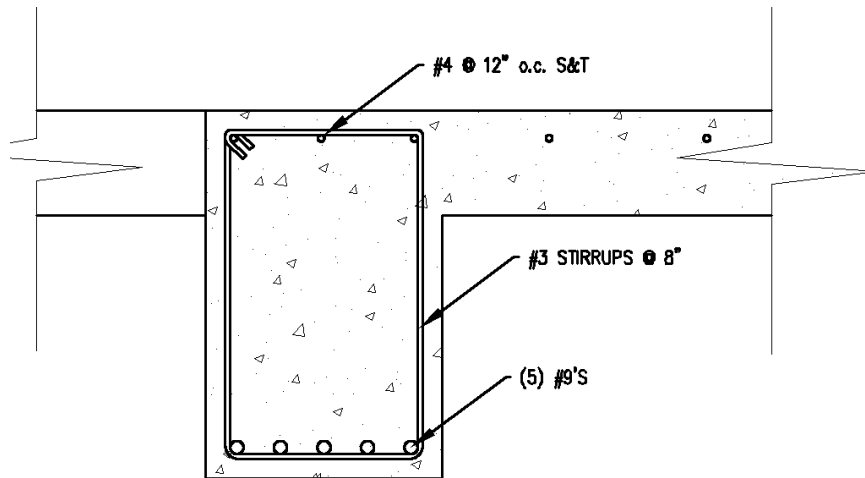


Figure 16: Cross Section of Normal Reinforced Beam

The small portion of the slab, between the girder and the normal reinforced beam, also supports the CMU wall. This area was not treated as a beam, but rather designed as a slab. Because the width of the concrete block has a nominal width of 8", an 8" wide slab strip was designed. ACI limited the slab thickness to 6" for deflection control, but to keep the slab thickness

uniform, the depth was set to 8". Calculations showed that by adding (1) #8 to the slab, no beam was required. The hand calculations involved in determining member sizes and reinforcing amounts can be found in the appendix.

With the normal reinforced beam designed, the post-tensioned design was completed. A challenge with designing the post-tensioning was the increased live load in the second span due to the mechanical room. Also, both spans had one continuous end, resulting in much higher midspan moments. Figure 17 shows the loading diagram for the two spans.

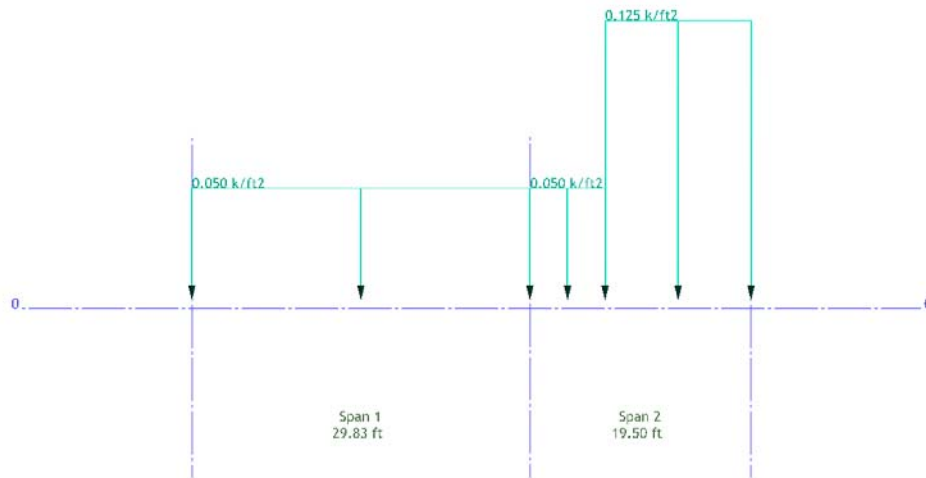


Figure 17: Zone 2 Live Loading Diagram

High tension stresses existed at transfer at the top of the slab, in the middle of span 2. This was because of the high post-tensioned force required to balance the high live load. In Figure 18, the top fiber stress is shown by the green “spike” in the graph.

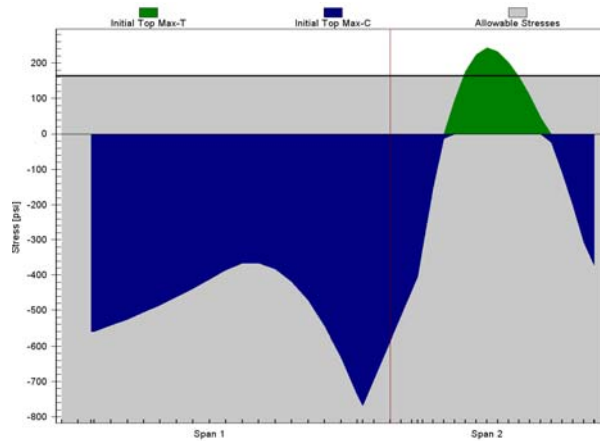


Figure 18: Top Fiber Stresses at Transfer w/ 1” Tendon Height

To reduce this stress, the tendon height in span 2 was increased from 1” to 2.5”. Doing so decreased the upward force created by the tendon, and brought the stress under the acceptable limit. The tendon height was reduced at the interior support to limit the deflection directly after the column.

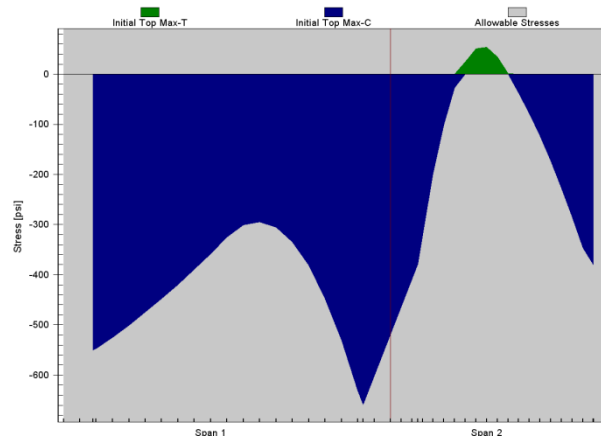


Figure 19: Top Fiber Stresses at Transfer w/ 2" Tendon Height

A PT force of 28.8kips/ft was required in both spans. Two tendons per foot were run continuously across both supports, with them being stressed from the outside of the slab and anchored at the opening. Again, (1) #4 @ 12" at the top of the slab was required over supports and (1) #4 @ 12" was required on the bottom at midspan. Figure 20 shows the tendon and conventional reinforcement layout for zone 2.

The maximum deflection was 0.32" at mid-distance in span 1. At the minimum service load, a slight upward camber existed in span 2, but it was very small.

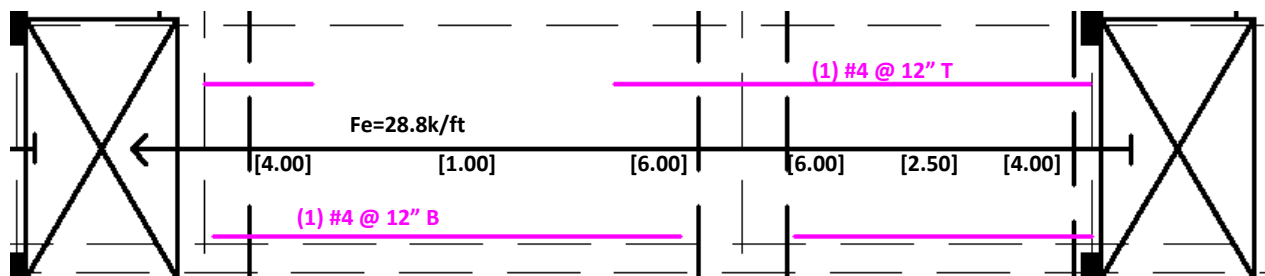


Figure 20: Post-Tensioned Zone 2

### Design of Zones 3 & 4

These zones did not present any structural challenges; the only difference between these zones and zone 1, was the number of spans. Again, the span geometry was entered into ADAPT PT, resulting in identical post-tensioning and conventional reinforcement amounts. The exterior bays required a PT force of 28.8 kips/ft and the interior bays required a force of 20.0 kips/ft. (1) #4 at both the top and bottom of the slab provided adequate flexural strength. The tendon layout for zone 3 and zone 4 are shown in

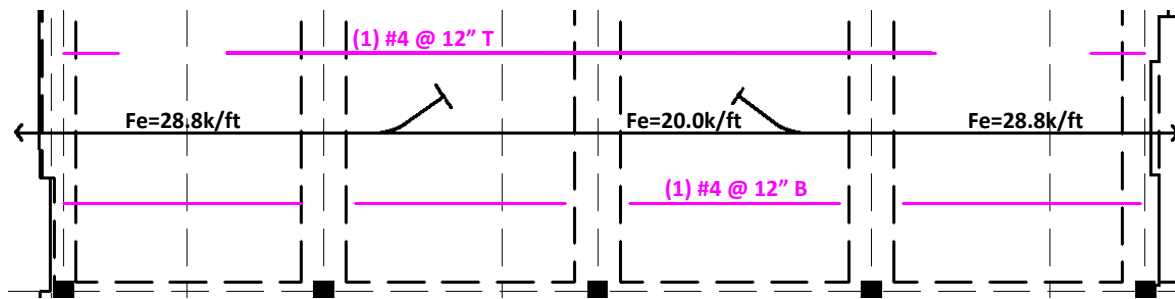


Figure 21: Post-Tensioned Zone 3

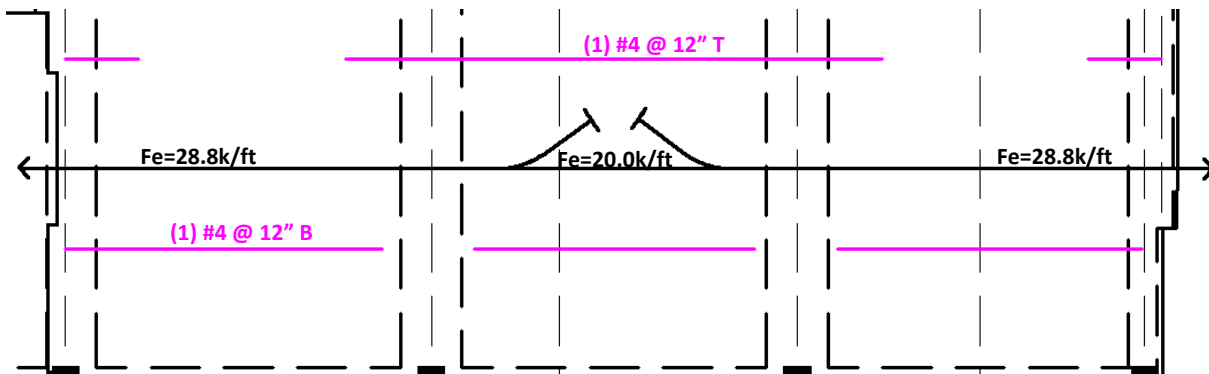


Figure 22: Post-Tensioned Zone 4

### Design of Post-Tensioned Girders

Support of the one-way slab is provided by 9 wide shallow post-tensioned girders, spanning in the short direction of the building. Each girder supports quite a large amount of load; as a result, much higher PT forces were required. Higher forces are achieved by banding several tendons together and encasing them in a plastic or metal sleeve. To get an estimate as to how many tendons could be grouped in a particular diameter, several ½" diameter circles were drawn together. It was determined that 7 tendons could be grouped in a 2 ¾" diameter conduit.

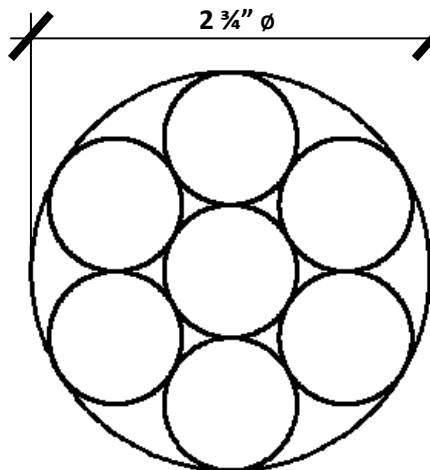


Table 6: Grouped Tendons

To conform with §7.7.2, the minimum clear cover depth for prestressed cast-in-place concrete beams, not exposed to earth or weather is 1 ½". The tendon height, from the face of the slab to the center of gravity of the tendon was set equal to:  $h_{tendon} =$

$$1 \frac{1}{2}'' + \frac{2 \frac{3}{4}''}{2} = 2 \frac{7}{8}'' \rightarrow 3.0''.$$

High precompression stresses are associated with high post-tensioning forces, therefore the effective flange width was considered in accordance with §8.12.3 and §8.12.4, to help decrease these. By including the effective flange area, all stresses were kept below 300 psi.



## Design Theory

Naturally, monolithically cast concrete provides a moment connection. Forces are therefore transferred from span to span, through the column supports. It is easy to visualize how the moments are transferred, by investigating the deflection of the frame due to different loading conditions; greater deflections are associated with higher moments. This theory was used to determine how to best adjust the tendon profile to achieve the most efficient design.

Because the length of the exterior span was much greater than the interior span, much more dead load had to be balanced. As a result, the higher PT force also existed in the interior span, and if the eccentricity of the tendon was maximized, < 100% of the dead load was balanced. By decreasing the eccentricity of the tendon profile in the middle bay, the upward force was reduced and the tension stress in the bottom fiber of the exterior span was reduced. This concept is illustrated in Figure 23.

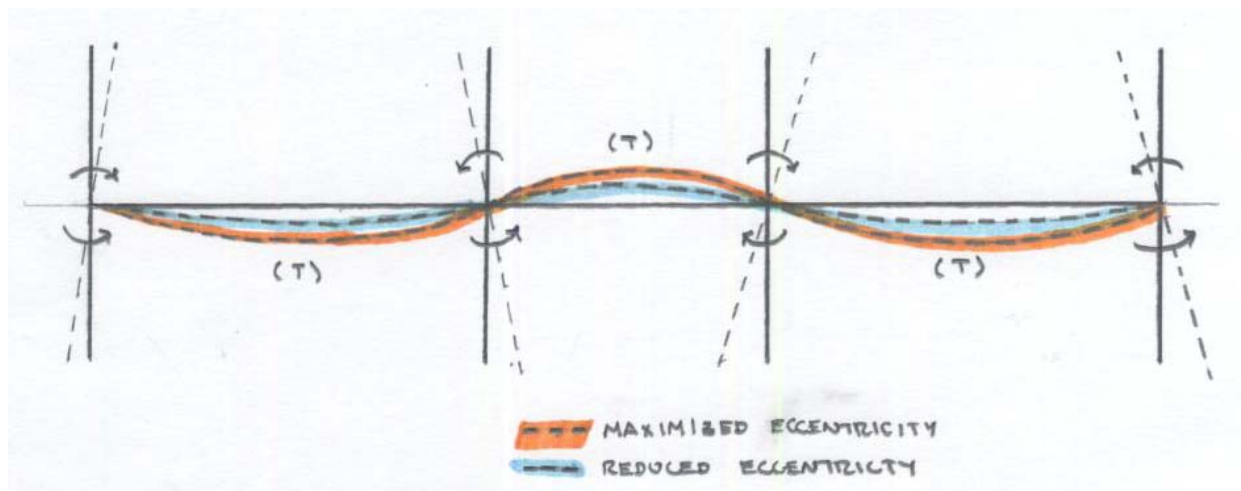


Figure 23: Frame Moment Transfer

To lower tensile stresses in the bottom of the slab at the columns, the most efficient method was to decrease the tendon height at the supports. This is because as the tendon passes over the support, it produces a downward force because of reverse curvature. The downward force, coupled with the upward force at midspan, creates bottom fiber tension at the supports. By lowering the support tendon height the PT force at both midspan and supports is decreased. Figure 24 shows the tendon force and the resulting frame deflection. Further design refinement was completed by adjusting the height of the tendon at either midspan or at the supports.

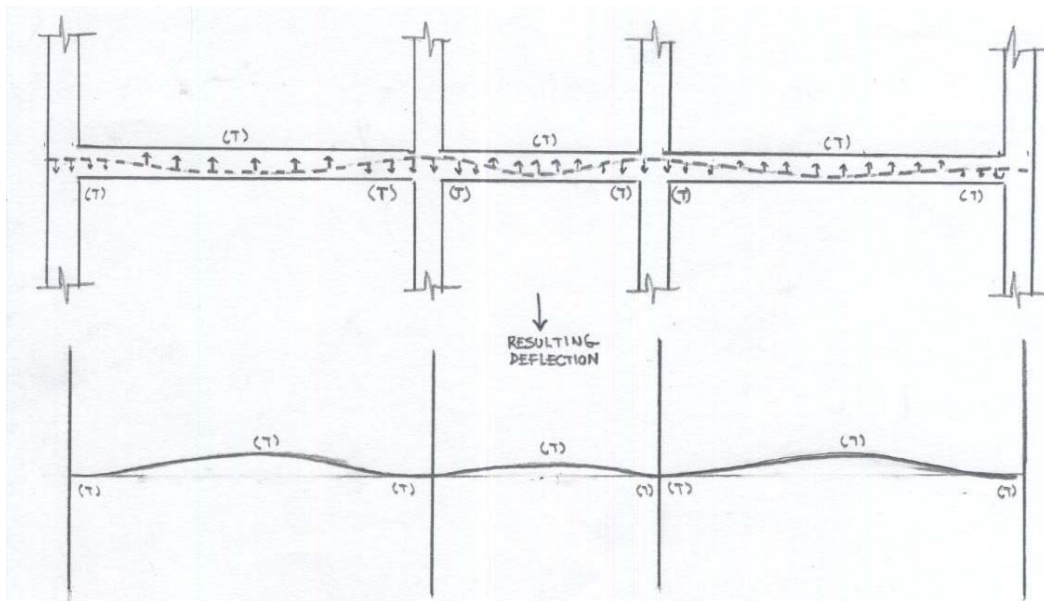


Figure 24: Frame Moment Transfer (2)

### Girder Designation

For design simplification, the girders were arranged into five typical groups, shown in Figure 25. In the plan, members having identical colors have the same design. Only girders 2 and 4 were specifically designed to determine the required amount of post-tensioning force. The assumption that half the girder size could support half the tributary area was confirmed by designing each span of girder 4. The second girder was designed because it was the most prominent member size throughout the building.

A girder on the fifth floor was also designed, to see if the member was capable of transferring the column loads from the roof. Figure 26 shows the fifth floor girder plan and Figure 27 shows the elevation of the girder.

