Belmont Executive Center; Building A

Ashburn, VA

Technical Report 2
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Executive Summary

The purpose of Technical Report 2 is to perform a pro/con analysis on alternative floor systems for Building A. Three systems were chosen to study; two-way flat slab, pre-cast concrete plank on steel, two-way post-tensioned slab. Comparisons between the systems were made in regard to unit costs, system weight, floor depth, foundation and lateral system impact, and constructability.

Because there are both long and short spans in the building two typical bays were considered for each alternative, an exterior bay, roughly 40'x30', and an interior bay, roughly 26'x30'. Hand and computer calculations determined that the two-way post-tensioned was the thinnest system, but was also considerably heavier than the existing composite beam, girder, and decking system. The pre-cast concrete plank was found to be very light, but increased the floor depth by 14", which would be difficult integrate into the building design, because of height limitations. Like the post-tensioned system, the two-way flat slab was also thinner, but was the heaviest system.

Introduction

The Belmont Executive Center; Building A is located in the Belmont Executive Center, which will include office, retail, restaurant, daycare, and hotel spaces. Residents of the Dulles North area will be offered daily shopping, specialty shopping, and dining choices.

Building A is a 125,000 SF, 5-story office building designed to accommodate multiple tenants. The façade of the building is constructed primarily of brick on light gage metal studs. Vertical brick columns are spaced around the perimeter façade, some of which enclose structural columns, others which do not support any load. A large curtain wall system distinguishes the entrance of the building, and the corners of the building also have a curtain wall system. The structural system of the building is constructed of steel framing with light weight concrete on composite deck as the floor system. Lateral bracing is provided by four concentrically braced frames.

Each floor provides unobstructed open space on both sides of the core, and a floor to ceiling height of 9'.

Structural System

Foundation System

The foundation system is made up of spread footings located at the base of the steel columns, and range from 19’-6” square to 10’-6” square, depending on the location. Larger footings are located in the center right part of the building, to support a mechanical room and the restrooms. Smaller foundations are located at the exterior columns. All larger foundations are shown in yellow in Figure 1 below. The perimeter footings are connected by grade beams that support the masonry facade. A stepped grade beam is located just to the left of the entrance to allow a connection to the sanitary line. There is also a stepped grade beam on the right side of the building for the domestic water line and fire service line connection. The ground floor is a 5” thick concrete slab on grade reinforced with #3 rebar @ 15”o.c. running both directions. A 6” slab on grade is located to the right side of the building to support a 30 yard trash compactor, and is highlighted in purple in Figure 1. It is reinforced with #3 rebar @ 12”o.c. each way. The slabs are supported by 4” granular material, on top of compacted soil.

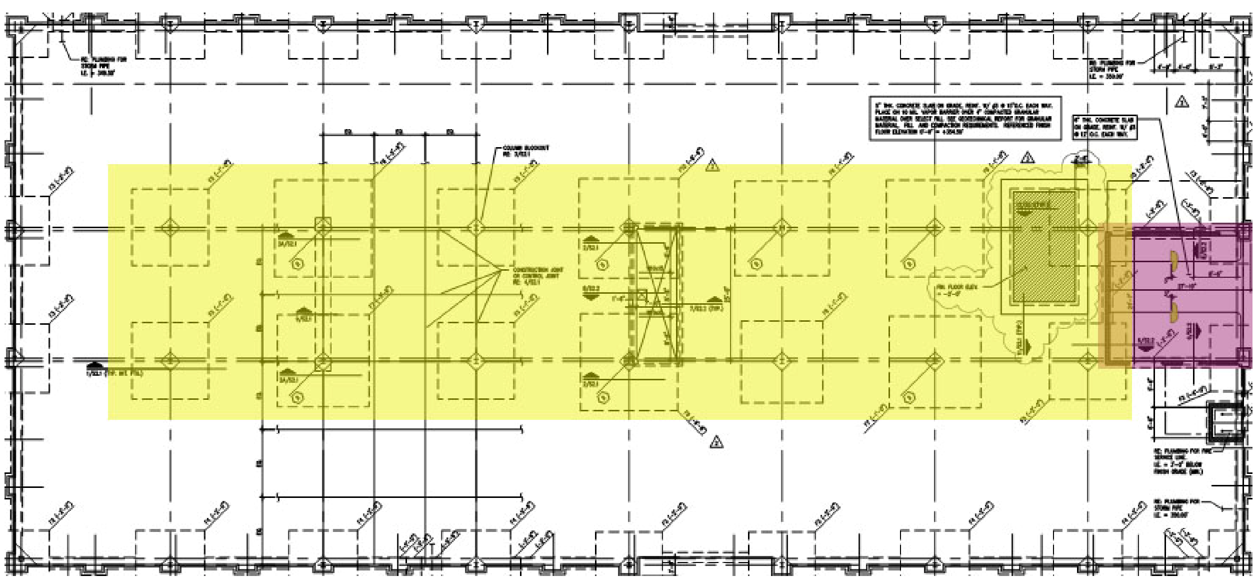


Figure 1: Foundation Layout

Column System

The floor and roof system are supported by three column lines in the north-south direction and nine rows of columns in the east-west direction. Because the exterior column spacing is dictated by the architecture of the building, the columns on the left and right side of the building are offset from those in the interior. At the corners of the building they are offset by 1'-3" and the interior columns are offset by 7 ¼". This offset creates a slight skew in the beams spanning from the exterior to the interior. Figure 2 shows the column offset. Most of the columns are W shape steel beams, and a few are HSS columns. Hollow structural steel columns are located at the front left and right corners of the building. They are also used in the left rear and right rear corners, on floors three to five, and to provide intermediate bracing below the exterior terrace on the fifth floor. The typical bay sizes for each floor is 38'x 30' and 26'x30'. Figure 3 shows the typical column layout.