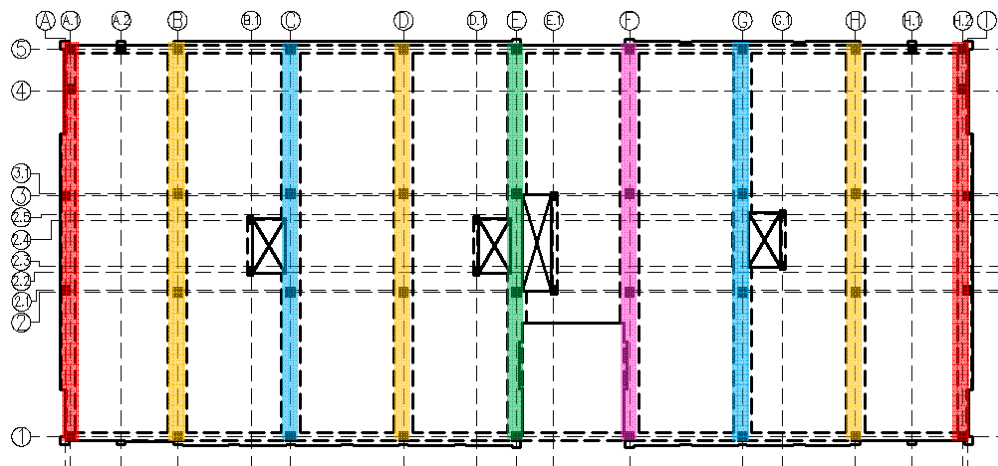


Figure 25: Typical Girders

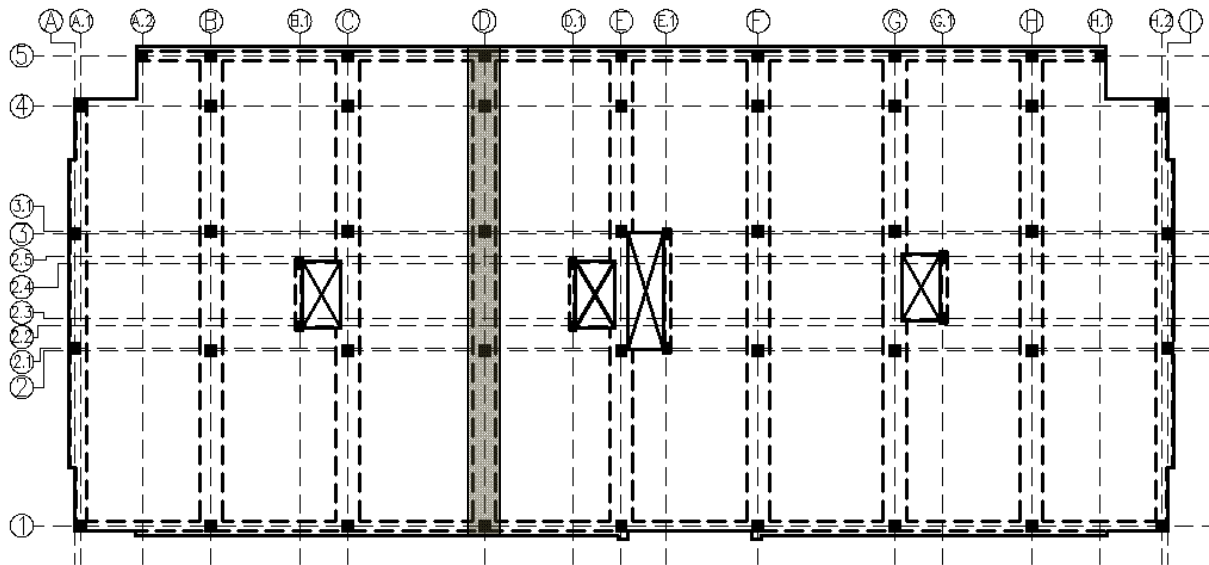


Figure 26: Fifth Floor Girder Plan

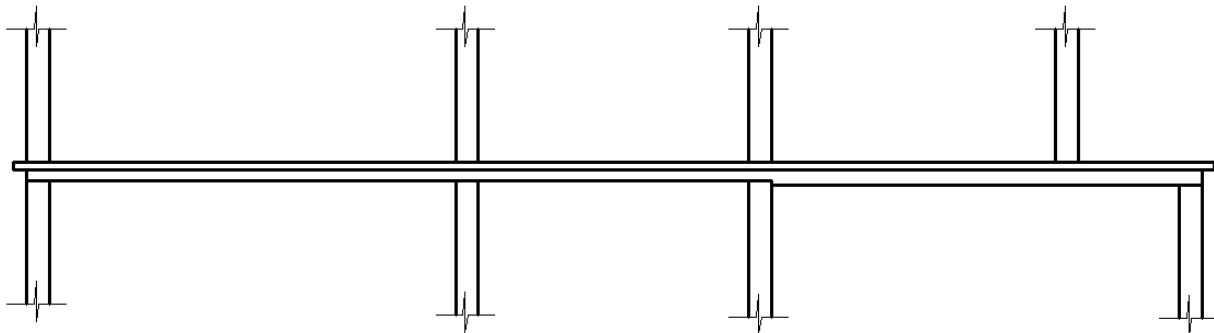


Figure 27: Fifth Floor Girder Elevation

	<b>Girder 1</b>
	<b>Girder 2</b>
	<b>Girder 3</b>
	<b>Girder 4</b>
	<b>Girder 5</b>

**Design Goals/Parameters**

Except for the tendon heights and conventional reinforcing size, all design parameters for the slab design were used. Precompression stresses were kept roughly around 300 psi, 50-100% of the dead load was balanced, and class U stress limits were implemented. Instead of using #4 rebar, #8's were used because of the anticipated increase in moment.

Table 7: Girder Designation

ADAPT PT was used for the post-tensioned design and live load reductions were performed following ASCE 7-05. A skip live load factor of 1 was applied for the girder design. Conventional reinforcement quantities were not tabulated, since the moments due to lateral loads were not known.

## Design of Girder 2

A total PT force of 600 kips was required to successfully achieve allowable stresses. The total number of tendons required was equal to 23. Therefore, at least 4 individual tendon bands would be required. Service maximum and minimum deflections were not excessively high. Design summary can be found in the appendix.

## Design of Girder 4

At midspan, girder 4 is bordered by a stair opening on the left and the elevator core opening on the right. Therefore, no effective flange width could be considered in the stress calculations. The required PT force in the outer bays created a very high precompression stress in the middle bay. To solve this problem, the amount of tendons varied over the beam length. In the first span a PT force of 500 kips was required, in the second span a PT force of 216 kips was required, and the third span required a PT force of 600 kips. A PT force of 216 kips in the middle span, created a precompression stress of 300 psi.

Less PT force was required in the first span, because it bordered the atrium opening. As assumed, half the girder size was capable of supporting half the tributary width.

Unneeded tendons from the outer bays were ran over the supports, and out through the side of the slab at the openings. Doing so helped reduce reinforcement congestion. A summary of the design and deflections can be found in the appendix.

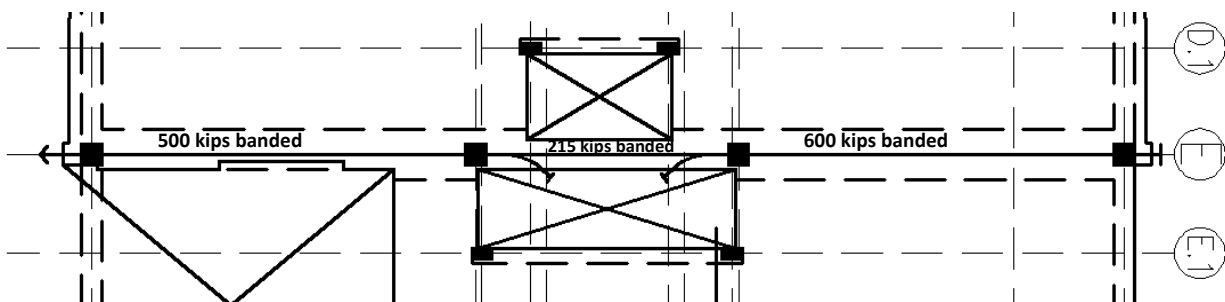


Figure 28: Girder 4 Tendon Layout

## Design of 5<sup>th</sup> Floor Girder

The amount of axial load on the girder from the column was determined by calculating the amount dead and snow load from the roof. Snow load was considered in lieu of the roof live load, because the column is located at the perimeter of the roof, where snow drifts exist.

It was impossible to have a 20" deep beam to support column, without exceeding the allowable stresses. The depth was therefore increased to 24". The tendon profile in the third span was semi-harped at the location of the column, to effectively balance the load.

Because of the higher load in the third span, a higher amount of PT force was required. 600 kips were provided in the first two spans and 1152 kips  $\rightarrow P/A = 300$  psi was provided in the third span. See the appendix for the tendon profile.

## Check of Gravity Column

Support of the normal reinforced beam next to the slab openings is provided by a 12"x24" concrete column. The member is not a part of the lateral system and only supports gravity loads. Due to architectural placement, the beam is not concentrically located on the column. An eccentricity of 3" exists. It was deemed necessary to investigate P- $\Delta$  effects for the column design.

Assuming a fixed-fixed column, the k-value was equal to 0.7. At the ground floor the unsupported length was 15'-0". From ACI 318-08, §10.10.1.2 the radius of gyration was permitted to be equal to 0.3 times the overall dimension in the direction stability is being considered  $\rightarrow 0.3 * 12 = 3.6$ ". Using the equation  $\frac{k * l_u}{r} \leq 22$  (10-6), the value was equal to 35. Hence, second-order effects had to be considered.

A take down of the dead and gravity loads from the roof to the first floor provided very small axial loads (PL = 27.0 kips, PD = 47.3 kips), because the tributary area of the column is not large. Moment due to the eccentricity was equal to 12.5 ft-kips at mid-height. A deflection calculation was performed, providing minimal displacement. As a result, this small deflection did not cause an increase in the moment at midspan.

PCA column was used to determine the required amount of reinforcing. The concrete strength was set to 5000 psi, and a trial area of steel of 6.32 in<sup>2</sup> was used ( $\rho = 2.19\%$ ). The ultimate axial load and moment were easily carried by the column. Hand calculations and the PCA printout are in the appendix.

## Design of Lateral System

### Load Distribution

To resist seismic and wind loads on the building, the nine post-tensioned girders two post-tensioned edge beams, along with the building columns, were used to limit deflections and provide lateral strength. In the short direction of the building, that lateral load on each floor is transferred through the diaphragm to all nine of the concrete moment frames. Each frame extends from the ground floor to the roof, which allows the moments to be transferred from the beams, and to the columns. In the long direction of the building, the rear frame does not extend entirely to the roof. Instead, it only extends from the ground to the fifth floor, due to the exterior terrace setback. This does not result in a direct transfer of moments through the building. Lateral loads applied to the roof in the long direction are transferred to the outer edge beams, but instead of being transferred to other columns, the frame causes torsion in the supporting girders. The torsion is then transferred to the supporting columns, and additional load from each floor diaphragm is transferred through the rest of the moment frames. Distribution of moment is shown in Figure 29. Because of the load transfer, the torsional capacity of the girders was later checked.

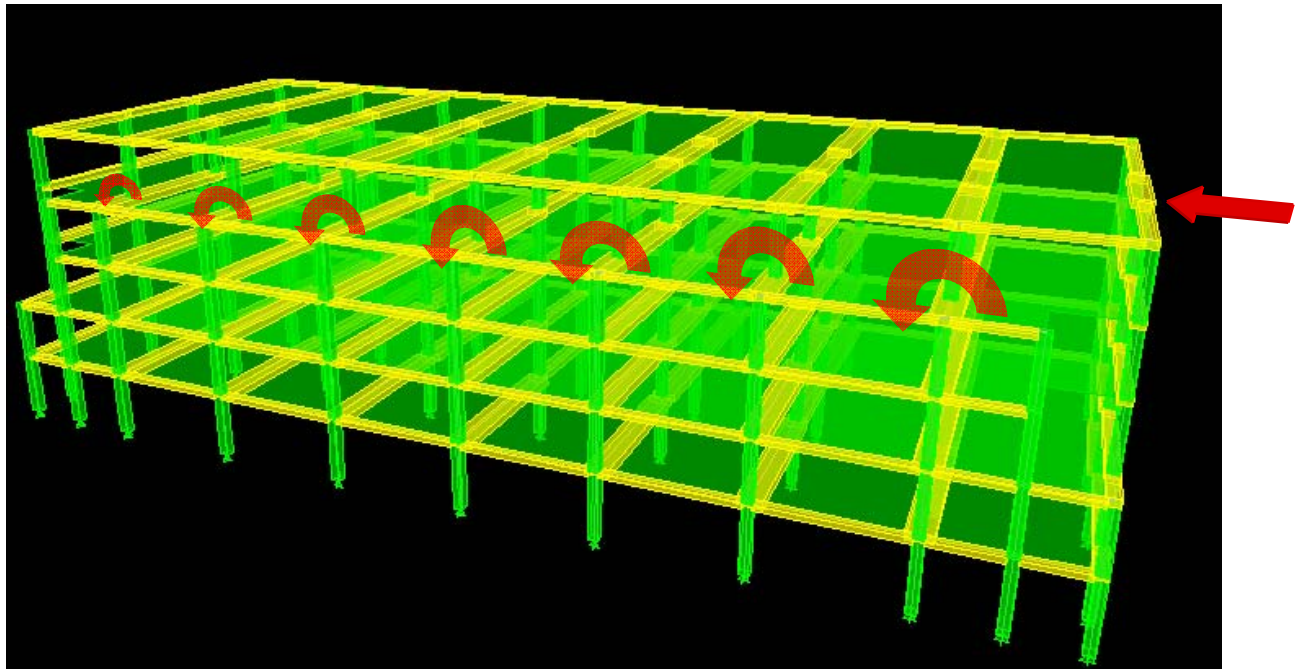


Figure 29: Moment Transfer Rear Moment Frame

### ETABS Model

The computer program ETABS was used to model only the members in the lateral system. Following §8.8.1-8.8.2, the gross moment of inertias for the columns and beams were reduced to mimic cracking. Columns were reduced by a factor of 0.70 and the beams were reduced by a factor of 0.40. It is permissible to increase the reduction factor of flexural members from 0.35, but not greater than 0.70. Post-tensioning helps resist cracking, which justified the 0.05 increase. Figure 30 and Figure 31 display the front and the back of the model, respectively.

Wind load cases 1 -4 from ASCE 7-05 were analyzed to determine which one controlled the building deflections. Case 1 created the largest deflections, which is verified by conceptual reasoning. With evenly distributed moment frames, and moment frames at the exterior of the building, torsion was unlikely to control. Seismic loads determined previously were applied to the center of mass to determine seismic story drift.

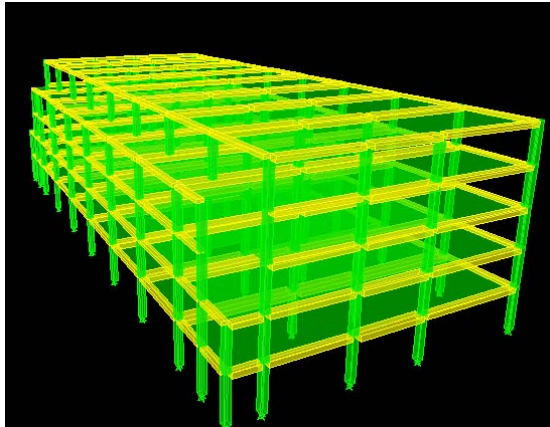


Figure 31: Back of ETABS Model

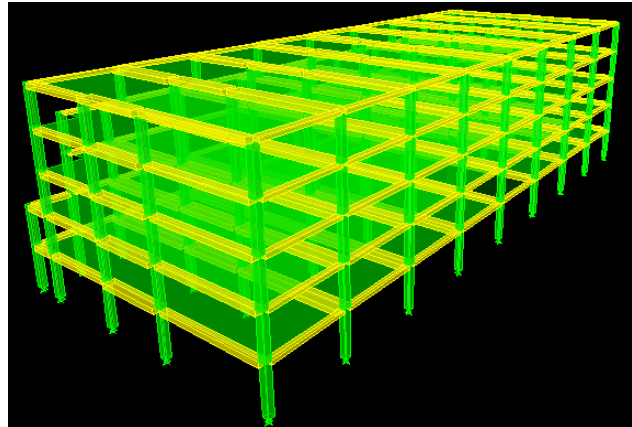


Figure 30: Front of ETABS Model

### Story Drift/Displacement

Story displacement and drift, created by wind load case 1 was limited to  $L/400$ . Seismic story drifts were limited to  $\Delta = 0.020 \cdot h_{sx}$ . Both wind and seismic displacement/drifts were all under the acceptable limits. Table 8 summarizes the wind load displacements and story drifts, while Table 9 displays the seismic story drifts.

Lateral Displacement Due to Wind					
Lateral Displacement in Y Direction					
Floor	Height (ft)	Displacement (in)	Allowable Displacement (in)*	Story Drift (in)	Allowable Story Drift (in)*
RF	68.32	0.921	2.05	0.05	0.40
5	54.99	0.868	1.65	0.10	0.40
4	41.66	0.772	1.25	0.14	0.40
3	28.33	0.637	0.85	0.19	0.40
2	15	0.444	0.45	0.44	0.45
Lateral Displacement in X Direction					
Floor	Height (ft)	Displacement (in)	Allowable Displacement (in)*	Story Drift (in)	Allowable Story Drift (in)*
RF	68.32	1.40	2.05	0.18	0.40
5	54.99	1.22	1.65	0.21	0.40
4	41.66	1.01	1.25	0.26	0.40
3	28.33	0.751	0.85	0.31	0.40
2	15	0.442	0.45	0.44	0.45

\* Limited to  $L/400$

Table 8: Wind Load Displacements/Story Drifts

Seismic Story Drift									
Story Drift - X Direction					Story Drift - Y Direction				
Floor	Story Height	Displacement	Story Drift	Allowable Drift*	Floor	Story Height	Displacement	Story Drift	Allowable Drift*
RF	13.33	3.04	0.35	3.2	RF	13.33	0.589	0.026	3.2
5	13.33	2.69	0.44	3.2	5	13.33	0.563	0.055	3.2
4	13.33	2.25	0.55	3.2	4	13.33	0.508	0.083	3.2
3	13.33	1.70	0.7	3.2	3	13.33	0.425	0.126	3.2
2	15	1.00	1.00	3.6	2	15	0.299	0.299	3.6

\* Limited to  $0.020h_{sx}$

Table 9: Seismic Story Drifts

## Torsion Check

As mentioned earlier, the transfer of lateral load in the long direction of the building induces torsion into the supporting girder. To determine if the member was capable resisting the load, the threshold torsion was compared to the ultimate torsion. Assuming #5 stirrups and a 1 ½" clear cover, the threshold torsion was calculated to be equal to 54.5 ft-kips. Wind load in the X direction created the highest torsion in the girder with a  $T_u = 25.3$  ft-kips. For the member the threshold torsion was greater than the ultimate torsion and therefore no extra design was required. Hand calculations can be found in the appendix.

## Lateral Column Design

After determining the increased axial load and column moments due to the lateral loads, an interior column at the first floor was designed to check the required amount of reinforcement. As a rule of thumb, the amount of steel was limited to 2%.

The column being investigated was located on the lower level, where the highest story drifts occurred due to wind load. P-Δ effects from the lateral deflection were considered to magnify the column moment. Load combination 4, 1.2D + 1.6W + L, was used because of the high wind moments. The ultimate axial load on the column was calculated to be equal to 1651 kips. The highest moment due to wind existed at the top and was equal to 205 ft-kips.

Second order moments were considered at the column mid-height. A maximum deflection of 0.47" occurs due to the wind load. A 776 ft-kips moment is created due to the combination of the axial load and displacement at the top of the column. This additional moment was combined with the wind load moment at midspan, to increase the midspan moment to 481 ft-kips. One deflection calculation and iteration further increased the moment to 523 ft kips.

With the  $P_u$ ,  $M_u$  of 1651 kips, 523 ft-kips PCA Column was used to determine the amount of reinforcement. An initial design was executed using (8) #11's, but the column was not adequate. The steel reinforcement was then increased to (12) #11's and was sufficient. Such a high amount of steel exceeded the desired 2% reinforcement. Consequently, mechanical splices would need to be used as to not exceed that max reinforcement of 4%. Detailed calculations can be found in the appendix.



## Architectural Breadth

It was important to determine if the movement of the vertical shafts would have a negative impact on the architectural layout. Three areas of the architectural plan were investigated.

1. Area 1: Stairwell 1
2. Area 2: Stairwell 2/Elevator Core
3. Area 3: Stairwell 3

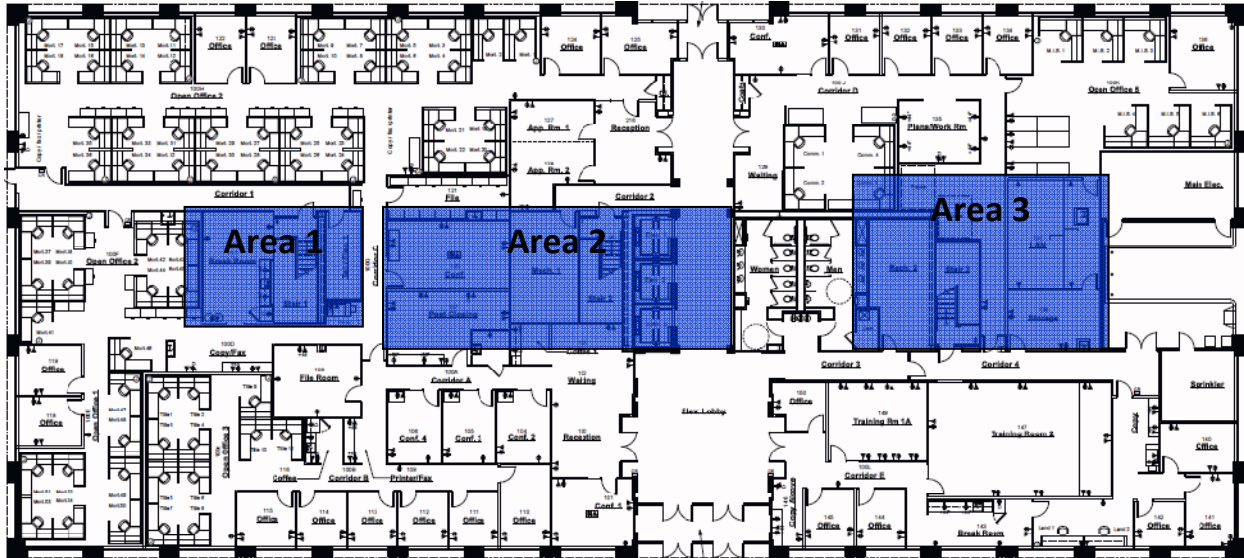


Figure 32: Architectural Investigations

### Area 1

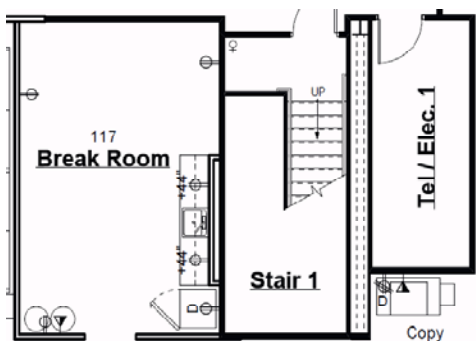


Figure 34: Area 1 Before

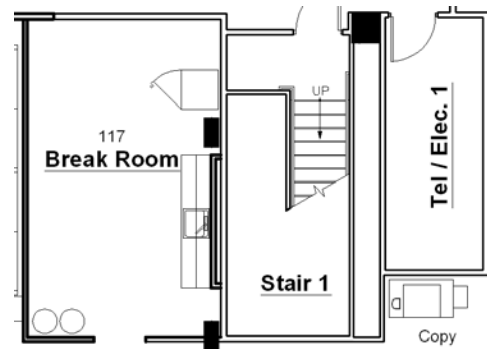


Figure 33: Area 1 After

To accommodate the wide girders, it was necessary to move stair 1 to the left by 7 1/4". Doing so decreased the width of the break room, but ample room is still available. Also, some of the breakroom cabinetry had to be moved to accommodate the smaller gravity columns.

## Area 2

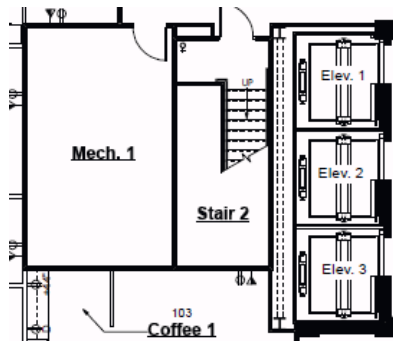


Figure 35: Area 2 Before

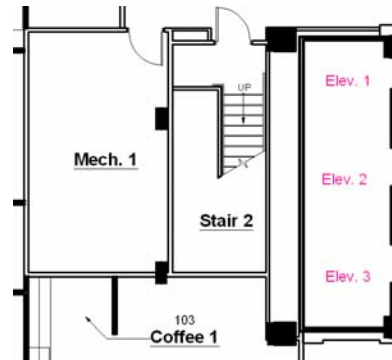


Figure 36: Area 2 After

In this area, stair 2 and the elevator core had to be moved left and right, respectively. The elevator core was only moved 3" to the right, which narrowed the circulation area at the elevators. However, because it was only decreased by a few inches, there were no negative impacts. Stair 2 was moved 7 ¼" to the right, decreasing the size of the mechanical room. After reviewing the mechanical room 1 plan, the sacrifice was deemed as not detrimental.

## Area 3

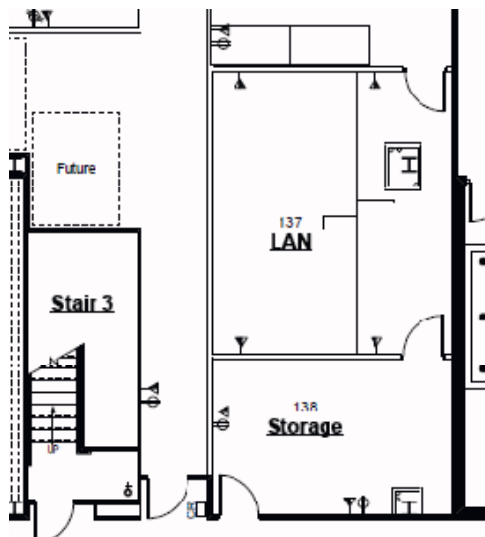


Figure 38: Area 3 Before

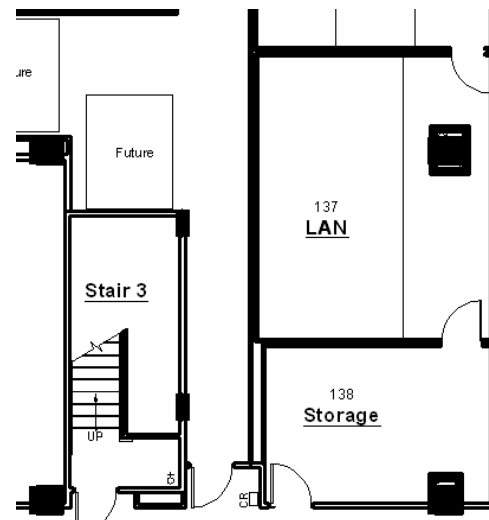


Figure 37: Area 3 After

Adjacent to stair 3, was a circulation corridor. As to not decrease the width of the hallway, the entire corridor was shifted to the left 7 ¼". Spaces effected by this shift included a computer server and a storage room; both which are not critical spaces.

None of the required architectural changes had a negative impact on the office layout. Functional alternatives were created for each area. Spaces that were impacted were not critical, and could be manipulated to accommodate the new structural system.

## Cost and Schedule Breadth

To determine which structural system would be the most cost efficient and quickest to be complete, a cost and schedule comparison was completed between the steel framing and post-tensioned concrete system. For the scope of this analysis, only the superstructure was investigated. Detailed take-offs for the slab on grade and foundations were not conducted.

## Cost Comparison

Cost comparison was completed using values from the most recent RS Means Cost Works Data. No overhead and profit was included into the estimates; only bare costs were used. Because each floor was very similar, the total cost of one floor was used to determine a rough square foot dollar value. An estimated \$5.00 per square foot was added onto the steel estimate to account for foundations. Larger foundations were required for the concrete structure, so \$10.00 was added to the square foot estimate.

Cost Comparison w/o Foundations		
System	Total Cost	Cost/SF
Steel	\$ 367,000.00	\$ 14.45
Concrete	\$ 501,000.00	\$ 19.72
Cost Comparison w/ Foundations		
Steel	\$ 367,000.00	\$ 19.45
Concrete	\$ 501,000.00	\$ 29.72

Table 10: Cost Comparison

The bare steel structure was roughly \$5.00 per square foot cheaper than the post-tensioned concrete structure. A primary reason for using post-tensioned concrete is to maximize the stories per height in a building. However, due to story limitations in the zoning area, this is an option. If the building were located elsewhere, an additional story could be added, increased profit from the additional floor would have to be calculated to see if PT would be a probable structural system.

## Schedule Comparison

Durations for each activity were calculated using RS Means Cost Works Data. Some guidelines followed for the schedules are as follows.

### Steel

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

## Concrete

- Place roughly 150 cubic yards of concrete per day
- Stress slabs and girders after curing for 3 days
- Separate construction into 3 equal floor areas to accelerate construction

For the steel superstructure, columns were first erected, followed by beams, and finally steel decking and reinforcing. Due to OSHA requirements, no more than 2 floors of columns could be free standing without decking installed. A schedule for the erection of the framing and casting of the slabs was completed for every floor. It was determined that the steel superstructure would take 50 days to complete.

To accelerate the concrete schedule, the slab was divided into three equal regions. Once columns were formed and reinforced in the first portion, the concrete crew would place the concrete. At the same time, other crews would be working at forming and reinforcing the next section of columns. This process of forming and pouring helped to decrease the project time. However, the post-tensioned structure did take longer to complete. Construction of the first two floors was predicted to take 28 days (14 days per floor). The total construction duration for the entire building was expected to be 70 days. This is 20 days longer than the steel superstructure.

Schedule Comparison	
Steel	Concrete
50 days	70 days

Table 11: Schedule Comparison

## Conclusions

### Structural Depth

After completing the structural design with post-tensioned concrete, a thinner floor depth was achieved. The existing maximum floor depth was 27" compared to the 24" on the 5<sup>th</sup> floor. One disadvantage of the change was the significant increase in building weight. Using concrete roughly doubled the total building weight, which would require foundation redesign.

### Architectural Breadth

To accommodate the wide flange girders, it was necessary to move the vertical transportation openings. Alternative functional layouts were created, without negatively impacting the architectural layout. The effects on the architecture were therefore neutral.

### **Construction Management Breadth**

A cost and schedule comparison was completed between the original and new structural systems. As expected, the steel framing was cheaper and quicker to erect. A total of 50 days was required. By sequencing the concrete construction, the construction time took 20 days longer. The costs associated with the concrete were also higher. Being able to add an additional floor to the building within the height limitations would be the only reason to make PT viable for Building A. However, the number of floors is restricted to 5 by the zoning ordinance.

## **Appendix**

## Snow Load Calculation

<p>NICK ZIEGLER</p>	<p>SNOW LOAD CALCULATIONS</p>	<p>(1)</p>
<p>FLAT ROOF SNOW LOAD</p> <p>ROOF SLOPE = 1/4 in PER FOOT</p> $p_f = 0.2 \cdot C_e \cdot C_t \cdot p_g \cdot I$ <p>THERMAL FACTOR: <math>C_t = 1.0</math> (TABLE 7-3)</p> <p>IMPORTANCE FACTOR: I</p> <p>BUILDING CATEGORY = II <math>\rightarrow I = 1.0</math> (TABLE 7-4)</p> <p>EXPOSURE FACTOR: <math>C_e</math></p> <p>TERRAIN CATEGORY = B PARTIALLY EXPOSED</p> $C_e = 1.0$ (TABLE 7-2) <p>GROUND SNOW LOAD: <math>p_g</math></p> $p_g = 30 \text{ psf}$ (FIGURE 7-1) $p_f = 0.2(1.0)(1.0)(30)(1.0) = 21 \text{ psf}$ <p>DRIFT SNOW LOADS FOR CERTAIN JOISTS</p> <p>ZLKSP1 - DRIFT ACCUMULATES AGAINST PARAPET</p> $s = 0.13(30) + 14 = 17.9 \text{ lb/ft}^3$ <p>WINDWARD DRIFT</p> $h_d = 0.75 \left[ 0.43(D_w)^{1/3} (p_g + 10)^{1/4} - 1.5 \right]$ $= 0.75 \left[ 0.43(107)^{1/3} (30+10)^{1/4} - 1.5 \right]$ $= 2.73 \text{ ft} < h_c \rightarrow w = 4(2.73) = 10.92 \text{ ft}$ $p_d = 17.9(2.73) = 48.967 = 49 \text{ lb/ft}$		

## Wind Load Calculation

Variables	
Basic Wind Speed	$V = 90\text{MPH}$
Wind Directionality Factor	$K_d = 0.85$
Importance Factor	$I = 1.0$
Exposure Category	B
Gust Factor	$G=0.85$
External Pressure Coefficient Windward Wall	$C_p=0.8$
External Pressure Coefficient (N-S Leeward Wall)	$C_p= -0.5$
External Pressure Coefficient (E-W Leeward Wall)	$C_p= -0.28$
Internal Pressure Coefficient	$GC_p= \pm 0.18$

Table 12: Wind Variables

Velocity Pressure Coefficients: $K_z, K_h$			
Story	Height	$K_z$	$q_z$
1	15	0.57	10.13
2	28.33	0.69	12.15
3	41.67	0.77	13.56
4	55	0.83	14.68
5	68.21	0.89	15.61
Roof Screen	84.5	0.94	16.60

Table 13: Velocity Pressure Coefficients



NICK ZIEGLER

WIND LOAD CALCULATIONS

(1)

METHOD 2

BASIC WIND SPEED:  $V = 90$  MPH  
 WIND DIRECTIONALITY FACTOR:  $K_d = 0.95$   
 IMPORTANCE FACTOR:  $I = 1.0$   
 EXPOSURE CATEGORY: B  
 TOPOGRAPHIC FACTOR:  $K_{zt} = 1.0$   
 VELOCITY PRESSURE COEFFICIENTS:  $K_h$

- CASE 2
- EXPOSURE B
- $h = 94' - 6"$

• INTERPOLATE VALUE FROM TABLE 6-3

$$K_h = \frac{(94.5 - 80)(0.96 - 0.93)}{(90 - 80)} + 0.93$$

$$= 0.9435$$

VELOCITY PRESSURE COEFFICIENTS:  $K_z$

- CASE 2
- EXPOSURE B

$$K_z = 0.59 \quad z \leq 15.5 \text{ ft}$$

$$K_z = 2.01 \left( \frac{z}{z_g} \right)^{2/\alpha}$$

$$z_g = 1200$$

$$\alpha = 7.0$$

HEIGHT	$K_z$	$q_z$
0' - 15'	0.59	10.05
20'	0.62	10.93
25'	0.66	11.63
30'	0.70	12.34
40'	0.76	13.40
50'	0.81	14.28
60'	0.85	14.98
70'	0.89	15.69
80'	0.93	16.39
94.5'	0.9435	16.63

\* DIFFERENT HEIGHTS  
 IN EXCEL FILE/TABLE  
 CALCULATED BY THE  
 SAME METHOD

VELOCITY PRESSURES @  $h$ :  $z$

$$q_h = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I$$

$$= 0.00256 (0.9435)(1.0)(0.95)(90)^2(1.0) = 16.63$$

• SAME FORMULA USED FOR  $q_z$ : SEE TABLE ABOVE FOR VALUES

GUST FACTOR:

$$G = \frac{100}{H} = \frac{100}{84.5} = 1.18 > 1.0$$

$$\therefore G = 0.85$$

NICK ZIEGLER

WIND LOAD CALCULATIONS

(2)

PRESSURE COEFFICIENTS:  $C_p$

• NORTH-SOUTH DIRECTION

$$B = 244'$$
$$L = 109'$$

$$L/B = \frac{109}{244} = 0.447$$

$$\text{WINDWARD WALL: } C_p = 0.9$$
$$\text{LEEWARD WALL: } C_p = -0.5$$

• EAST-WEST DIRECTION

$$B = 109'$$
$$L = 244'$$

$$L/B = \frac{244}{109} = 2.24$$

$$\text{WINDWARD WALL: } C_p = 0.9$$
$$\text{LEEWARD WALL: } C_p = -0.29 \text{ - INTERPOLATED}$$

INTERNAL PRESSURE COEFFICIENTS:  $G C_{pi}$

$$G C_{pi} = \pm 0.18$$

DETERMINE DESIGN WIND PRESSURES

- WINDWARD WALLS:  $p_z = q_z G C_p - [q_h (G C_{pi})]$  <sup>NEGLECT FOR WINDWARD</sup> <sub>LEEWARD PRESSURE</sub>
- LEEWARD WALLS:  $p_h = q_h G C_p - [q_h (G C_{pi})]$

\* SEE TABLE FOR VALUES

## Seismic Calculation

### SEISMIC CALCULATIONS

SPECTRAL RESPONSE ACCELERATIONS:

$$S_s = 0.158 \quad (\text{USGS MAPS})$$

$$S_1 = 0.051$$

SITE CLASS OF SOIL: C

$$S_{MS} = F_a \cdot S_s \quad (\text{EQ. 11.4-1})$$

$$S_{M1} = F_v \cdot S_1 \quad (\text{EQ. 11.4-2})$$

$$F_a = 1.2 \quad (\text{TABLE 11.4-1})$$

$$F_v = 1.7 \quad (\text{TABLE 11.4-2})$$

$$S_{MS} = 1.2(0.158) = 0.1896$$

$$S_{M1} = 1.7(0.051) = 0.0867$$

$$S_{DS} = 2 \frac{S_{MS}}{3} \quad (\text{EQ. 11.4-3})$$

$$S_{D1} = 2 \frac{S_{M1}}{3} \quad (\text{EQ. 11.4-4})$$

$$S_{DS} = 2 \frac{0.1896}{3} = 0.1264$$

$$S_{D1} = 2 \frac{0.0867}{3} = 0.0578$$

OCCUPANCY CATEGORY: II

$$T_a = C_L \cdot h_n^x \quad \text{Eq. 12.8-7}$$

$$= (0.02)(92.6)^{0.75}$$

$$= 0.648$$

\* CONCENTRICALLY BRACED FRAME

$$h_n = 92.6'$$

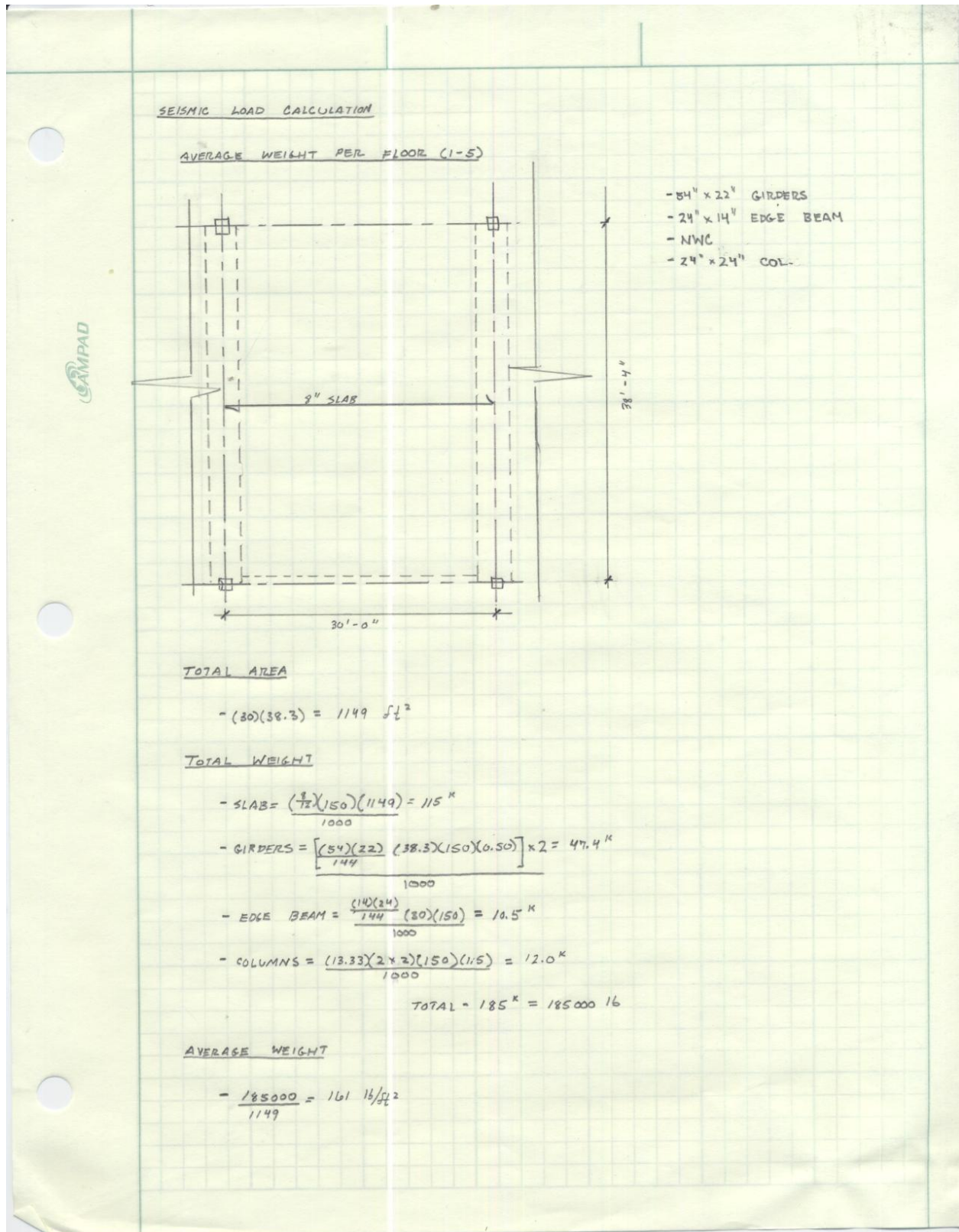
$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.0578}{0.1264} = 0.457$$