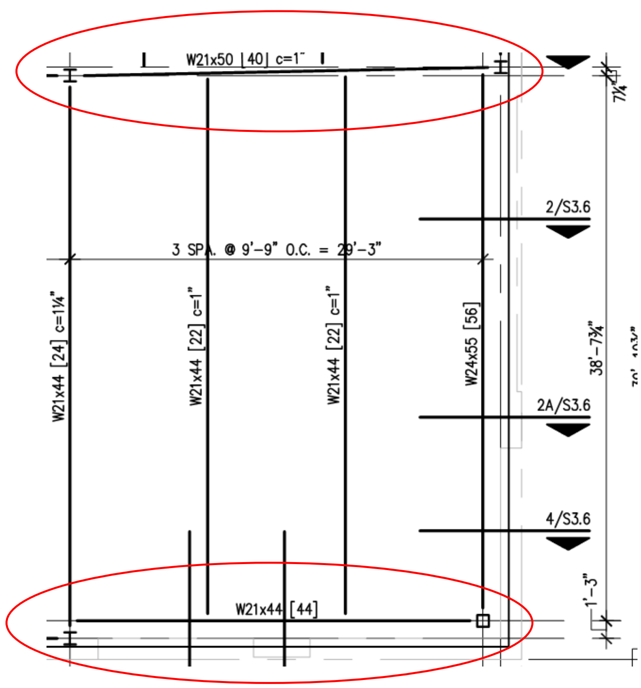


Figure 2: Column Offset

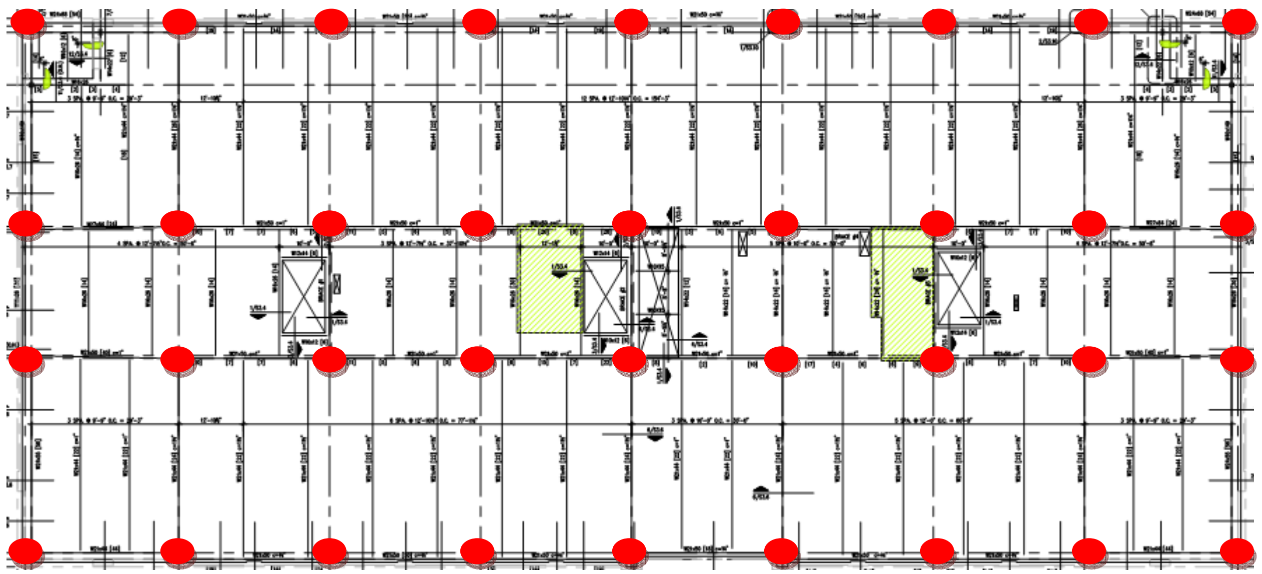


Figure 3: Column Layout

Floor System

Floors 2-4 are constructed of 3-1/4” light weight concrete, on 3” composite metal deck. The deck is reinforced by 6 x6 - W1.4 x W1.4 welded wire fabric, and supported by W-shape steel beams. There are three bays in the north-south direction, and ten in the east-west direction of the building. For reference, the outer lying bays are highlighted in red, and the middle bay is highlighted in green, see Figure 4. Typically, there are W21x44 beams spaced 12’-10 ¼” to 9’-9”, on floors 2 through 5, in the two outside bays. In the middle bay the beams are typically W16x26. Between the elevators and stairwell three, the steel beams are W14x22. Composite action is provided shear studs, and most beams also have upward camber ranging from ¾” to 1” to compensate for service and live load deflections. W 21x50 girders support the load reactions from the beams. On the second floor there is no framing at the main entrance, because this area is open to the ground floor. Floors 3-5 are framed similarly. On the fifth floor the exterior terrace floor is supported by W10x12 steel beams.

The mechanical equipment in the penthouse is supported by a typical concrete floor, constructed of lightweight concrete on composite metal deck. This is the only concrete slab on the roof level. W16x26 beams span across the bay to support the floor.

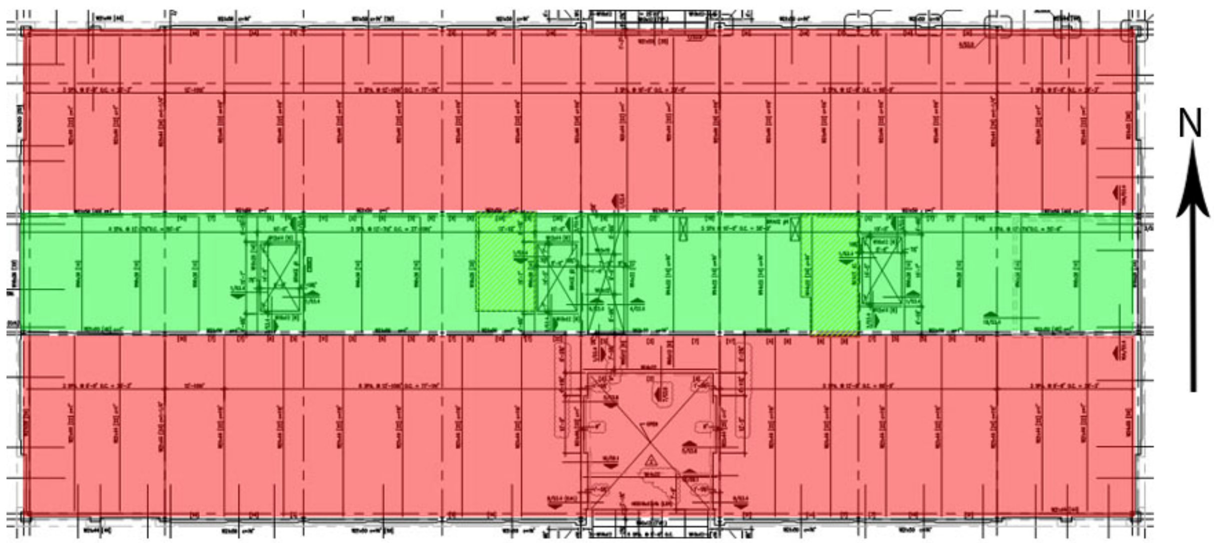


Figure 4: Typical beam size and spacing

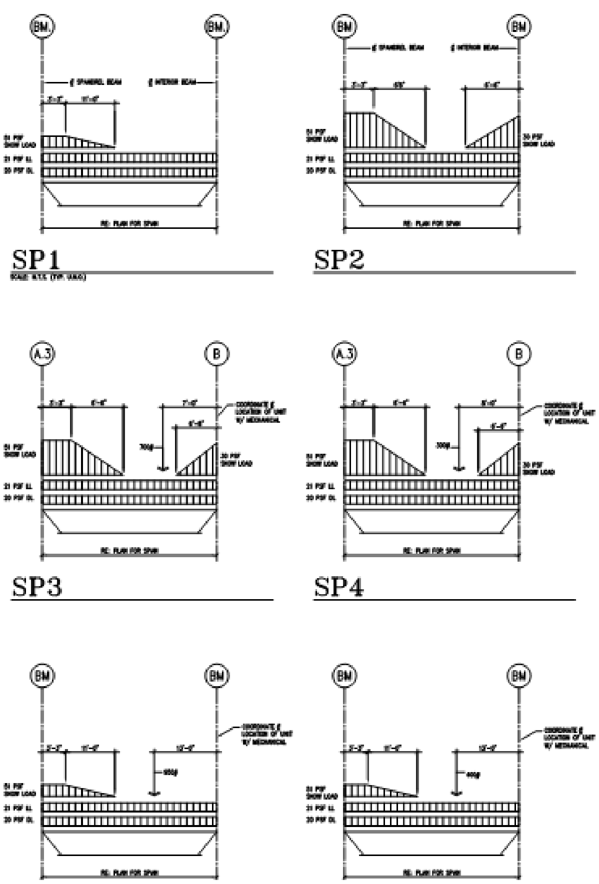


Figure 5: Special Loading Conditions

Roof System

The roof system is supported by K-series joist, spanning across the three bays in the north-south direction. All the joists in the outside two bays are spaced at 6'-0" on center. Joists in the front and rear bays were designed for specifically by the joist manufacturer for snow drifting, because this can be a critical load failure for open web joists. All joists that were specially designed are denoted by SP, and there are 6 different loading conditions. Each loading condition is shown in Figure 5. Three rows of bracing are provided in the rear bay, to prevent lateral torsional buckling. Regular K series joists ranging from 22K5 to 18K3 support the roof in the middle bay. The penthouse roof is supported by 20K3 spaced at 6'-0", with 3 rows of bridging.

The standing seam metal roof screen that shields the penthouse from view is supported by a combination of K Series joists and W shape beams. At roughly every 30' W shaped steel beams are angled at 45 degrees, and are supported by steel posts. Between the beams, four K series joists run parallel to the building perimeter. L 2 x 2 x1/8" angle provides bracing at 6', between the joists. Figure 6 shows the angled beams, highlighted in yellow, and the joists can be seen spanning between them. Figure 7 shows a typical cross section of the roof screen.