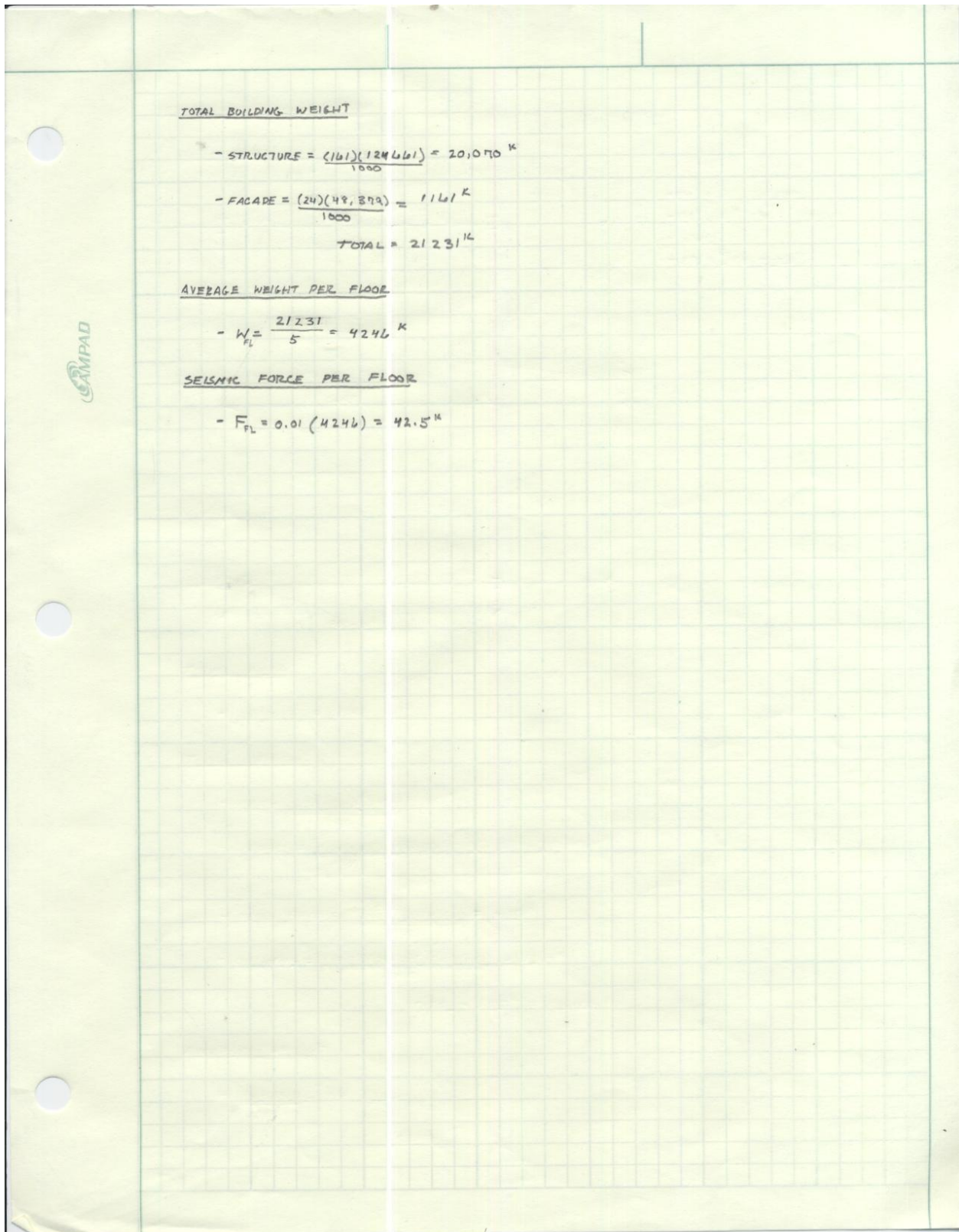
$$0.8T_s = 0.8(0.457) = 0.3656 < T_a$$

$$T_a > 0.8T_s$$

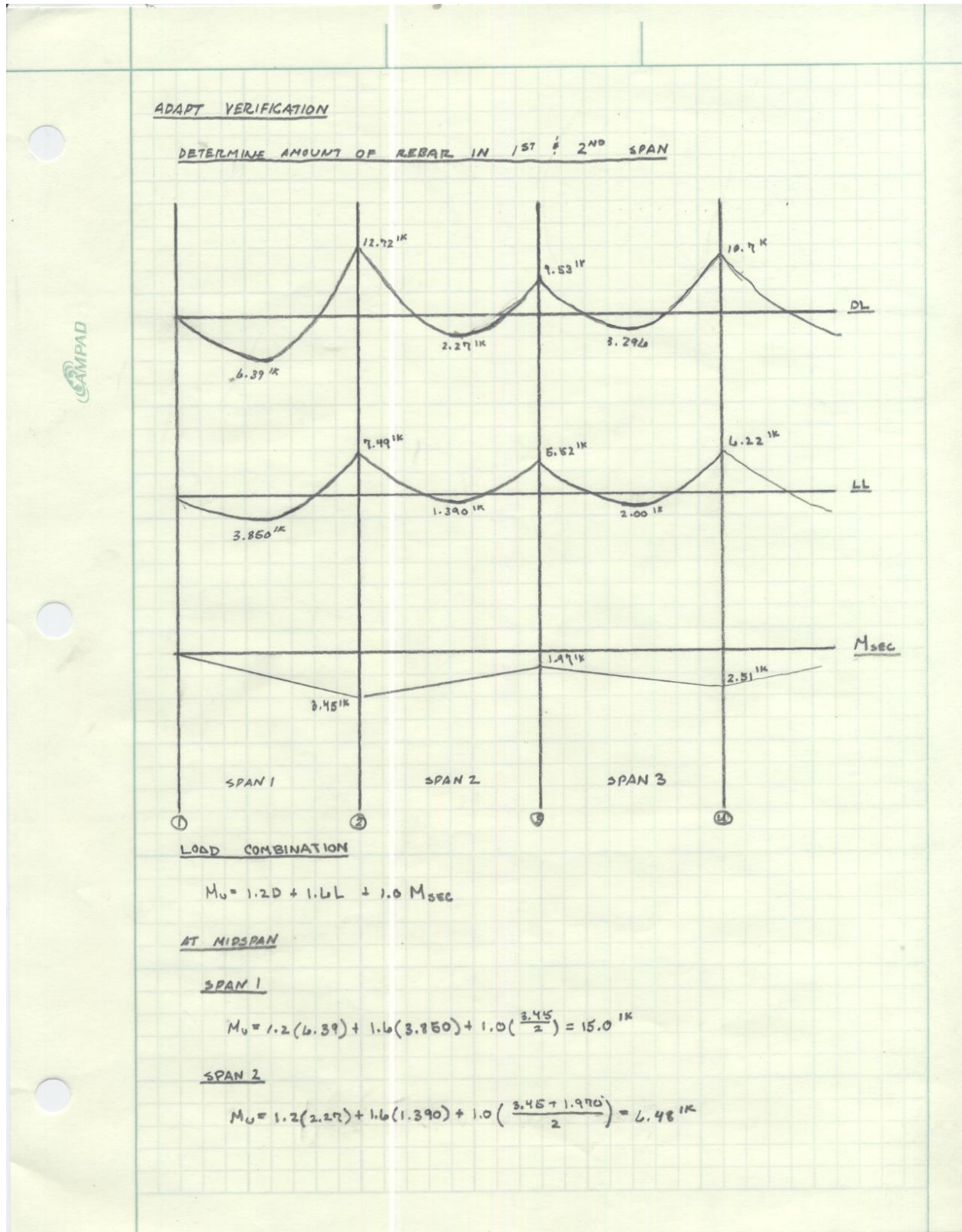
$$S_s \leq 0.15 \quad \& \quad S_1 \leq 0.04$$

SITE DESIGN CATEGORY A





## Verification of ADAPT



AT SUPPORT 2

$$M_u = 1.2(-12.72) + 1.6(-7.49) + (3.45) = -23.8 \text{ k} \leftarrow \text{LARGEST MOMENT}$$

THEREFORE CHECK REINFORCEMENT HERE

CHECK MINIMUM REINFORCEMENT FOR ULTIMATE STRENGTH

$$M_n = (A_s \cdot f_y + A_{ps} \cdot f_{ps})(d - y_z)$$

$$A_{ps} = 0.153(2) = 0.306 \text{ m}^2$$

$$d = 8 - \frac{3}{4} - \frac{1}{4} = 7 \text{ m}$$

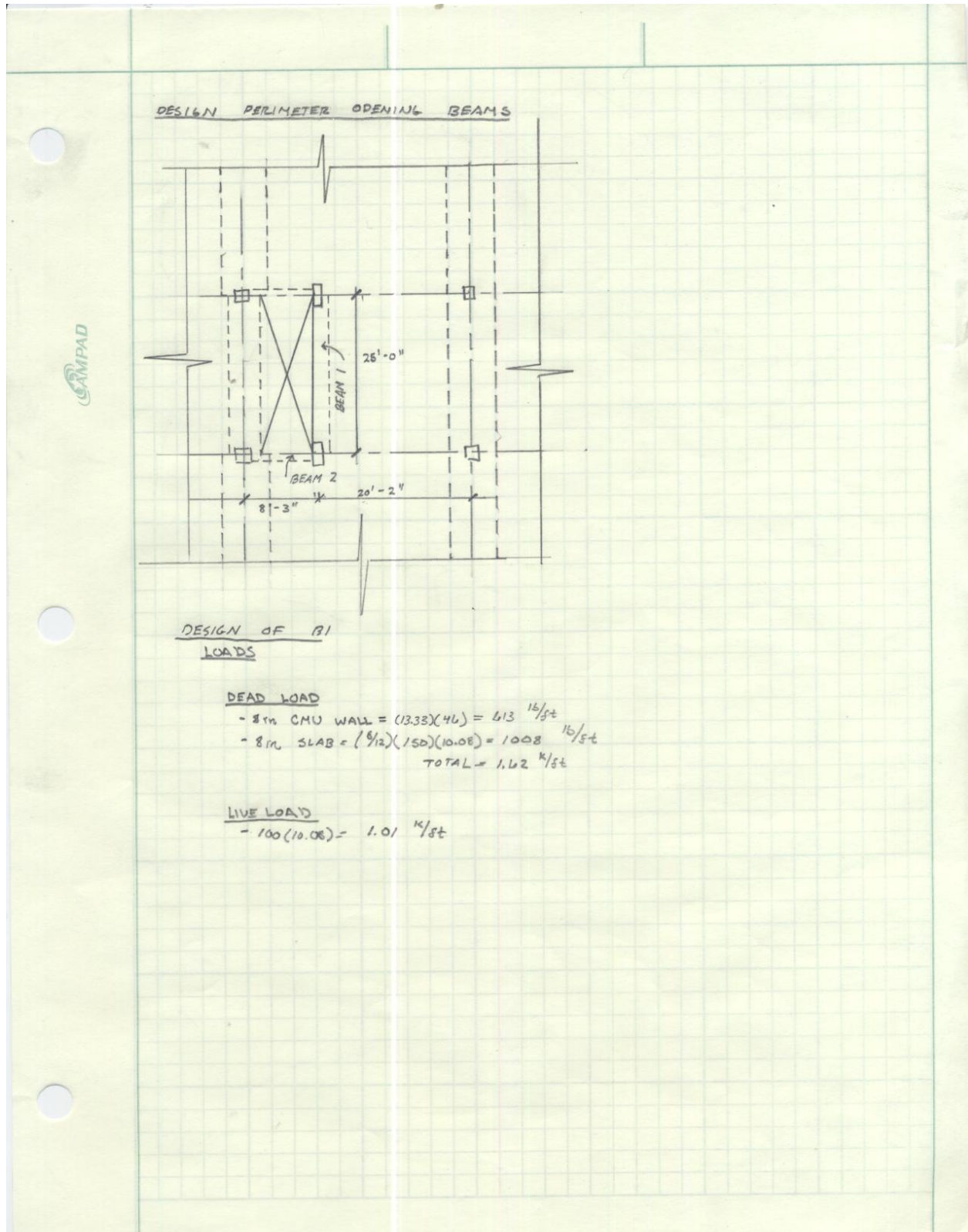
$$f_{ps} = 174,000 + 10,500 + \frac{5000(12 \times 7)}{300(0.306)} = 188,400 \text{ p si}$$

$$\text{TRY (1) \#4 } A_s = 0.20 \text{ m}^2$$

$$a = \frac{0.20(60) + 0.306(189)}{0.85(5 \times 12)} = 1.37 \text{ m}$$

$$\phi M_n = (0.9)[0.20(60) + 0.306(189)] \left[ 7 - \frac{1.37}{2} \right] = 397 \text{ m} \cdot \text{k}$$
$$= 33.1 \text{ k} > 23.8 \text{ k} \therefore \underline{\underline{OK}}$$

## Normal Reinforced Beam Design





FACTORED LOAD

$$w_u = 1.2(1.62) + 1.6(1.01) = 3.56 \text{ k/ft}$$

$$M_u = \frac{3.56(25)^2}{8} \times 1.1 = 306 \text{ k}$$

ESTIMATE SIZE

$$d^3 = 20(306)\left(\frac{5}{4}\right) = 19,175$$

$$b \cdot d^2 = 18(19.5)^2 = 6845 \text{ in}^2$$

TRIAL BEAM SIZE  $\rightarrow h = 20 \text{ in} \quad b = 18 \text{ in}$

SELFWEIGHT EFFECTS

$$w_{sw} = \frac{(20)(18)}{144}(150) = 375 \text{ lb/ft} = 0.375 \text{ k/ft}$$

$$w_u = 3.56 + 1.2(0.375) = 4.01 \text{ k}$$

$$M_u = \frac{4.01(25)^2}{8} = 313 \text{ k}$$

$$20 \times 313 = 6260 \text{ in}^2 < 6845 \checkmark$$

ESTIMATE  $A_s$

$$d = 20 - 1.5 - 0.5 - \frac{1.128}{2} = 17.4 \text{ in}$$

$$A_s = \frac{313}{4(17.4)} = 4.50 \text{ in}^2 \rightarrow \text{PROVIDE (5) \#9} \rightarrow A_s = 5.00 \text{ in}^2$$

CHECK  $\phi M_n > M_u$

$$a = \frac{(5.00)(60)}{0.85(5)(18)} = 3.92 \text{ in}$$

$$c = \frac{3.92}{0.80} = 4.9 \text{ in}$$

$$\epsilon_s = \frac{0.003}{4.90}(17.4 - 4.9) = 0.00765 > 0.005 \rightarrow \phi = 0.9$$

$$\phi M_n = \frac{0.9(5.00)(60)}{12} \left( 17.4 - \frac{3.92}{2} \right) = 347 \text{ k} > M_u = 306 \text{ k} \therefore \text{OK}$$

DESIGN OF SHEAR REINFORCEMENT

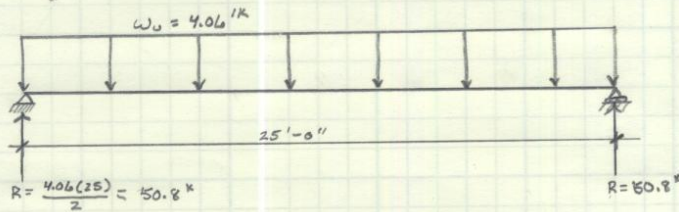
CONCRETE SHEAR STRENGTH

$$V_c = 2 \cdot \lambda \cdot \sqrt{f'_c} \cdot b_w \cdot d = \frac{2(\sqrt{5000})(18)(17.4)}{1000} = 44.3^k$$

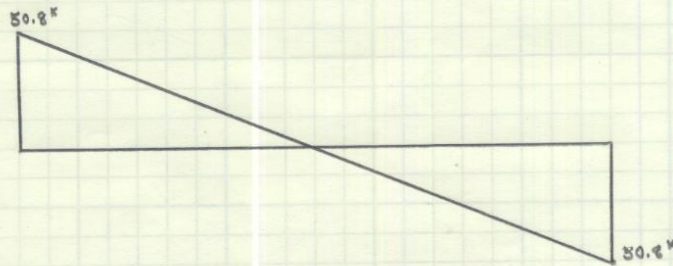
$$\phi V_n = 0.5(0.75)(44.3) = 16.6^k$$

SHEAR STRENGTH REQUIRED BY SHEAR REINFORCING

$V_u$  @  $d$  AWAY FROM FACE OF SUPPORT



LOADING



$$V_u @ d = 50.8 - 4.06 \left( \frac{17.4}{2} \right) = 44.9^k$$

$$V_s = \frac{44.9}{0.75} - 44.3 = 15.6^k$$

$$\leq \frac{3 \cdot \sqrt{5000} (18)(17.4)}{1000} = 177^k \quad \therefore \text{OK}$$

MAXIMUM SPACING

$$4 \cdot \sqrt{5000} (18)(17.4) = 88.5^k > V_s \rightarrow s_{MAX} \leq \left\{ \begin{array}{l} \frac{17.4}{2} \\ 24" \end{array} \right. = 8.7" \rightarrow 8"$$

MINIMUM REINFORCEMENT

$$A_{V,MIN} = \text{MAX} \left\{ \begin{array}{l} \frac{0.75 \cdot \sqrt{5000} (18)(8)}{60,000} = 0.127 \text{ in}^2 \\ \frac{50(18)(8)}{60,000} = 0.120 \text{ in}^2 \end{array} \right. \quad \text{USE \# 3 STIRRUPS @ 8" o.c.}$$

$$A_V = 2 \text{ LEGS} \times 0.11 = 0.22 \text{ in}^2 > 0.127$$

PROVIDE #3 x [ ] @ 8" o.c.

DESIGN OF 8" SLAB STRIP

$$l = 8' - 3" = 8.25'$$

TRIAL SIZE  $\rightarrow h = 8 \text{ in}$   $b = 8 \text{ in}$

LOADS

DEAD LOAD

- 8 in CMU WALL = 413 lb/ft
- SELFWEIGHT =  $(\frac{8}{12})^2 (150) = 46.7 \text{ lb/ft}$
- TOTAL = 0.680 k/ft

LIVE LOAD

$$= 100 (\frac{8}{12}) = 46.7 \text{ lb/ft} = 0.0467 \text{ k/ft}$$

FACTORED LOAD

$$w_u = 1.4(0.680) = 0.952 \text{ k/ft}$$

$$M_u = \frac{0.952(8.25)^2}{8} = 8.10 \text{ k}$$

ESTIMATE  $A_s$

$$d = 8 - 1.0 - 0.375 - \frac{1.00}{2} = 6.13 \text{ in}$$

$$A_s = \frac{8.10}{4(6.13)} = 0.330 \text{ in}^2 \rightarrow \text{PROVIDE (1) \#8} \rightarrow A_s = 0.79 \text{ in}^2$$

CHECK  $\phi M_n > M_u$

$$a = \frac{(0.79)(40)}{(0.85)(8)(8)} = 1.39 \text{ in} \quad c = \frac{1.39}{0.80} = 1.74 \text{ in}$$

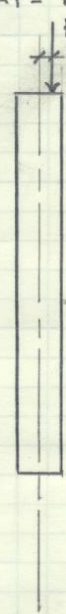
$$\epsilon_s = \frac{0.003}{1.74} (6.13 - 1.74) = 0.00757 > 0.005$$

$$\phi M_n = \frac{0.9(0.79)(40)(6.13 - \frac{1.39}{2})}{12} = 19.3 \text{ k} > M_u = 8.10 \text{ k} \therefore \text{OK}$$

## Gravity Column Design

SMALL GRAVITY COLUMN DESIGN

$A_T = 96.0 \text{ ft}^2$



$P_D = 47.3 \text{ K}$   
 $P_L = 27.0 \text{ K}$

$K_C = 1.2D + 1.6L$

$P_U = 1.2(47.3) + 1.6(27.0) = 100 \text{ K}$

$M_{U,1D} = \frac{100 \left(\frac{3}{2}\right)}{2} = 12.5 \text{ K} = 150,000 \text{ in}\cdot\text{lb}$

$I_G = \frac{24(12)^3}{12} = 3456 \text{ in}^4$        $E = 4030509 \text{ psi}$

$I_{CR} = 0.70 I_G = 2419 \text{ in}^4$

P-Δ EFFECTS →  $\frac{k \cdot l_u}{r} = \frac{0.7(15 \times 12)}{(12)(0.3)} = 35 > 22$

$\delta_u = \frac{5(150,000)(15 \times 12)^2}{48(4030509)(2419)} = 0.0519 \text{ in}$

$M_{U2} = 150,000 + 100,000(0.0519) = 150,519 \text{ in}\cdot\text{lb}$

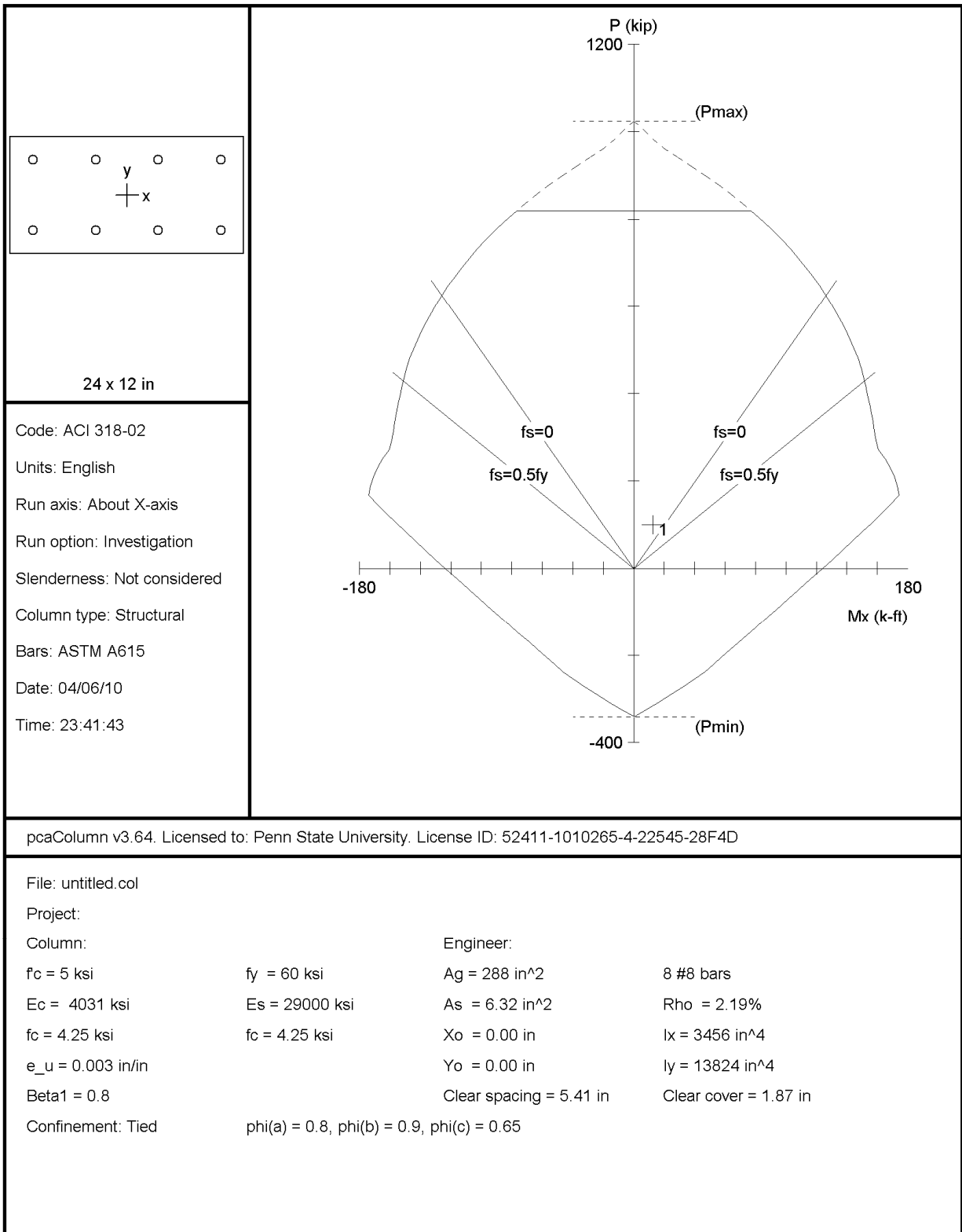
$P_U = 100 \text{ K}$      $M_U = 12.5 \text{ K}$  → P-Δ EFFECTS ARE NEGLIGIBLE

$f'_c = 5000 \text{ psi}$

ESTIMATED  $A_s$

$A_s = 24(12)(0.02) = 5.76 \text{ in}^2 \rightarrow (8) \#8's$

PCA COLUMN VERIFIES THIS  $A_{ST}$



## Design of Lateral Column

DESIGN OF INTERIOR LATERAL COLUMN

$P_{DL} = 1064 \text{ K}$        $M_{LL}^{TOP} = 128 \text{ K}$   
 $P_{LL} = 352 \text{ K}$   
 $P_{WL} = 14.1 \text{ K}$

$I = \frac{24(24)^3}{12} = 27648 \text{ in}^4$   
 $I_{cr} = 0.2 I = 19354$

$f'_c = 6000 \text{ ksi}$        $E = 4,415,201$

LC:  $1.2D + 1.6W + L$

$P_u^{TOP} = 1.2(1064) + 1.6(14.1) + 352 = 1651 \text{ K}$   
 $M_u^{TOP} = 1.6(128) = 205 \text{ K}$

P-Δ EFFECTS

MAX DISP. AT TOP =  $0.47 \text{ in}$

$M_u^{MID} = \frac{1}{2} [205 + 776] = 491 \text{ K} = 5,892,000 \text{ in-lb}$

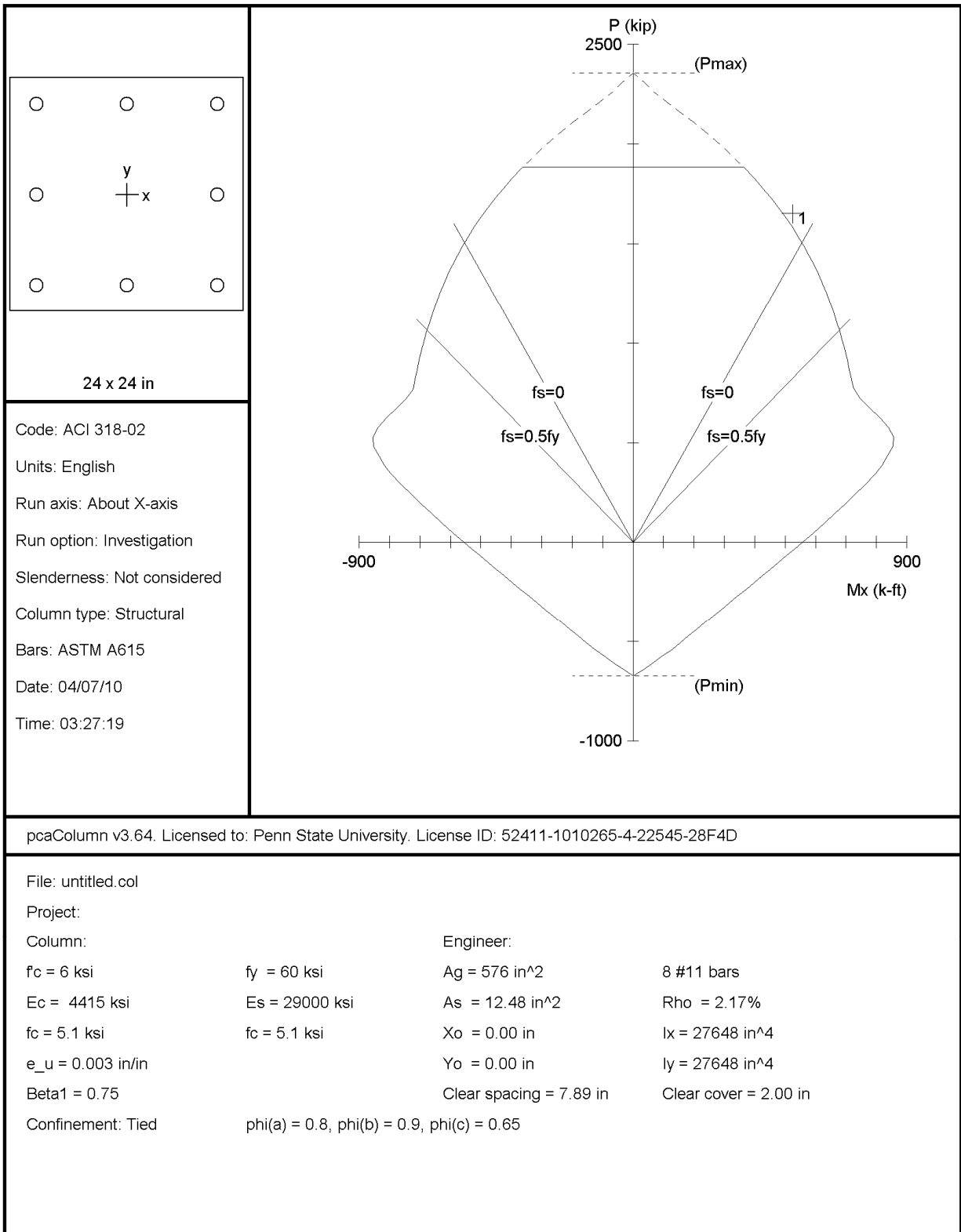
$\delta_u = \frac{5(5,892,000)(15 \times 12)^2}{48(4,415,201)(19354)} = 0.234 \text{ in}$

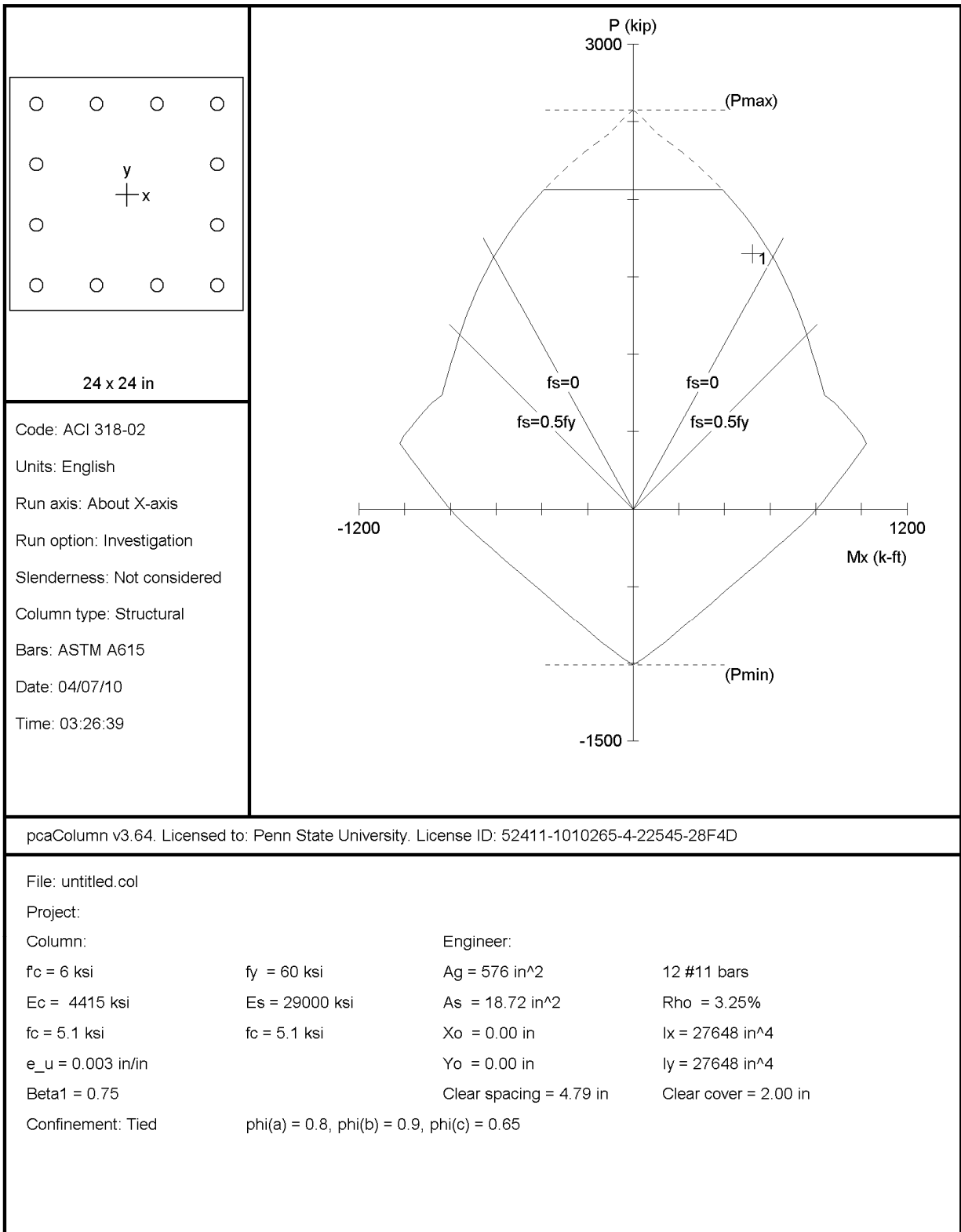
$M_{u2} = \frac{5,892,000 + 1651,000(0.234)}{1000(12)} = 523 \text{ K}$

ESTIMATED AMOUNT OF STEEL

$A_{st} = 0.02(24)(24) = 11.52 \text{ in}^2 \rightarrow \text{TRY (8) \#11'S} \rightarrow \text{NG}$   
 $P_u = 1651 \text{ K}$        $M_u = 523 \text{ K}$