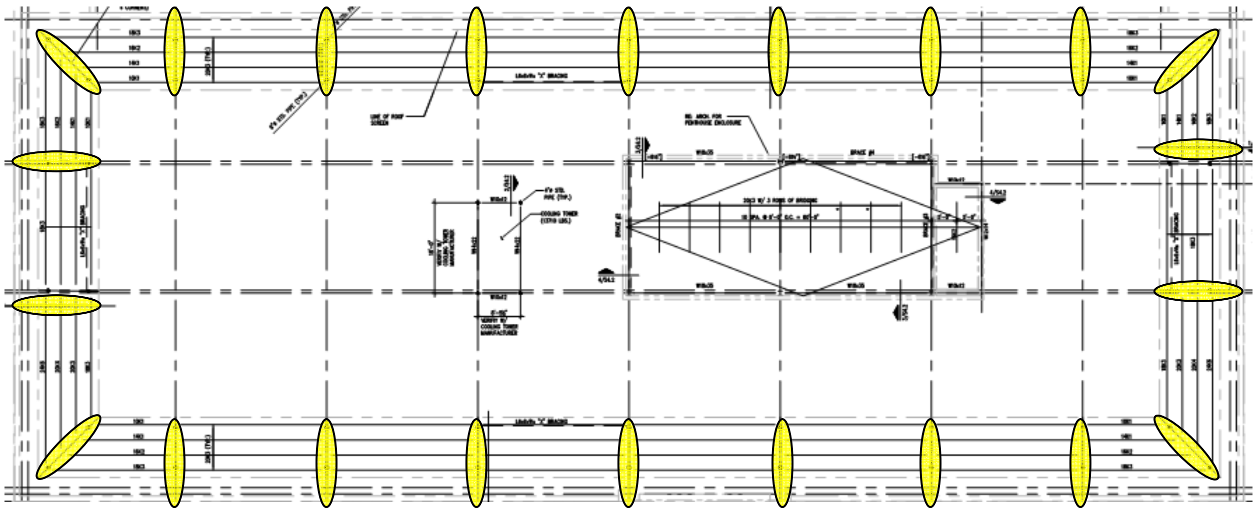


Figure 6: Angled W Shape Beams

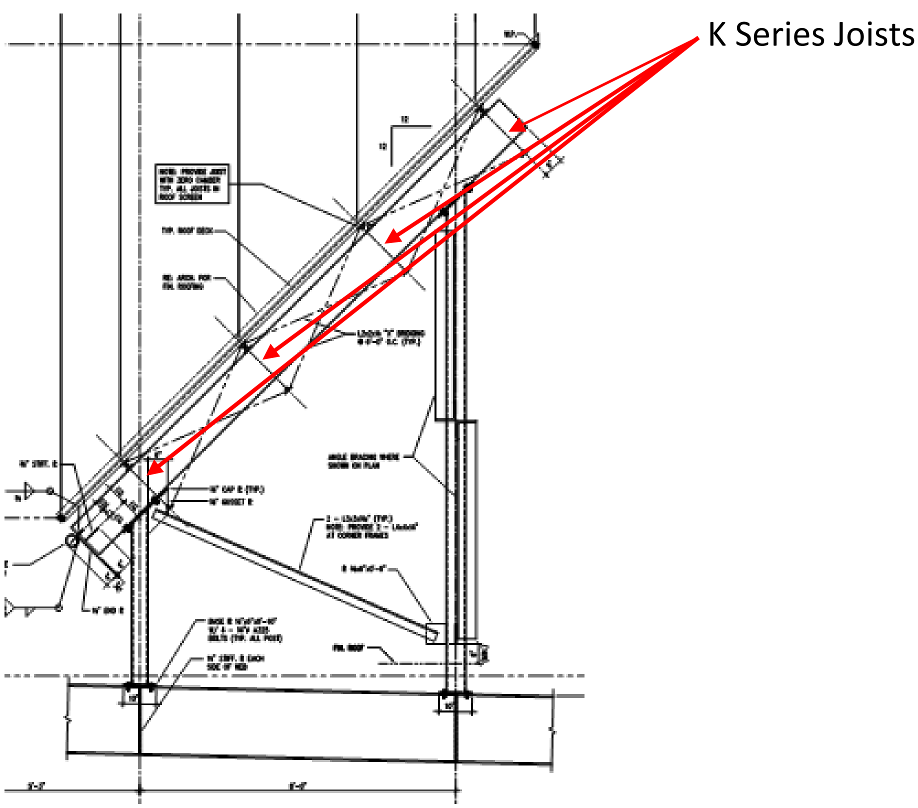


Figure 7: Roof Screen Support

Lateral System

Lateral loads on the building are supported by four concentrically braced frames. Three of the frames are located in the north-south direction to support higher wind loads from the broad side of the building, and one frame is located in the east-west direction. The three frames in the north-south direction are located on the column lines, adjacent to stairwells one and two. The other is located to the left of stairwell three. In the east-west direction the frame is located between columns B6 and B7. All frames are braced with hollow structural steel ranging in size 8 x 8 x ¼ at the first floor to 4 x 4 x ¼ on the fifth floor. Figure 8 shows the elevations of each braced frame, and Figure 9 shows the location of each frame.