TRY (12) \#11'S → GOOD  
 $\rho = 3.25\% \rightarrow \text{CANNOT LAP SPlice}$

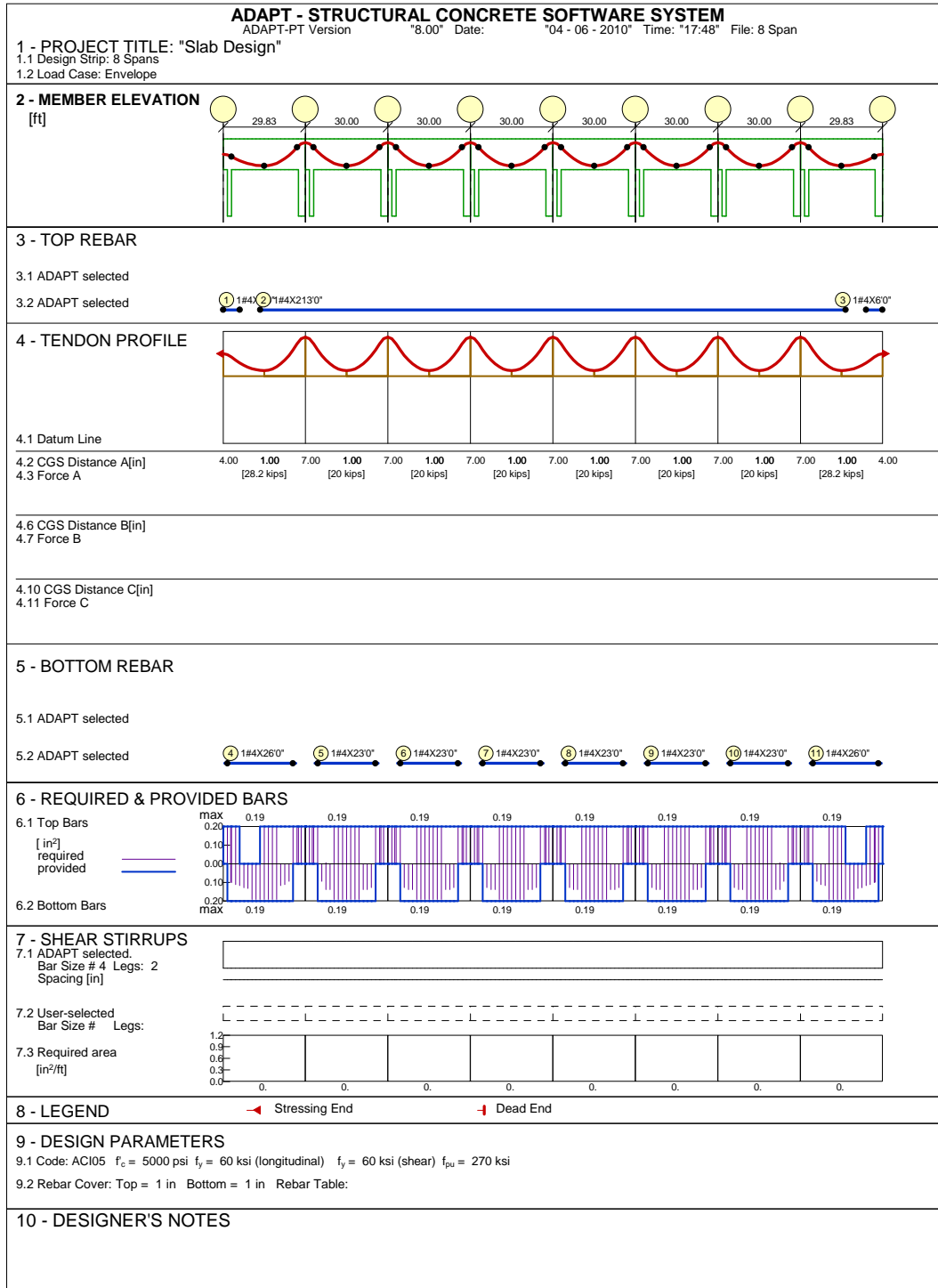






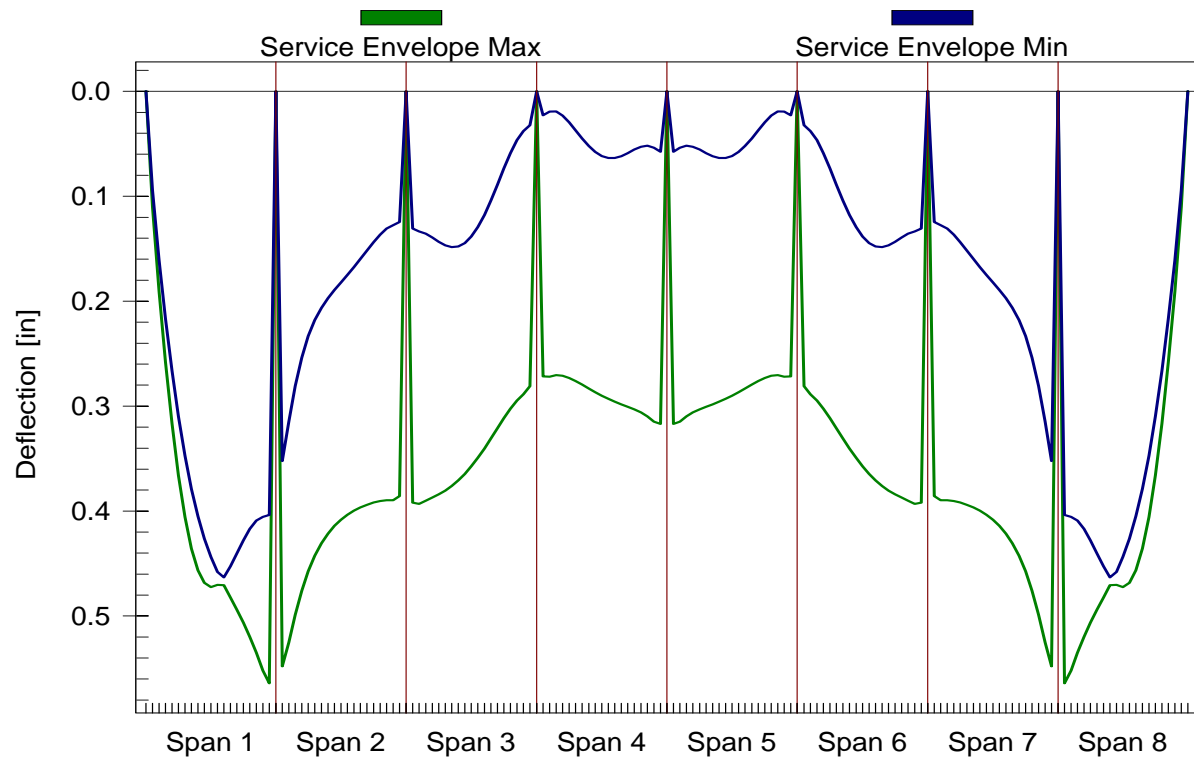
## **PT Design Summary/Deflection Diagrams**