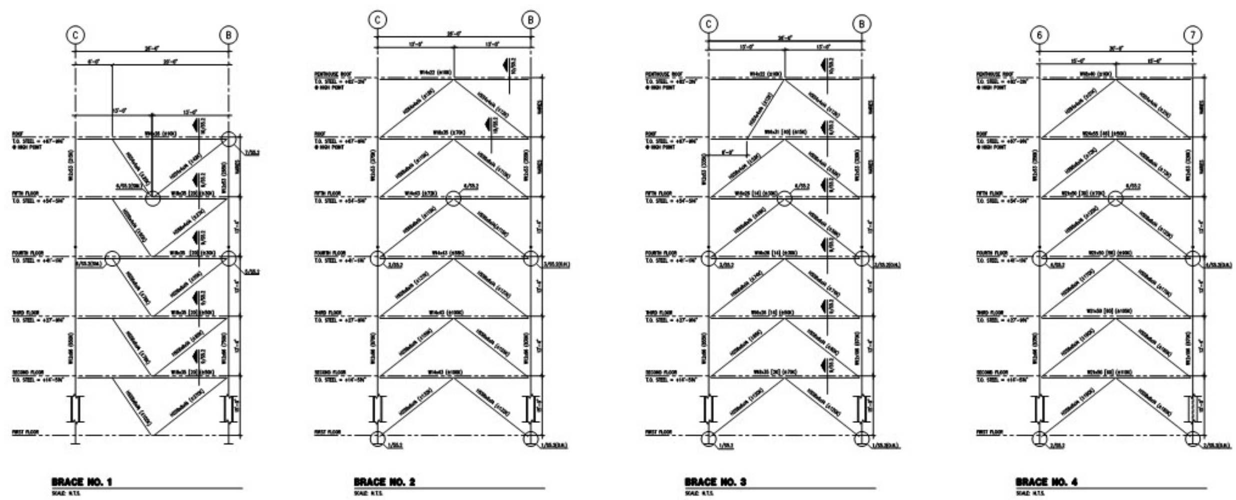


Figure 8: Braced Frame Elevations

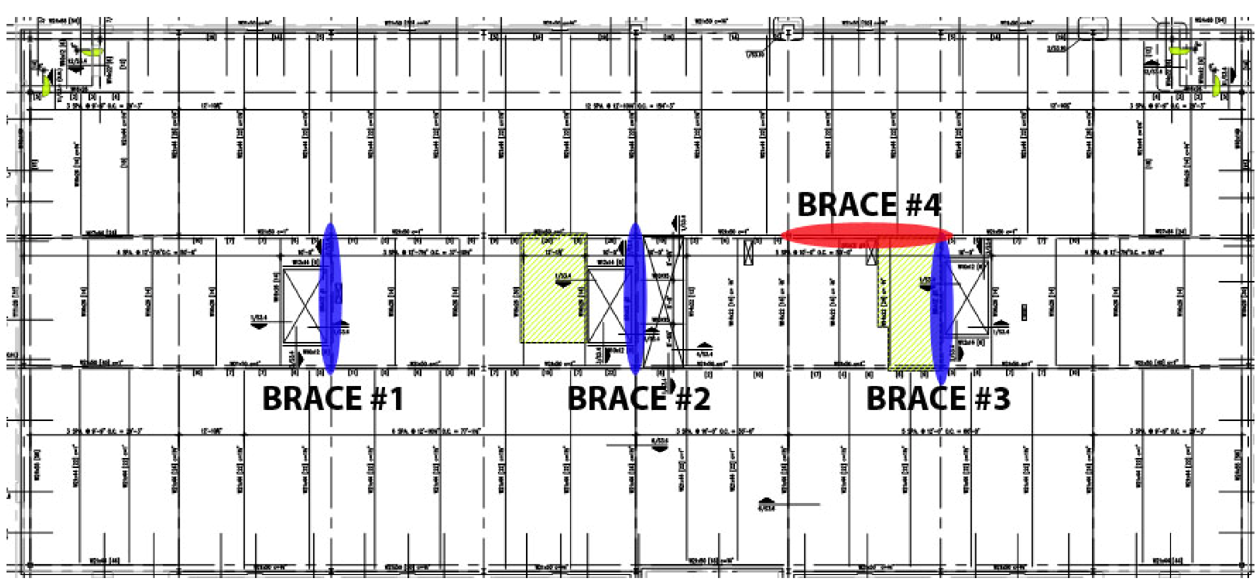


Figure 9: Location of Braced Frames

Materials

Concrete – All concrete shall have natural sand fine aggregate, and Type I Portland Cement conforming to ASTM C150. Concrete in the footings, pilasters, and slabs on grade shall be prepared with normal weight course aggregates conforming to ASTM C33. The concrete in the composite slabs shall have lightweight course aggregates conforming to ASTM C330, and a maximum unit weight of 115 pounds per cubic foot.

Compressive Strength

Footings	3000 psi
Pilasters	3000 psi
Slabs on Grade	4000 psi
Composite Slabs	3500 psi

Reinforcing Bars – Must conform to ASTM A615, grade 60.

Welded Wire Fabric – Must conform to ASTM A185.

Roof Deck – All Type B deck shall be 22 gage cold formed steel conforming to ASTM A653 SQ Grade 33. The deck shall be 1 – ½ inches deep and have a minimum section modulus of 0.186 inches cubed per foot of width.

Composite Steel Deck - Composite steel deck shall be 18 gage minimum cold-formed steel conforming to ASTM A611, Grade D and shall have a phosphatized and painted lower surface and a phosphatized only top surface. The deck shall be 3 inches deep and shall have a minimum section modulus of 0.803 inches cubed per foot of width.

Structural Steel

W Shapes – Shall conform to ASTM A992
Other Steel Shapes, Plates, Angles and Channels – Shall conform to ASTM A36
Steel Pipe – Shall conform to ASTM A53, Grade B
Steel Tubing – Shall conform to ASTM A500, Grade B
Anchor Bolts – Shall conform to ASTM F1554, Grade 36
Bolts – Shall meet or exceed the requirements of ASTM A325, Type N, X, or F

Concrete Masonry

Concrete masonry shall have a minimum compressive strength of 1500 PSI on the net cross sectional area at 28 days

Masonry Units – Shall be grade N, Type I light weight or medium weight hollow concrete units meeting fire rating requirements and conforming to the requirements of ASTM C90

Mortar – Shall conform to the requirements of ASTM C270, type M or S

Grout – Shall conform to ASTM C476

Codes

Building Code

Virginia USBC (IBC 2000)

Structural Steel

AISC Specification for Structural Steel Buildings

AISC Code of Standard Practice for Steel Buildings and Bridges

*Exception of paragraph 4.2.1 – Deletion of the following sentence: "This approval constitutes the owner's acceptance of all responsibility for the design adequacy of any connections designed by the fabricator as part of his preparation of these shop drawings."

AISC Manual of Steel Construction – Allowable Stress Design, 9th Addition

Steel Joist Institute Standard Specifications for Open Web Steel Joists

AISI Specification for the Design of Cold-Formed Steel Structural Members

Concrete

ACI Details and Detailing of Concrete Reinforcement, ACI 315

ACI Detailing Manual, ACI SP-66

ACI Manual of Engineering and Placing Drawings for Reinforced Concrete Structures, ACI 315R

CRSI Manual of Standard Practice

Concrete Masonry

ACI Building Code Requirements for Concrete Masonry Construction, ACI 530

ACI Specifications for Masonry Structures, ACI 530.1

Design Loads

International Building Code 2000

American Society of Civil Engineers (ASCE), ASC- 7

Design References

Concrete Design

ACI 318-08

Reinforced Concrete, Mechanics & Design, 5th Edition, by Wight & MacGregor

PCA two-way post-tensioned example provided by Dr. Memari

PCI Industry Handbook

Steel Design

AISC Steel Construction Manual

Unified Design of Steel Structures, by Louis F. Geschwindner

Gravity Loads

Dead/Live Loads

Live Loads	
Area	**Design Load (psf)**
Office Space	100
Permanent Corridors	100
Lobbies, Stairs, and Assembly Areas	100
Mechanical Space	125
Light Storage (Mechanical Rooms)	125
Roof	30
Dead Loads	
MEP	5
Exterior Wall	15
Ballasted Single Ply Roof	11
Finishes/Partitions	20
3 1/4" Lightweight Concrete on 3" Metal Deck	60

Table 1: Design Gravity Loads

Alternative Framing Systems

Systems Considered

- Composite steel decking, beams, and girders (*existing*)
- Two-way flat slab
- Pre-stressed concrete plank on steel
- One-way post tensioned concrete slab

Because the Belmont Executive Center; Building A is an office building, large open spaces were desired to maximize tenant space. As a result there are two typical bay sizes in the framing plan. One is located in the outer bays, typically 39'-10¾" x 30', and the other is located in the middle bay which is typically 30' x 26'-2½". In the long direction of the building, the beams in the exterior bays are slightly skewed due to the difference in the exterior column and interior column lines. For this report these bays were not looked at, and for simplification purposes only the rectangular interior bays were considered. If a floor system poses as a feasible alternative, further investigation, analysis, and design will be conducted for these perimeter bays. A typical outer bay (bay 1) is highlighted in red, and a typical middle bay (bay 2) is highlighted in green, see Figure 10.

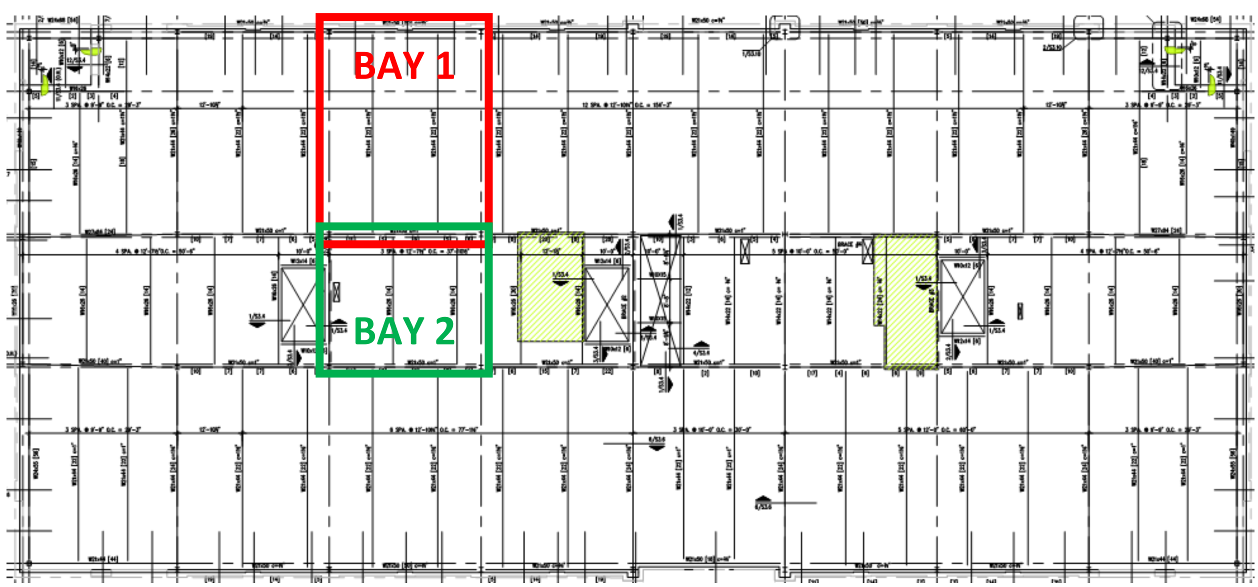


Figure 10: Typical Bay Locations

Composite Steel Decking, Beams, and Girders (*existing)*

The existing floor system in the Belmont Executive Center; Building A utilizes shear studs to bond the existing concrete slab and the beams together to increase the flexural capacity of the beams. In bay 1 the beams are W21x44's with 22 shear studs and a camber of 1½". Both the interior and exterior girders are W21x50 members with 50 shear studs and a camber of ¾". The interior girder has no shear studs, but has a camber of 1". Bay 2 is supported by W16x26 beams with 14 shear studs and no camber. Both interior girders are described above. The floor spanning between the beams is a total of 6¼", with 3¼" of light-weight concrete, and 3" composite metal deck. The slab is reinforced with 6x6-W1.4xW1.4 welded wire fabric. Beams and columns were found to provide adequate strength in Technical Report 1.

Advantages

Using shear studs allows the floor to span large distance while maintaining a relatively thin floor depth, and reduces the total weight of the floor.

Disadvantages

Although shear studs helps reduce the slab thickness, the studs are very labor intensive and costly to install. Also, the steel beams supporting the slab need to be fireproofed.

Two-Way Flat Slab

To determine if this system would be a viable alternative, the ACI direct design method was considered. There are 5 main limitations set forth by ACI 318-08 that must be met in order to use the direct design method. Due to the bay sizes in the Belmont Executive Center not all limitations were met. Table 2 summarizes the requirements, and although the third limitation is not met, the difference between the required span length and the actual is only 4”. Because the difference is minimal, engineering judgment suggests that his method can be used for the scope of this report. Therefore, the direct design method was used.

ACI Direct Design Limitations			
Limitation	**Existing**		**Section**
There shall be a minimum of three continuous spans in each direction.	There are 3 spans in one direction and 9 in the other.	YES	§13.6.1.1
Panels shall be rectangular, with a ratio of longer to shorter span center-to-center of supports within a panel not greater than 2.	The panel in bay 1 has a longer to shorter ratio of 1.33, and bay 2 has a ratio of 1.15.	YES	§13.6.1.2
Successive span lengths center-to-center of supports in each direction shall not differ by more than one-third the longer span.	The successive span lengths in the short direction of the building differ by 13’8”, and 1/3 the longer span is 13’4”	NO	§13.6.1.3
Offset of columns by a maximum of 10 percent of the span from either axis between centerlines of successive columns shall be permitted.	Each bay is rectangular; none of the columns are offset.	YES	§13.6.1.4
All loads shall be due to gravity only and uniformly distributed over an entire panel. The unfactored live load shall not exceed two times the unfactored dead load.	All loads are dead and live gravity loads.	YES	§13.6.1.5
All other limitations can be reviewed in §13.6.1 of ACI 318-08.			

Table 2: Direct Design Limitations

Two frames were looked at when designing the flat plate system. Figure 11 shows frame A highlighted in blue, which includes both bay 1 and bay 2, and frame B which includes only bay 2, highlighted in red. No floor openings were considered for the design of the slab in this report.

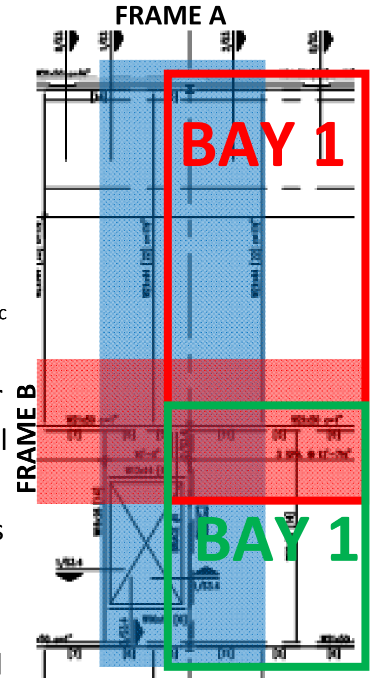


Figure 11: Flat Slab Frames

It was assumed the columns would be roughly 20"x20" with an f'_c of 5000 psi. The minimum floor thickness was determined from Table 9.5(c) from ACI 318-08 to limit deflections, and the exterior bay, (bay 1), required a larger slab thickness than bay 2. So, a trial size of 15.5" was chosen. It was determined that the shear strength provided by the floor slab, around the columns, was less than the ultimate shear. Therefore, drop panels were required at the columns to prevent punching shear. With drop panels, the slab thickness was further reduced to 14", and it was determined that 2" deep, 2'-6"x2'-6" drop panels could provide adequate shear strength. Alternatively the column could be increased by 2" on each side, or stud rails could be used to increase the shear strength around the exterior columns. If this system proves to be economical further costs investigation will be completed for these two options.

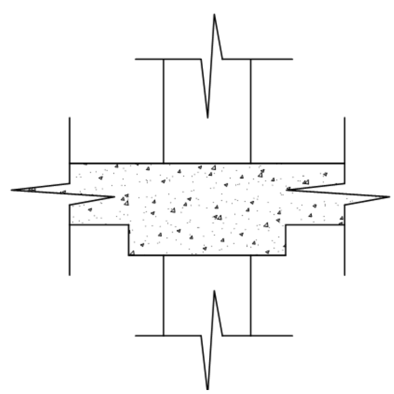


Figure 12: Drop Panel Design

Two checks were then performed; one was to determine if the amount of required reinforcement for the column strip in Frame A would not be excessive, and to see if a 20"x20" column would be capable of supporting the slab, superimposed dead loads, and live loads. Hand calculations were performed and the amount of reinforcement in Frame A was reasonable. PCA column was used and a 20"x20" column with 12 #9 bars could support the loads. See Table 4 for reinforcement design, and Figure 15 for the PCA column print out. Other detailed calculations can be found in the Appendix. Figure 12 shows what the typical drop panel design would be.

Advantages

Using a two-way flat slab floor system maintains the large spans required for the open office plan, and also reduces the depth of the floor system. With a floor thickness of 14" and a drop panel depth of 2” the total depth is equal to 16”. This depth is 11” less than the existing, but there is a significant increase in floor weight. Using a flat plate system also eliminates the need for fireproofing, because sufficient concrete cover is incorporated into the design. The formwork for the slab is very simple to construct, and upper floors can be formed and reinforced above lower floors, if sufficient shoring is provided.

Disadvantages

With a slab thickness of 14” there is a significant increase in the total weight of the floor, and the footings would have to be redesigned to accommodate the added weight. Also there would be a decrease in the rate of construction if concrete construction is used. With steel construction, the structure can be quickly erected, and floor construction can begin while the upper floor members are being set. With concrete construction, time has to be spent waiting for columns to cure before the slabs can be poured. Along with a drastic increase in self weight, the deep slab will also be more expensive to construct. Using a concrete floor system and concrete columns would require a different lateral system, because the existing system is concentrically braced steel frames. Also, the 20”x20” columns supporting the slabs are larger than the existing column cavities on the exterior perimeter. There would be a slight change in the exterior façade if this system is used.

Pre-Stressed Concrete Plank on Steel

Figure 13 shows how the beams and girders were spaced by the engineer to support the building loads. To accommodate the concrete plank the beams will frame into the columns as shown in orange. There are no intermediate beams between the outer beams, which creates a span of 30' for the concrete planks. On Figure 13 the red arrows show which way the planks will span. Similar changes were made to the middle bay, and the line colors signify the same notations as stated earlier. The PCI Industry Handbook was referenced to determine if the plank could support the service gravity loads.