**Slab Zone 1**

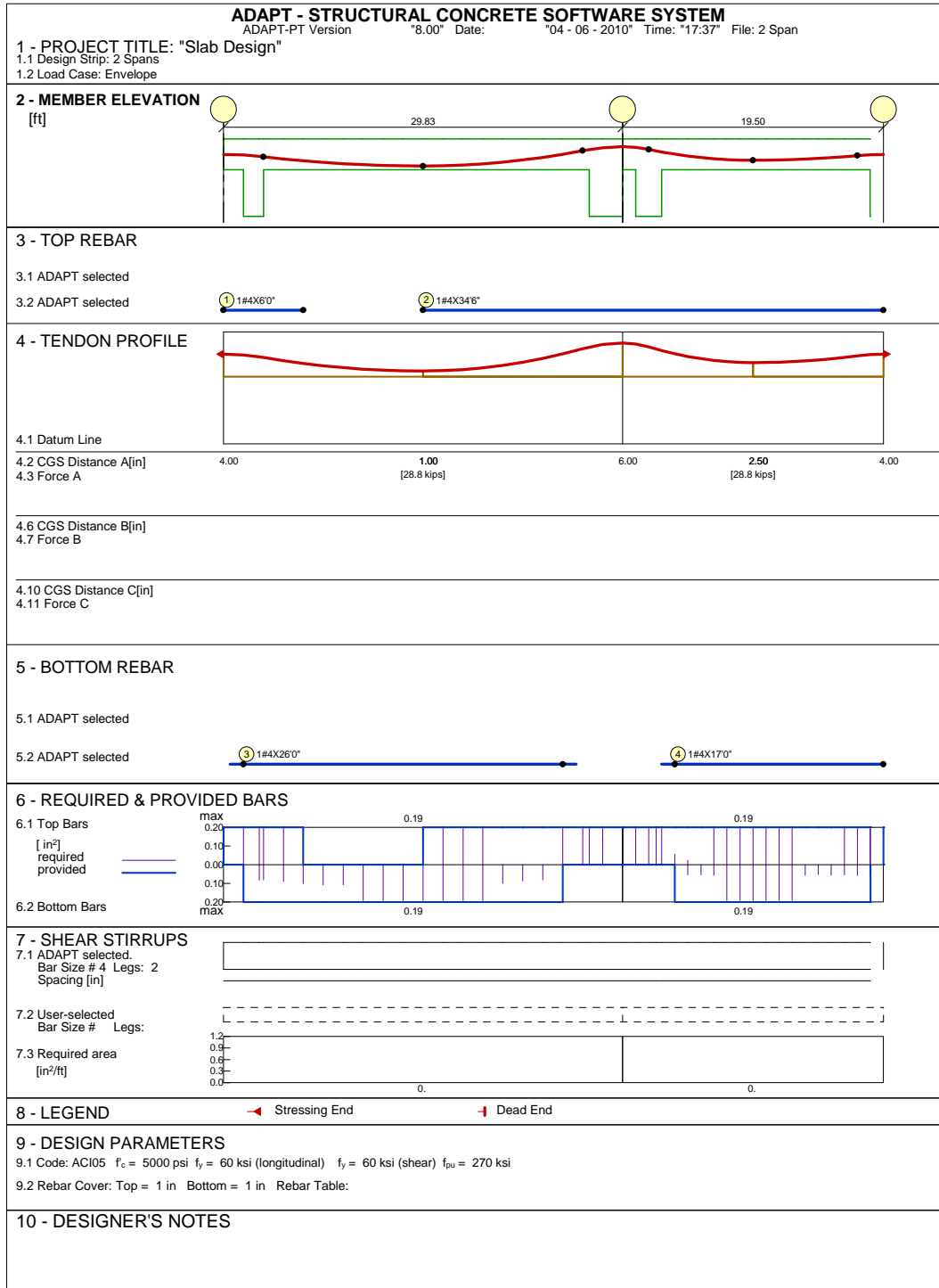


## Deflection Diagrams

File: 8 Span

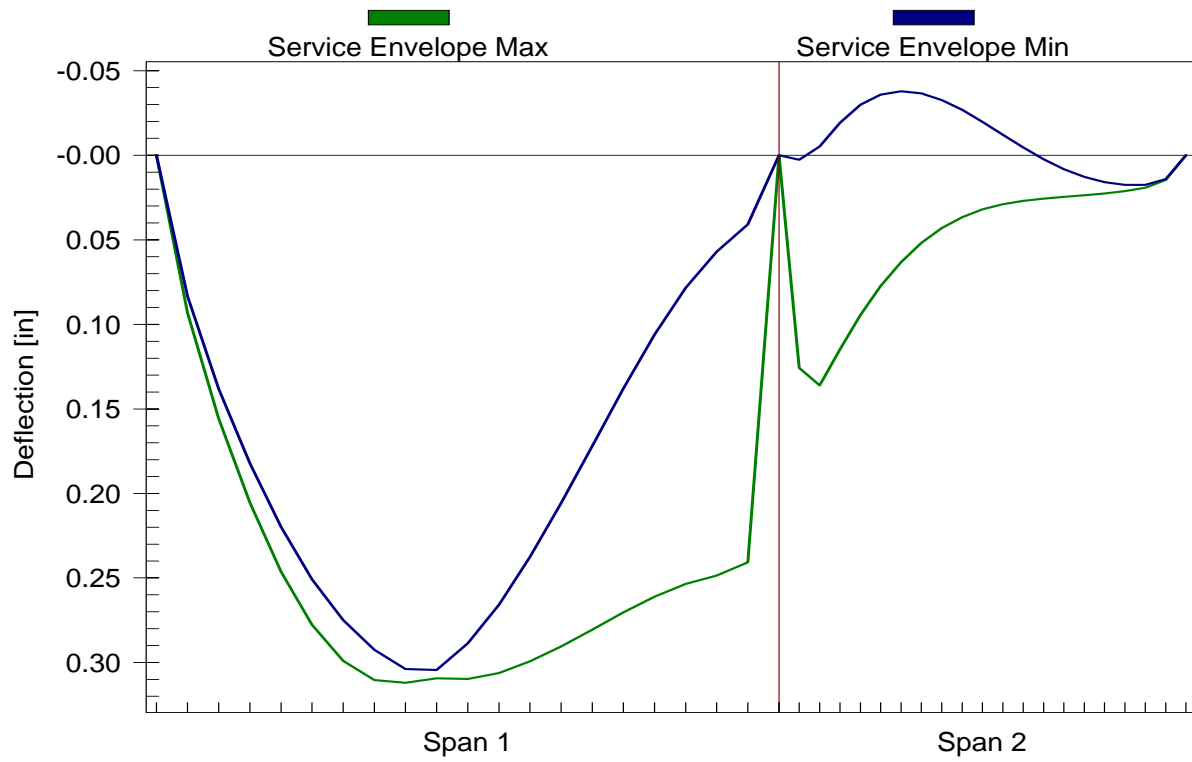


**Slab Zone 2**

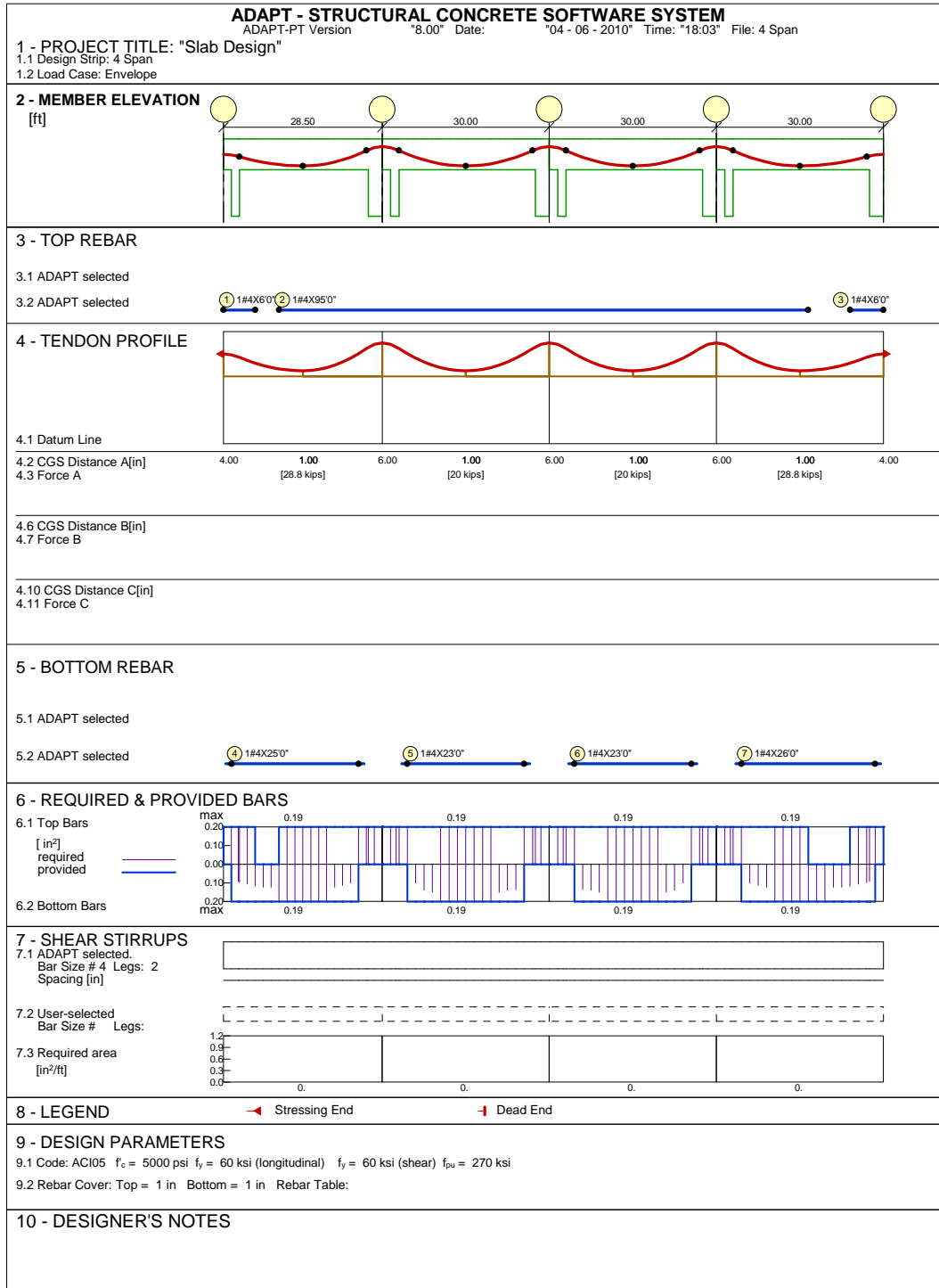


## Deflection Diagrams

File: 2 Span

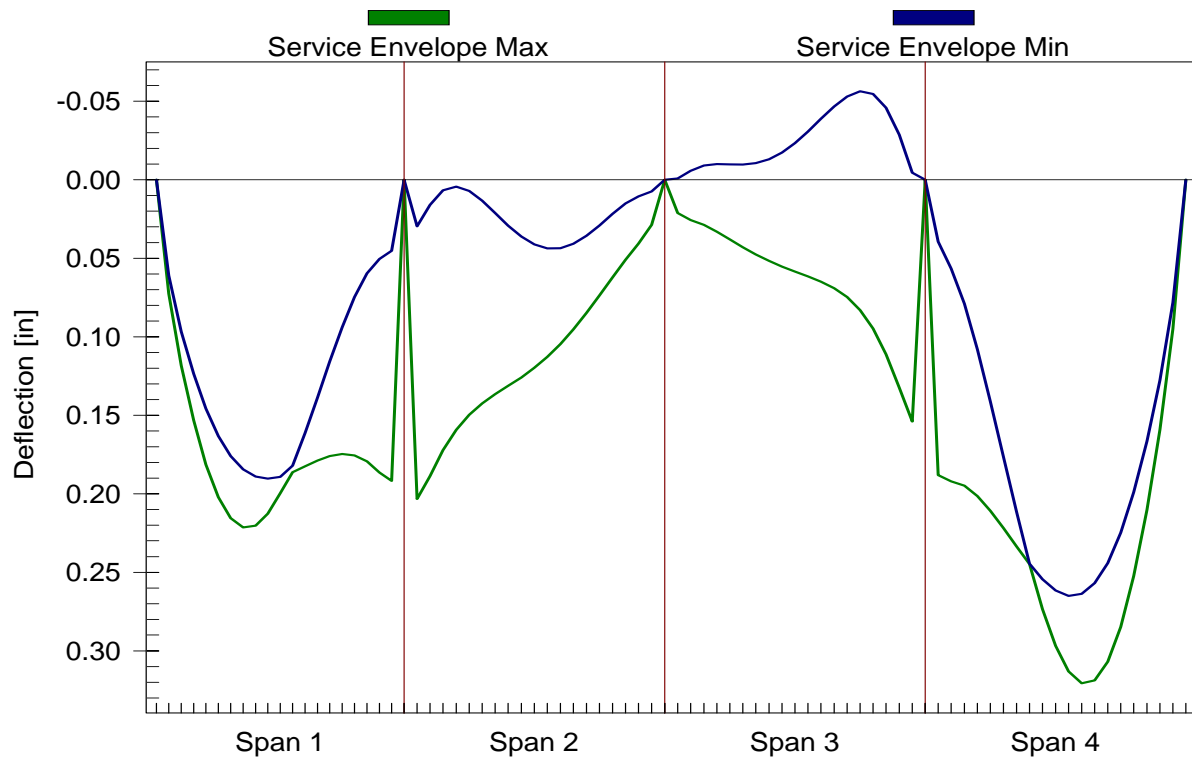


**Slab Zone 3**

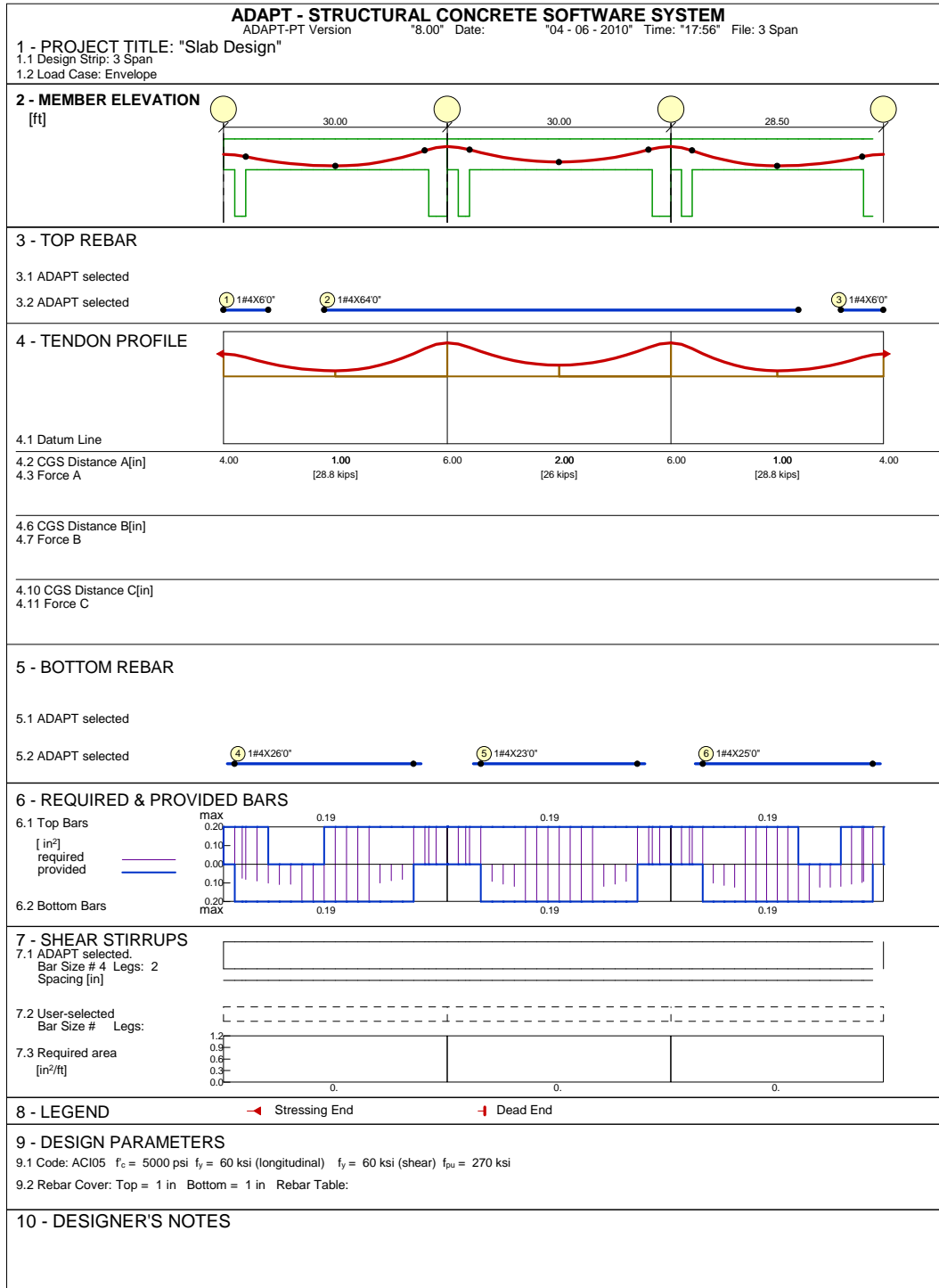


## Deflection Diagrams

File: 4 Span



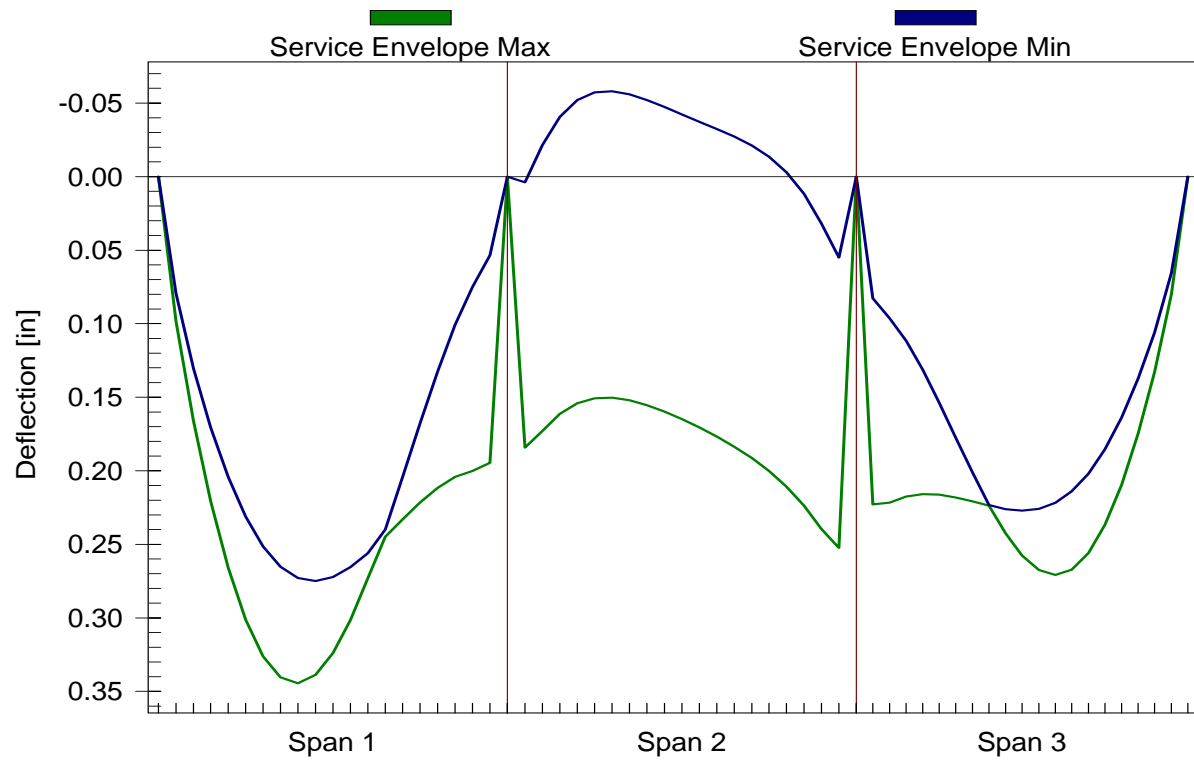
**Slab Zone 4**



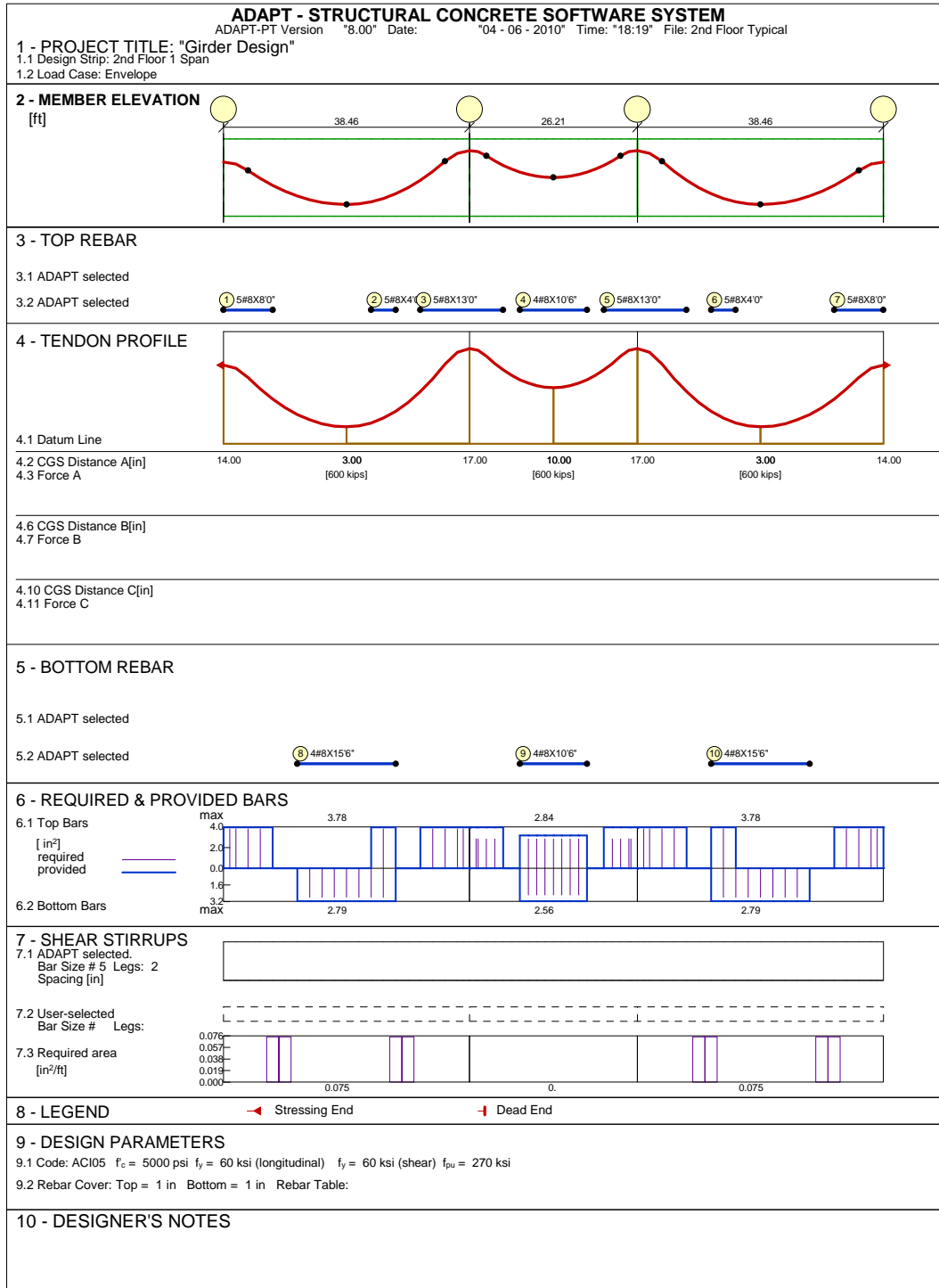


## Deflection Diagrams

File: 3 Span

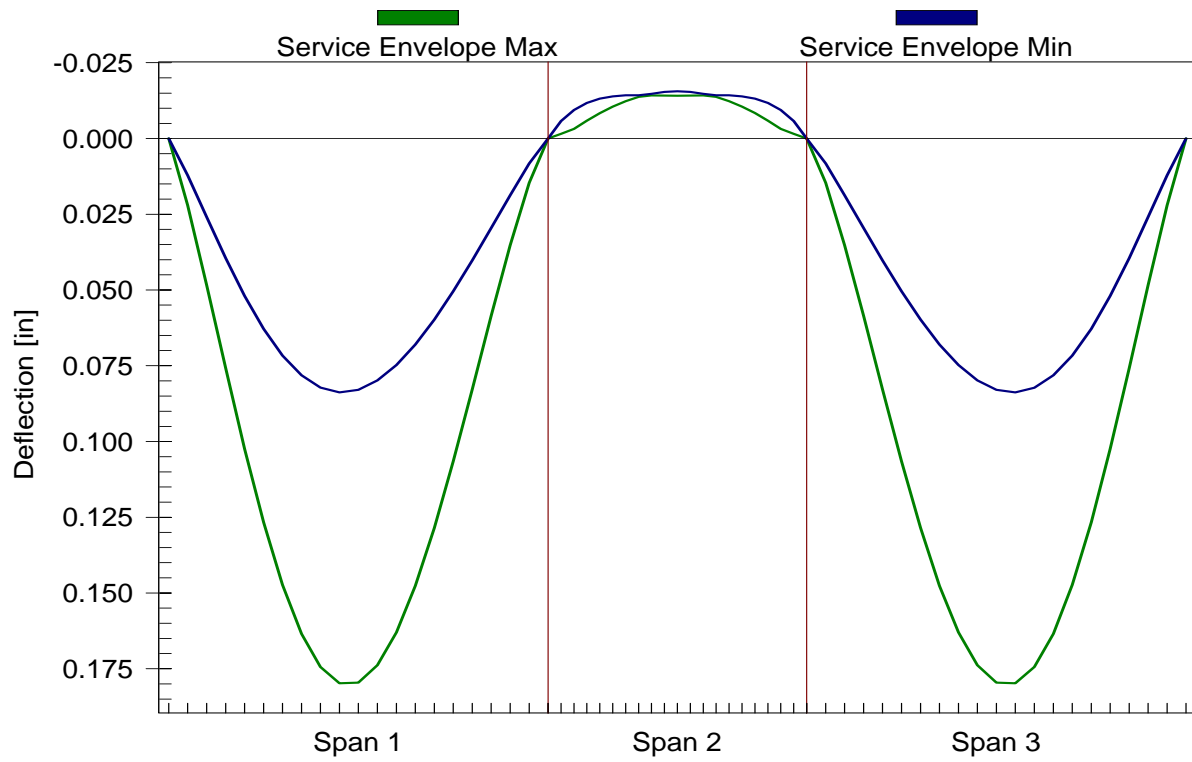


**Girder 2**

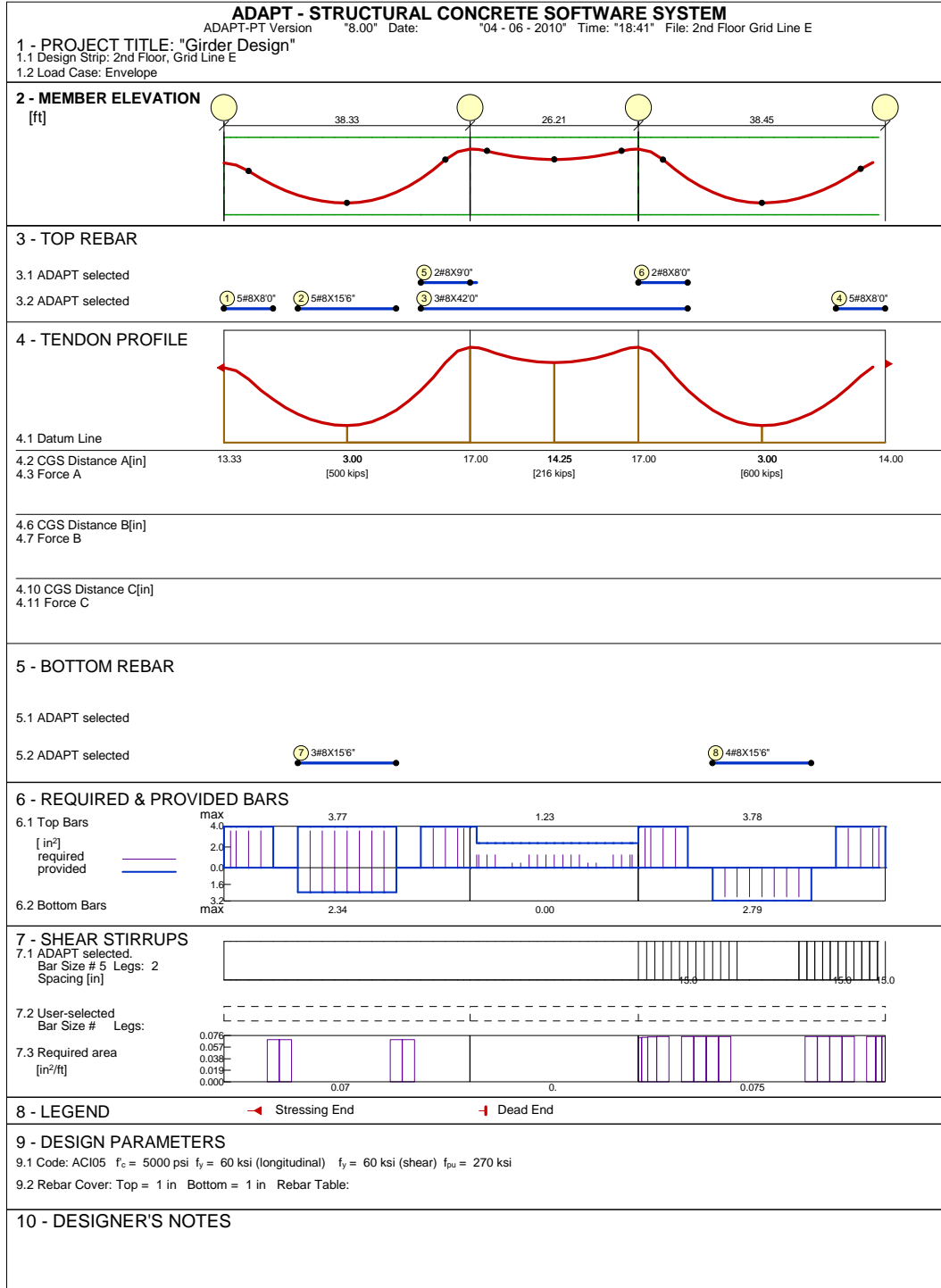


## Deflection Diagrams

File: 2nd Floor Typical

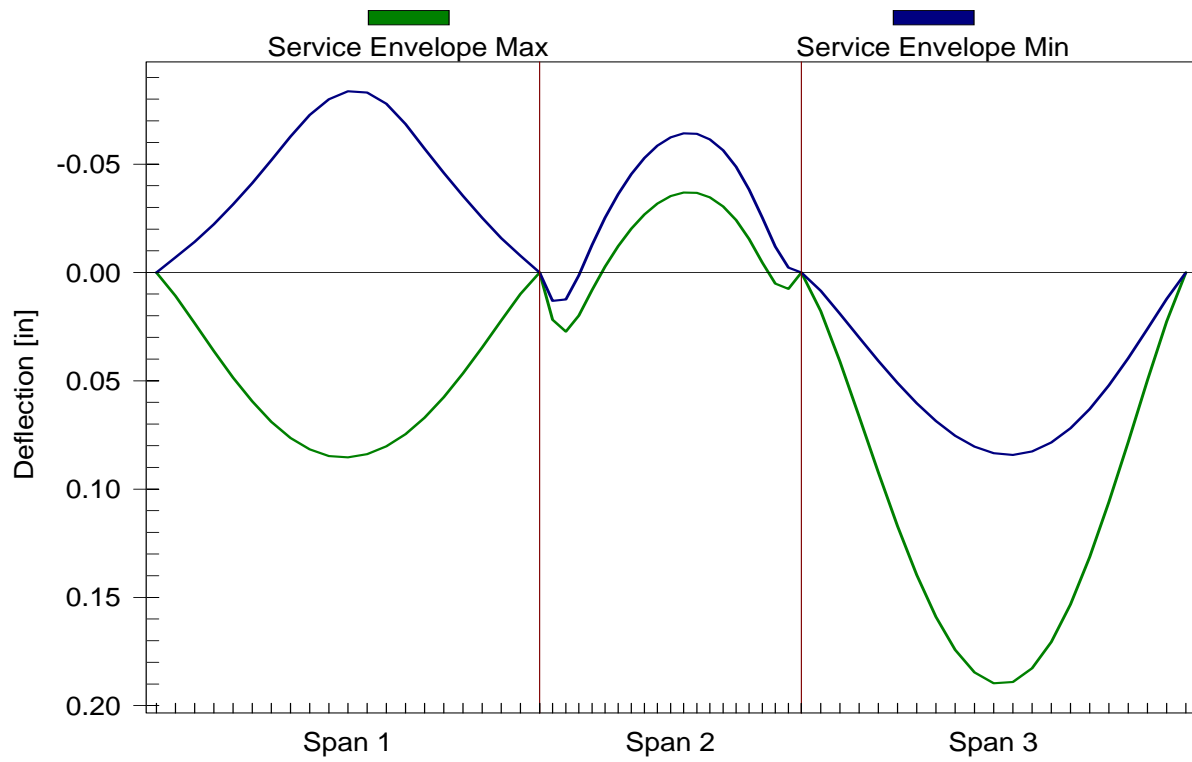


**Girder 4**

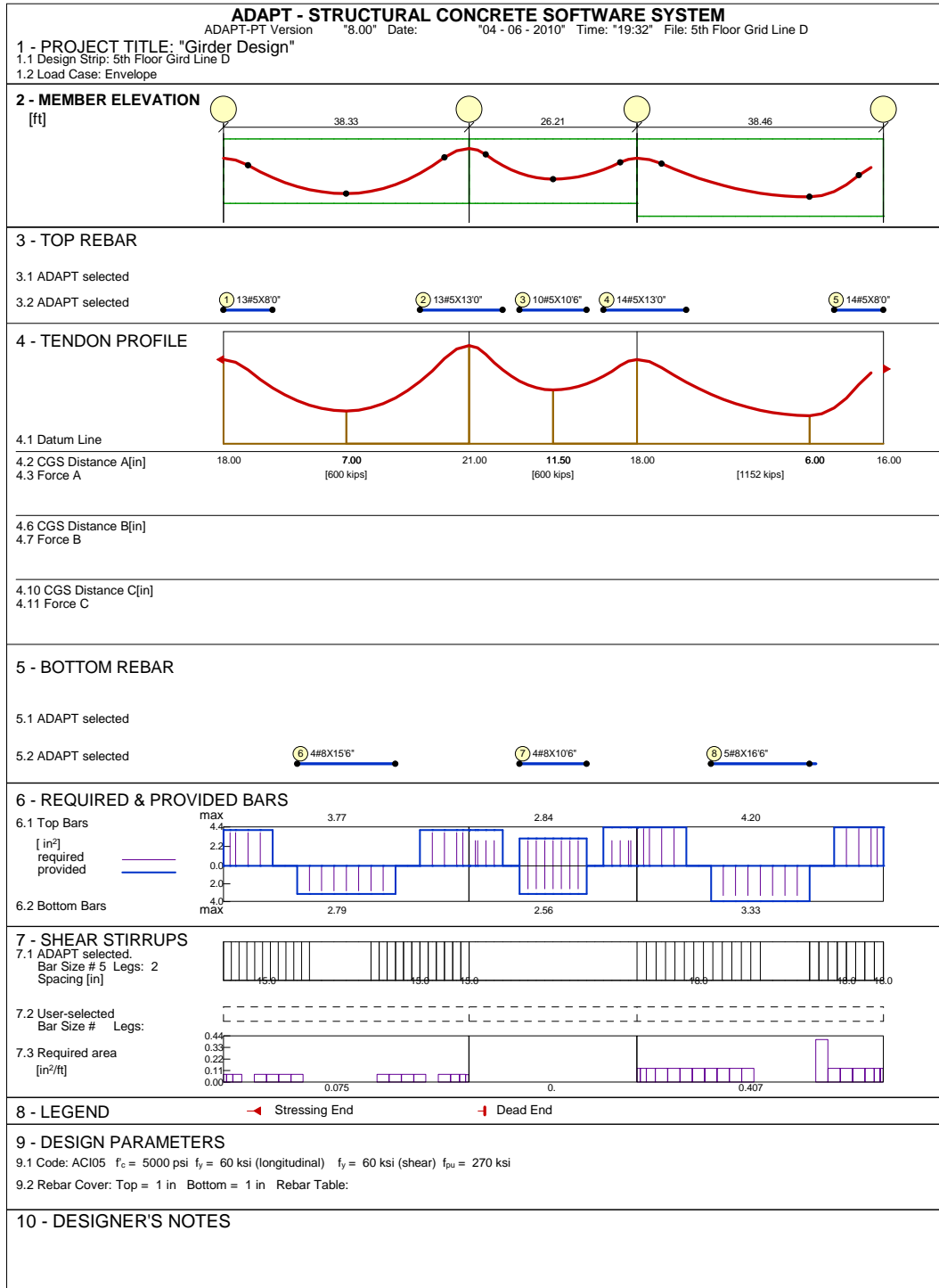


## Deflection Diagrams

File: 2nd Floor Grid Line E

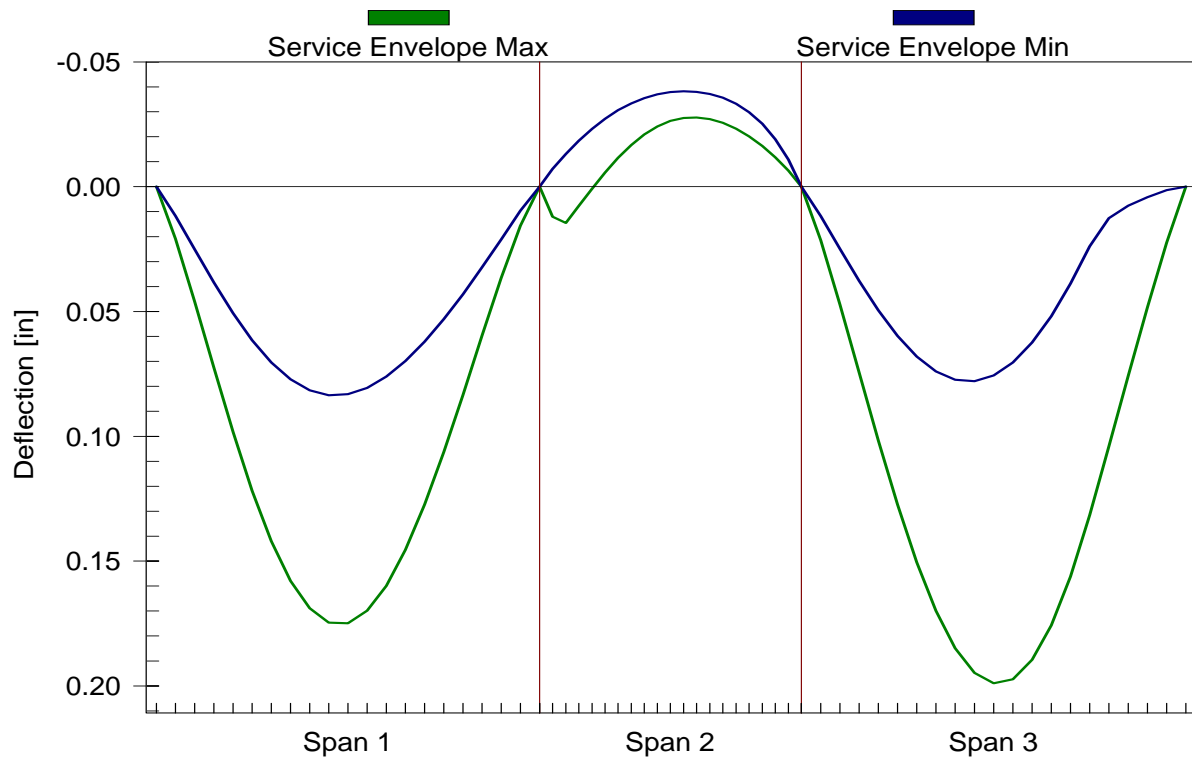


5<sup>th</sup> Floor Girder



## Deflection Diagrams

File: 5th Floor Grid Line D



## Detailed Cost Estimate

Post-Tensioned Structure						
<b>Formwork</b>						
<b>Columns</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
Job Built Plywood 24" x 24", 4 use		238	\$ 0.82	\$ 4.18	\$ -	\$ 5.00
	<b>Total</b>	18	\$ 3,497.79	\$ 17,830.21	\$ -	\$ 21,328.00
<b>Slab</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
Flat, plate, job-built plywood, 4 use		560	\$ 1.49	\$ 2.73	\$ -	\$ 5.98
	<b>Total</b>	45	\$ 37,846.00	\$ 69,342.00	\$ -	\$ 151,892.00
<b>Girders</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
Job-built plywood, 30" wide, 4 use		238	\$0.82	\$4.18	\$ -	\$5.00
	<b>Total</b>	33	\$ 6,462.67	\$ 32,943.83	\$ -	\$ 39,406.50
	<b>Total</b>	96	\$ 47,806.46	\$ 120,116.04	\$ -	\$ 212,626.50
<b>Reinforcement</b>						
<b>Columns</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
Columns, #8 to #18		2.38	\$1,759.25	\$564.82	\$ -	\$2,324.07
	<b>Total</b>	5	\$ 20,196.19	\$ 6,484.13	\$ -	\$ 26,680.32
<b>Slab</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
Elevated slabs, #4 to #7		2.9	\$1,872.75	\$446.39	\$ -	\$2,319.14
	<b>Total</b>	8	\$ 45,208.19	\$ 10,775.85	\$ -	\$ 55,984.04
<b>Girders</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
Elevated slabs, #4 to #7		2.7	\$1,759.25	\$482.83	\$ -	\$2,242.08
	<b>Total</b>	2	\$ 7,766.21	\$ 2,131.45	\$ -	\$ 9,897.66
	<b>Total</b>	15	\$ 73,170.58	\$ 19,391.44	\$ -	\$ 92,562.03
<b>Stressing Tendons</b>						
<b>Slab</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
UngROUTED single strand, 100' slab, 25 kip		1200	\$0.70	\$1.09	\$0.02	\$1.81
	<b>Total</b>	14	\$ 12,139.33	\$ 18,902.67	\$ 24.00	\$ 31,388.84
<b>Girders</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
UngROUTED single strand, 100' slab, 25 kip		1200	\$0.70	\$1.09	\$0.02	\$1.81
	<b>Total</b>	8	\$ 7,068.60	\$ 11,006.82	\$ 24.00	\$ 18,277.38
	<b>Total</b>	23	\$ 19,207.93	\$ 29,909.49	\$ 48.00	\$ 49,666.22
<b>Concrete</b>						
<b>Columns</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
5000 psi concrete		0	\$149.29	\$ -	\$ -	\$149.29
	<b>Total</b>	0	\$ 11,792.80	\$ -	\$ -	\$ 11,792.80
<b>Slab</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
5000 psi concrete		0	\$149.29	\$ -	\$ -	\$149.29
	<b>Total</b>	0	\$93,604.83	\$ -	\$ -	\$93,604.83
<b>Girders</b>						
Type		Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total
5000 psi concrete		0	\$149.29	\$ -	\$ -	\$149.29
	<b>Total</b>	0	\$26,125.75	\$ -	\$ -	\$26,125.75