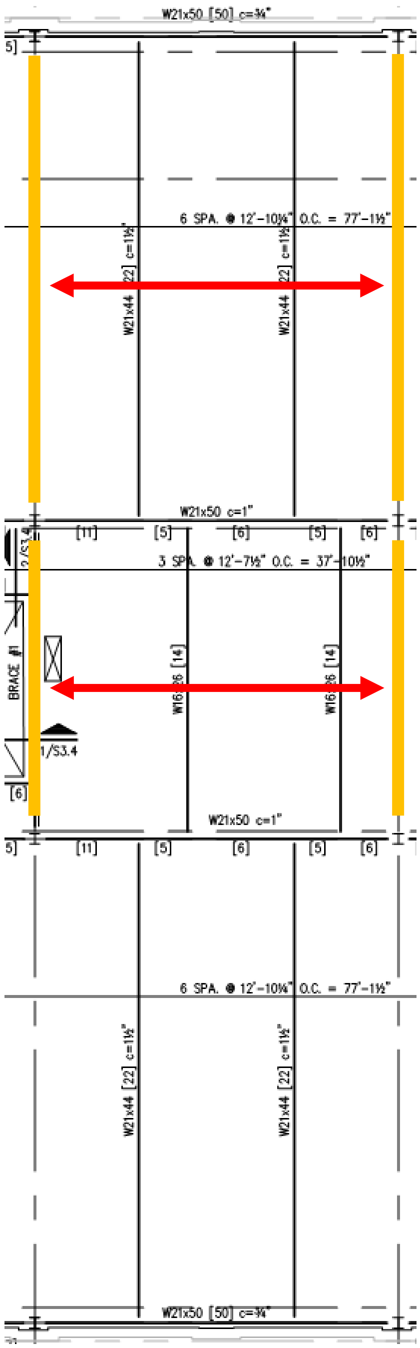


Figure 13: Existing Beam and Girder Layout

Steel beams and girders were sized using the AISC Steel Construction Manual to determine the depth of the floor system. Live load reduction factors were used in accordance to the IBC 2006, where applicable. Total deflection was limited to L/240, and live load deflections were limited to L/360. Deflections caused by construction loads were not considered because the decking is pre-cast, and there is no weight from wet concrete. The most economical members were chosen from the AISC Manual. Detailed calculations can be found in the Appendix.

Design of Bay 1

The total service load on each bay was determined to be 125 psf. For the office space a live load of 100 psf was used, and a superimposed dead load of 25 psf was used to account for MEP and partitions. The tables in the PCI handbook include a superimposed dead load of 10 psf for planks with no topping; therefore a service load of 115 psf was used when referencing the tables. From the PCI Handbook, (see Table 6 in the Appendix), it was determined that an 8", 78-S hollow core plank with no topping, and 1.1" of camber, can support a superimposed service load of 126 psf. This maximum load is larger than the load on the building, and therefore the plank will work.

The selection of girders in bay 1 was governed by deflection criteria. A smaller girder was originally selected, because it was capable of supporting the factored moment. However, when the total load deflection was checked it exceeded allowable limit. Therefore, a W33x118 was selected because it had adequate section properties to prevent the girder from excessively deflecting.

Design of Bay 2

The same hollow core plank was again chosen to support the gravity loads. There is no difference between the dead and live service loads between bays and therefore no extra calculations were needed.

Like the girders in bay 1, deflection was again the controlling criteria, and the most economical member was determined to be a W24x62.

Theoretically no load is supported by the beams spanning between the columns in the long direction of the building. Therefore, they were not designed in this report. Further investigation into the constructability, added costs, lateral system selection, and need for column support may require the design of these beams. If need be these beams will be designed at a later time.

The largest shape depth was governed by the W33x118 girders in bay 1, which have a depth of 32.9". Adding this depth to the thickness of the concrete plank brings the total floor system depth to 40.9".

Advantages

One of the biggest advantages of pre-stressed hollow core planks is their performance during a fire. Fireproofing does not have to be applied to the underside of the floor systems, but the supporting steel beams and girders do have to be fireproofed. The existing floor system has exposed steel beams that were fireproofed as well as the underside of the slab. Material and labor fireproofing costs are saved by using a concrete plank on steel system. Using hollow core planks also eliminates the need for formwork, which reduces labor and material costs. By not having to construct formwork time can also be saved. Also, because steel framing is still used the concentrically braced steel frames could still be used as the lateral system. Construction can proceed quickly because the planks are simply set in place with a crane and later grouted.

Disadvantages

Using concrete plank on steel beams also has several disadvantages. The existing floor system is very efficient and has a minimal floor depth. Calculations showed that a plank on steel floor system would result in a larger floor depth than the existing floor depth of 27.1". Between the floor systems there is a difference of roughly 14", which would have an impact on the routing of duct work and MEP equipment. Although a thin floor system was not the controlling factor in the design of the building, increasing the depth by 14” will have a detrimental impact on space restrictions and is not desirable. The building height is limited to 75’, which would make it difficult to increase the floor to floor height to maintain the existing clear floor to ceiling distance. Constructing a building with concrete plank also creates constructability issues. It is very difficult to maneuver the planks into position between the erected steel members. Also, each plank is manufactured in 4’ wide sections. The existing bay 1 size is 39’ 10¾”, it could be increased to 40’ so 10 planks could be evenly placed in the bay. Similarly, bay 2 could be decreased from 26’ 2½” to 26’, and 6 planks would evenly fit in the bay. These slight changes will cause very little change to the total building dimensions, but may result in problems in other parts of the building. Compared to the existing system, hollow core planks on steel is a lighter system. Therefore the foundations would not have to be altered.

Two-Way Post-tensioned Slab

For the design of the the two-way post-tensioned slab, the interior frame along column line 3 was only considered. This was done because the largest moments would exist in bay 1, in the short direction of the building. Figure 14 shows the selected frame circled in blue, and bay 1 and bay 2 are shown for reference. Design of the system was completed following the PCA Two-Way Post-Tensioned Design provided by Dr. Memari, and as before no openings were considered in the slab.

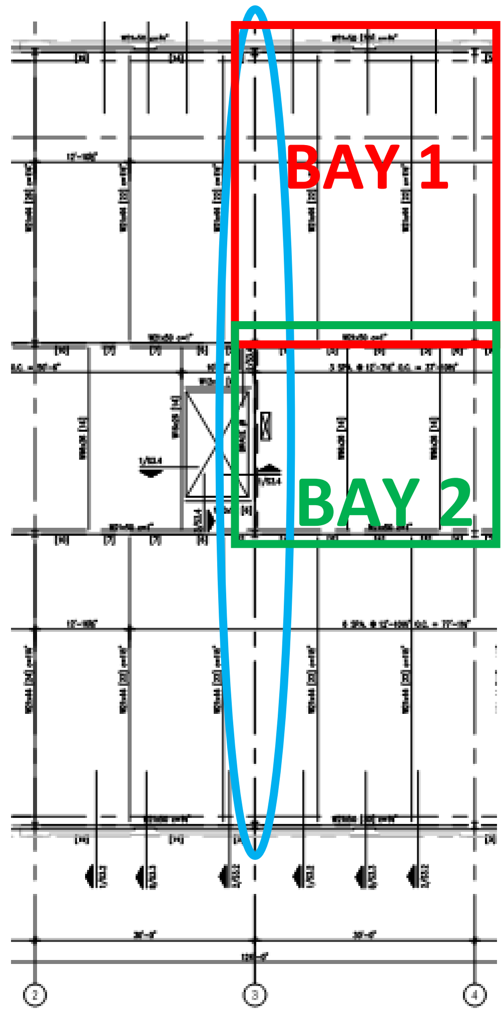


Figure 14: PT Frame Selection

As stated before, bay 1 controlled the design of the system. To begin the designm a preliminary slab thickness of 11" was calculated. Live load reductions were performed for each bay, and the post-tension tendons were selected to carry 80% of the selfweight dead load. 50 tendons, with a P_u of 26.6k were required to support the set percentage of the dead load. However, the number of tendons was limited to 44, as to not exceed the max precompression stress of 300 psi. Maximizing the drape in the interior bay created a tendond force which counteracted the dead load of the slab, and was essentially "ripping" the concrete out of bay 2. To counteract the force the drape was adjusted to 3.5" to reduce the upward force created by the tendons.