	<b>Total</b>	0	\$ 131,523.38	\$ -	\$ -	\$ 131,523.38

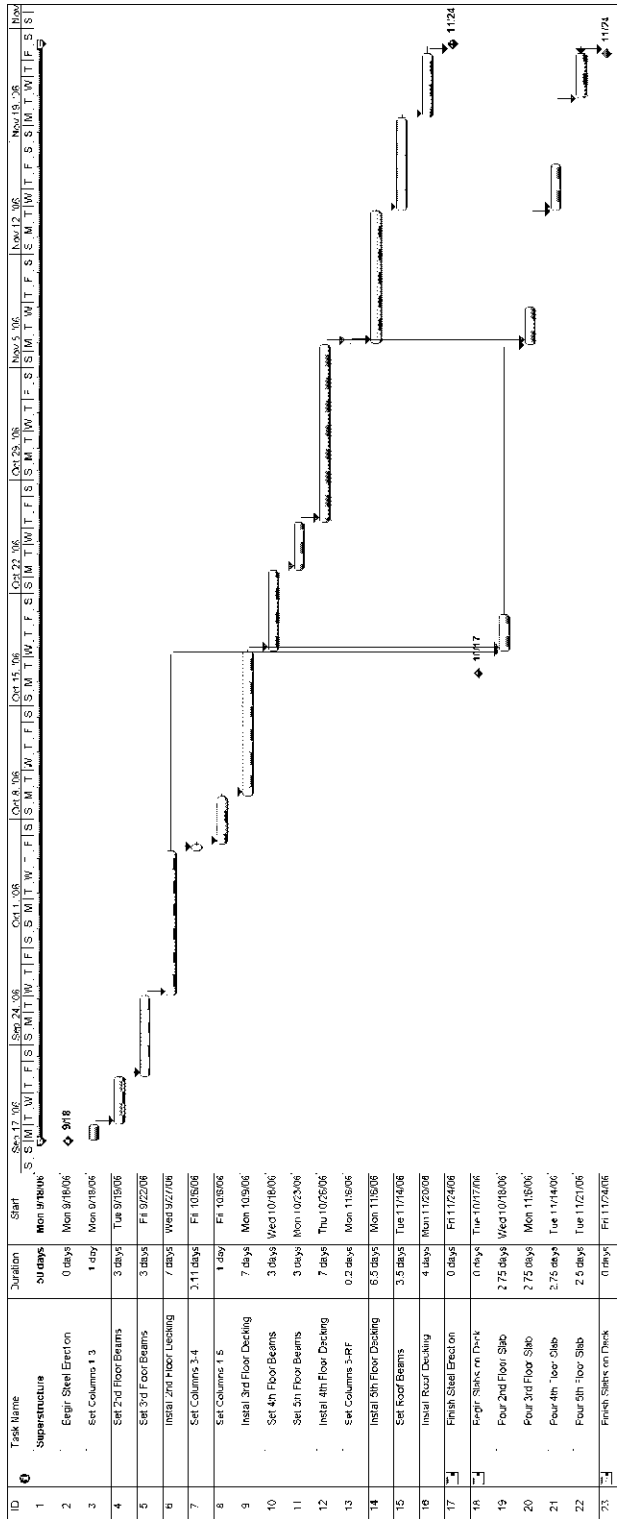


<b>Placing Concrete</b>						
<b>Columns</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
24" thick, pumped	92	\$ -	\$20.80	\$8.91	\$29.71	
<b>Total</b>	<b>1</b>	<b>\$ -</b>	<b>\$ 1,643.05</b>	<b>\$ 703.82</b>	<b>\$ 2,346.87</b>	
<b>Slab and Girders</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
6" to 10" thick, pumped	180	\$ -	\$10.66	\$4.55	\$15.21	
<b>Total</b>	<b>4</b>	<b>\$ -</b>	<b>\$ 8,549.32</b>	<b>\$ 3,649.10</b>	<b>\$ 12,198.42</b>	
<b>Total</b>	<b>5</b>	<b>\$ -</b>	<b>\$ 10,192.37</b>	<b>\$ 4,352.92</b>	<b>\$ 14,545.29</b>	
<b>Finishing Concrete</b>						
<b>Slab</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
Power screed, bull float, machine float (ride on)	4000	\$ -	\$0.19	\$0.06	\$0.25	
<b>Total</b>	<b>6</b>	<b>\$ -</b>	<b>\$ 4,826.00</b>	<b>\$ 1,524.00</b>	<b>\$ 6,350.00</b>	
<b>Total</b>	<b>6</b>	<b>\$ -</b>	<b>\$ 4,826.00</b>	<b>\$ 1,524.00</b>	<b>\$ 6,350.00</b>	
<b>Total</b>	<b>139</b>	<b>271708</b>	<b>\$179,609.34</b>	<b>\$ 4,400.92</b>	<b>\$500,923.42</b>	

Steel Structure						
<b>Structural Steel</b>						
<b>Columns</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
W10x68 Typical	984	113.9	2.68	1.83	117.4	
<b>Total</b>	0.365762195	\$ 40,993.75	\$ 964.56	\$ 658.64	\$ 42,253.43	
<b>Beams</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
W18x50 Typical	912	83.9	4.18	2.13	90.21	
<b>Total</b>	1.637017544	\$ 125,259.34	\$ 6,240.57	\$ 3,180.00	\$ 134,679.92	
<b>Steel Decking</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
18 gauge, over 500 squares	3600	3.69	0.44	0.04	4.17	
<b>Total</b>	7.055555556	\$ 93,726.00	\$ 11,176.00	\$ 1,016.00	\$ 105,918.00	
<b>Welded Wire Fabric</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
6x6 W1.4-W1.4	35	20.49	18.68	0	39.17	
<b>Total</b>	8890	\$ 5,204.46	\$ 4,744.72	\$ -	\$ 9,949.18	
<b>Total</b>	8899.058335	\$ 265,183.55	\$ 23,125.85	\$ 4,854.64	\$ 292,800.54	
<b>Concrete</b>						
<b>Slab</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
5000 psi concrete	0	\$149.29	\$0.00		\$149.29	
<b>Total</b>	0	\$61,806.06	\$0.00		\$61,806.06	
<b>Total</b>	0	\$ 61,806.06	\$ -	\$ -	\$ 61,806.06	
<b>Placing Concrete</b>						
<b>Slab</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
6" to 10" thick, pumped	180	\$0.00	\$10.66	\$4.55	\$15.21	
<b>Total</b>	2	\$ -	\$ 4,413.24	\$ 1,883.70	\$ 6,296.94	
<b>Total</b>	2	\$ -	\$ 4,413.24	\$ 1,883.70	\$ 6,296.94	
<b>Finishing Concrete</b>						
<b>Slab</b>						
Type	Daily Output	Bare Material	Bare Labor	Bare Equipment	Bare Total	
Power screed, bull float, machine float (ride on)	4000	\$0.00	\$0.19	\$0.06	\$0.25	
<b>Total</b>	6	\$ -	\$ 4,826.00	\$ 1,524.00	\$ 6,350.00	
<b>Total</b>	6	0	\$ 4,826.00	\$ 1,524.00	\$ 6,350.00	
<b>Total</b>	\$ 8,907.71	\$ 326,989.61	\$ 32,365.09	\$ 8,262.34	\$ 367,253.54	

## **Project Schedules**

Steel



Project: Steel Schedule.app  
 Date: Wed 4/7/10

Task: Spill

Progress Milestones: [Progress bar]

Summary: [Summary bar]

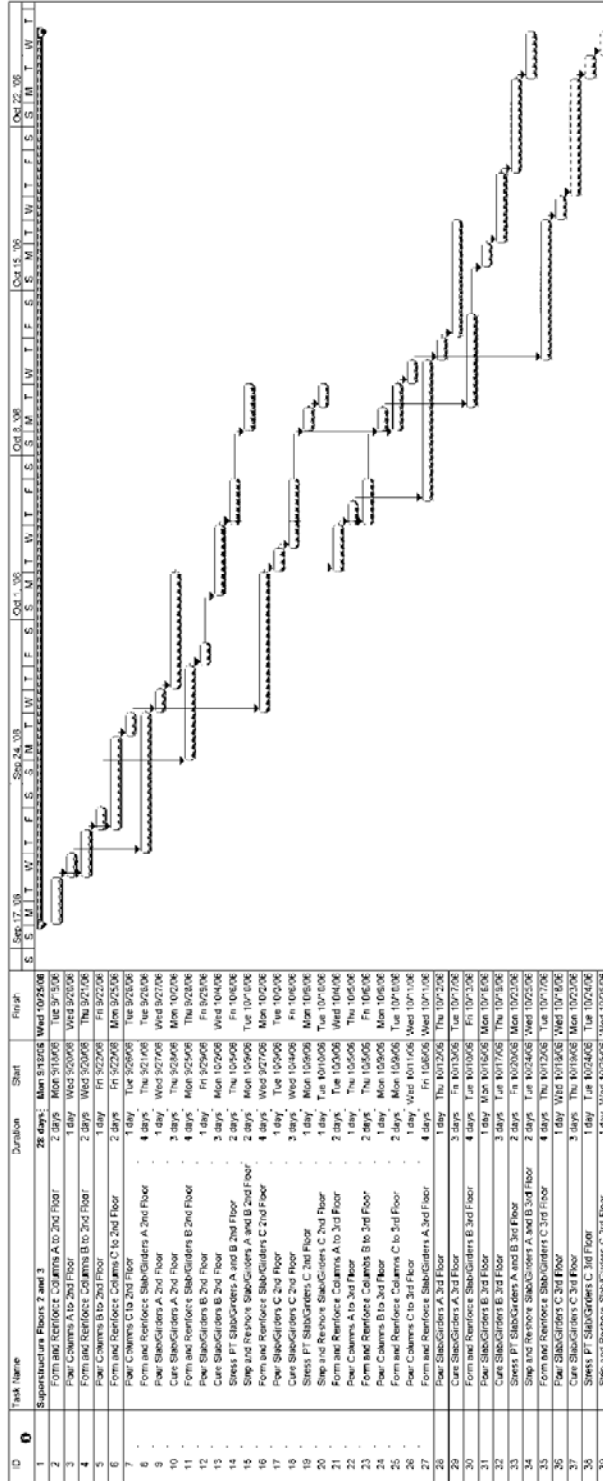
External Tasks: [External Tasks bar]

External Milestone: [External Milestone bar]

Deadline: [Deadline bar]

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Concrete



Project Summary	Summary	External Tasks	Deadline
Project Summary	Summary	External Tasks	Deadline