Dead load, live load, and balancing moments were then calculated using SAP2000. Table 7 is an abridged table from the SAP outupt, and it displays all the critical moments for each span in the frame; including moments between supports and at the supports. Figure 16 displays the moment diagrams from SAP.

In order to keep the top and bottom slab stresses within the allowable limits set by ACI 318-08, either the floor depth, or the concrete strength had to be increased. It was determined that a concrete strength of 14,000 psi was required for the stresses to be acceptable. Because this is not economical the floor depth was then increased. With a floor depth of 12" and a concrete strength of 6,000 psi the slab stresses fell within the allowable limits. Punching shear around the interior and exterior columns was then considered, and no drop panels were required at the exterior columns. However, to provide adequate shear strength at the interior columns 2'-6"x2'-6"x2" drop panels were required. As with the two-way flat slab, either the column size could be increased or stud rails could be used to eliminate the need for the panels. If this system is used, detailed cost investigations will be completed. The amound of required reinforcement was not determined for this system. Because it is similar in size to the two-way slab, roughly the same amount of reinforcement will be requierd. Detailed calculations for the post-tensioned design can be found in the Appendix.

Advantages

Like the two-way flat slab floor system, the floor depth would be smaller than the existing composite deck on steel. Using two-way post tension would require a slab thickness of 14". Out of the four system analyzed this is the thinnest floor system. Also, the large open bay sizes are maintained, and as with most concrete construction, there would be no need for fireproofing, because clear cover requirements for fire resistance were met in the design of the slab.

Disadvantages

Although large open bay sizes are maintained, and the floor depth is decreased, there would again be a significant increase in the selfweight of the building. The slab is not as thick as the two-way system, but is only 2" thinner. Therefore the foundation system would have to be redesigned to support the large increase in load. Concrete moment frames, or a different lateral system would have to be designed for the building if this system is used. There would also be a decrease in construction speed because of the need to wait for the concrete to reach the required strength. Also, the exterior column size may have to modified to accommodate the larger columns.

Comparison Summary

All cost estimates were performed using RS Means Costworks. Takeoffs were were only completed on bay 1 for comparison. System weights were calculated by determining the percentage per member for bay 1, divided by the square footage of bay 1.

Floor System Comparison – Bay 1				
Criteria	**Compsite Steel (*existing*)**	**Two-Way Flat Slab**	**Two-Way Post-Tensioned Slab**	**Pre-stressed Hollow Core Plank on Steel**
Self Weight (psf)	68	187	162	58
Floor Depth (in)	21.7	16*	15¥	40.9
Constructability	Medium	Medium	Hard	Medium
Fireproofing	Yes	No	No	Yes
Architectural Impact	-	Yes	Yes	Yes
Foundation Impact	-	Major	Major	No
Lateral System Impact	-	Yes	Yes	No
Vibration	Good	Best	Good	Good
Cost ($\$/ft^2$)	18.10	16.68	22.03	15.08
Possible Alternative	-	Yes	No	Yes
Additional Study	-	Yes	No	Yes

*14” slab with 2” interior drop panels
¥ 12” slab with 3” interior drop panels

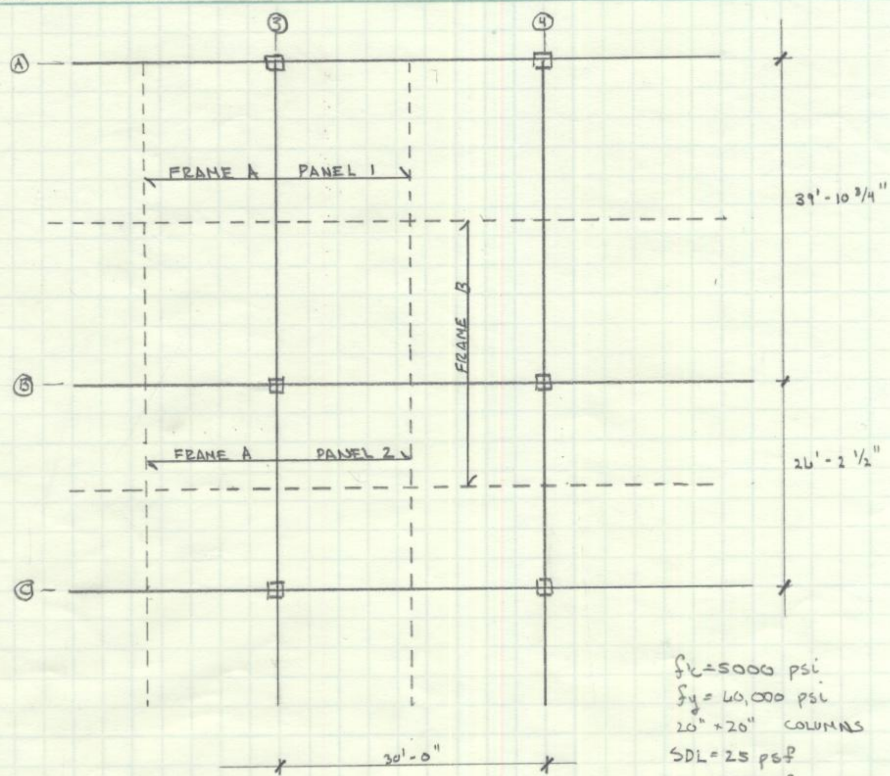
Table 3: Pro/Con Comparison

Conclusion

Although none of the systems are clearly better than the others, futher study will be conducted for the two-way flat slab system and the pre-stressed hollow core plank on steel. The largest deciding factor for the two-way flat slab system was the decrease in floor depth. Both the two-way flat slab and the two-way post-tensioned slab have similar slab thicknesses but there is much more labor costs involved with a post-tensioned system. The pre-stressed hollow core plank on steel is lighter than the existing system, has relatively low costs associated with it, and would have the least impact on the existing structure. However, the floor thickness is drastically increased, and with the height restrictions limiting the building to only 7’6” more than the existing height; it may be difficult to modifiy the story height and maintain the 9’ clear space between the floor and ceiling, but it is possible.

Appendix: Two-Way Flat Slab

TWO-WAY SLAB (1)



$f'_c = 5000$ psi
$f_y = 60{,}000$ psi
20" × 20" COLUMNS
SDL = 25 psf
LL = 100 psf

DETERMINE THICKNESS

PANEL A-B-3-4

MIN. SLAB THICKNESS: $\frac{l_n}{30} = \frac{(39.9 \times 12) - 20}{30} = 15.3\text{ in} \Rightarrow 15.5\text{ in} \leftarrow$ GOVERNS

PANEL B-C-3-4

MIN: $\frac{l_n}{33} = \frac{(30 \times 12) - 20}{33} = 10.3\text{ in} \Rightarrow 10.5\text{ in}$

$w_u = 1.2\left(\frac{15.5}{12} \times 150 + 25\right) + 1.6(100) = 423$ psf

TWO-WAY SLAB (2)

CHECK PUNCHING SHEAR @ COLUMN A3 & B3

COLUMN A-3

$V_u = .423\left[19.95 \times 30 - 1.67 \times 1.67\right] = 252^k$

ASSUME #5 BARS

$d = 15.5 - 0.75 - 0.625 = 14.1 \text{ in}$

$b_o = (20 + 14.1)3 = 102 \text{ in}$

$\frac{b_1}{b_2} = 1.0$ ACI 11-35 GOVERNS

$V_c = 4 \cdot \sqrt{f'_c} \cdot b_o \cdot d = 4\sqrt{5000} \cdot 102 \cdot 14.1 = 407^k$

$\phi V_c = 0.75(407) = 305^k > V_u$ ∴ DO NOT NEED DROP PANELS @ EXT. COLS.

COLUMN B-3

$V_u = .423\left[33.1 \times 30 - 1.67 \times 1.67\right] = 419^k$

ASSUME #5 BARS

$d = 14.1 \text{ in}$

$b_o = (20 + 14.1) \cdot 4 = 136 \text{ in}$

$V_c = 4 \cdot \sqrt{5000} \cdot 136 \cdot 14.1 = 542^k$

$\phi V_c = 0.75(542) = 407^k < 419^k$ ∴ NEED DROP PANELS @ INT. COLS. USE DROP PANELS @ ALL COLS.

$t_{SLAB_{MIN}} = \frac{39.9 \times 12 - 26}{33} = 13.9 \text{ in} \longrightarrow 14 \text{ in}$ GOVERNS

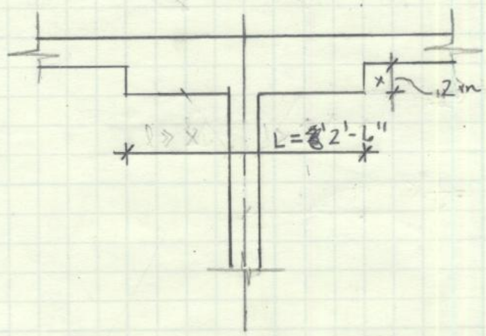
OR

$\frac{30 \times 12 - 26}{36} = 9.4 \text{ in}$

TWO-WAY SLAB (3)

SIZE DROP PANEL

ACI 13.2.6.



EXTEND PANEL 5 in FROM FACE OF COL.

2'-6" SQUARE

RECHECK PUNCHING SHEAR AFTER DETERMINING REINF.

TOTAL FACTORED MOMENT

$$M_o = \frac{q_u \cdot l_2 \cdot l_n^2}{8}$$

$$q_u = 1.2\left(\frac{14}{12} \times 150 + 25\right) + 1.6(100) = 400 \text{ psf}$$

FRAME A: PANEL 1

$$M_o = \frac{(0.400)(30)(39.9 - 1.67)^2}{8} = 2192\ ^{'k}$$

PANEL 2

$$M_o = \frac{(0.400)(30)(26.2 - 1.67)^2}{8} = 903\ ^{'k}$$

FRAME B:

$$M_o = \frac{(0.400)(33.1)(30 - 1.67)^2}{8} = 1328\ ^{'k}$$

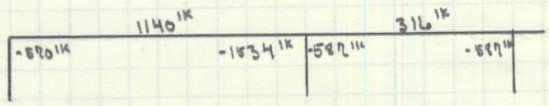
TWO-WAY SLAB (4)

DISTRIBUTION OF MOMENTS
NO EDGE BEAMS OR INT BEAMS

FRAME A:

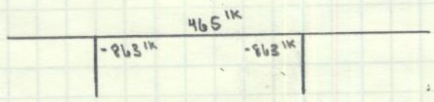
$M^-_{EXT} = -0.26 \cdot M_o$ $\quad M^+ = 0.52\ M_o$ $\quad M^-_{INT} = -0.70\ M_o$ — EXT

$M^+ = 0.35\ M_o$ $\quad M^- = -0.65\ M_o$ — INT



FRAME B:

$M^+ = 0.35\ M_o$ $\quad M^- = -0.65\ M_o$



DISTRIBUTION OF MOMENTS IN CS, BEAM, MS

FRAME A: PANEL 1

a) NEG. MOM @ EXT. FACE

$\frac{l_2}{l_1} = \frac{30}{39.9} = 0.752$

$\beta_t = 0$
$\alpha = 0$ $\longrightarrow$ 100%

-570 ⟨ 100% TO CS = -570
0% TO MS

b.) NEG. MOM @ INT. FACE

-1534 ⟨ 75% TO CS = -1151
25% TO MS = -383

c.) POSITIVE MOMENTS

1140 ⟨ 60% TO CS = 684
40% TO MS = 456

TWO-WAY SLAB (5)

DIST. OF MOM.

FRAME A: PANEL 2

a) NEG. MOM. @ INT. FACES

-597 < 75% TO CS = -440
25% TO MS = -147

b) POS. MOM

316 < 60% TO CS = 190
40% TO MS = 126

FRAME B:

a) NEG. MOM @ INT. FACES

-863 < 75% TO CS = -647
25% TO MS = -216

b.) POS. MOM.

465 < 60% TO CS = 279
40% TO MS = 186

0.00186

TWO-WAY SLAB (6)

SUMMARY OF MOMENTS

FRAME A: TOTAL WIDTH = 30' CS WIDTH = 15' MS WIDTH = 15'

TOTAL MOM.	-570	1140	-1534	-587	316	-587
CS MOM	-570	684	-1151	-440	190	-440
MS MOM	0	456	-383	-147	126	-147

FRAME B: TOTAL WIDTH = 30' CS WIDTH = 15' MS WIDTH = 15'

TOTAL MOM.	-863	465	-863
CS MOM.	-647	279	-647
MS MOM	-216	186	-216

TWO-WAY SLAB ~~t~~(7)

PUNCHING SHEAR @ DROP PANELS

DEPTH OF DROP PANEL = 2 in

$d_1 = 16 - 0.75 - \frac{0.625}{2} = 14.9 \text{ in}$

$b_o = (20 + 14.9)(4) = 140 \text{ in}$

$\phi V_c = 0.75\,(4)\sqrt{5000}\,(14.9)(140) = 443^k > V_u$ OK

USE 14 in SLAB W/ TWO IN DROP PANEL

A CHECK TO SEE IF REQ'D REINF IS REASONABLE FOR CS

COLUMN IS CHECKED TO SEE IF A 20x20 in COLUMN COULD SUPPORT LOADS

14 in SLAB

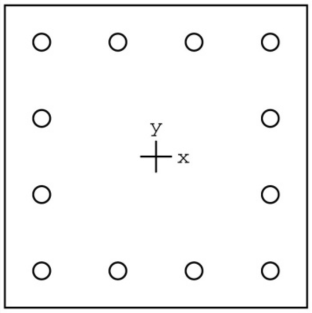
WITH 2'x6" x 2" DROP PANELS @ INT. COLS.

Reinforcement Design for Frame A Column Strip						
Item	Description	Exterior Span			Interior Span	
		$-M_{ext}$	$+M$	$-M_{int}$	$-M$	$+M$
1	M_u	-570	684	-1151	-440	190
2	CS Width	180	180	180	180	180
3	Effective Depth, d	13.2	13.2	13.2	13.2	13.2
4	M_ux12/b	-38.0	45.6	-76.7	-29.3	12.7
5	M_n	-633	760	-1279	-489	211
6	R	242	291	489	187	81
7	$\rho_{required}$	0.00463	0.00504	0.00868	0.00319	0.00152
8	$A_{s,required}$	11	12	21	8	4
9	$A_{s,min}$	5	5	5	5	5
10	N	11	12	21	8	5
11	N_{min}	6	6	6	6	6

Table 4: Reinforcement Design, Frame A CS

Floor	Tributary Area (ft^2)	Dead Load (psf)	Live Load (psf)	Influence Area (ft^2)	Reduction Factor	Live Load (Kips)	Dead Load (kips)	Load Combination	Load at Floor (Kips)	Accumulated Load (kips)
Roof	992	36	20	3966	0.49	9.7	35.7	1.2D + 0.5L_r	47.7	27
5	992	200	100	7932	0.42	41.5	198.4	1.2D + 1.6L	287.9	336
4	992	200	100	11898	0.40	39.7	198.4	1.2D + 1.6L	285.7	621
3	992	200	100	15864	0.40	39.7	198.4	1.2D + 1.6L	285.7	907
2	992	200	100	19830	0.40	39.7	198.4	1.2D + 1.6L	285.7	1193

Table 5: Column Spot Check



20 x 20 in

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Design

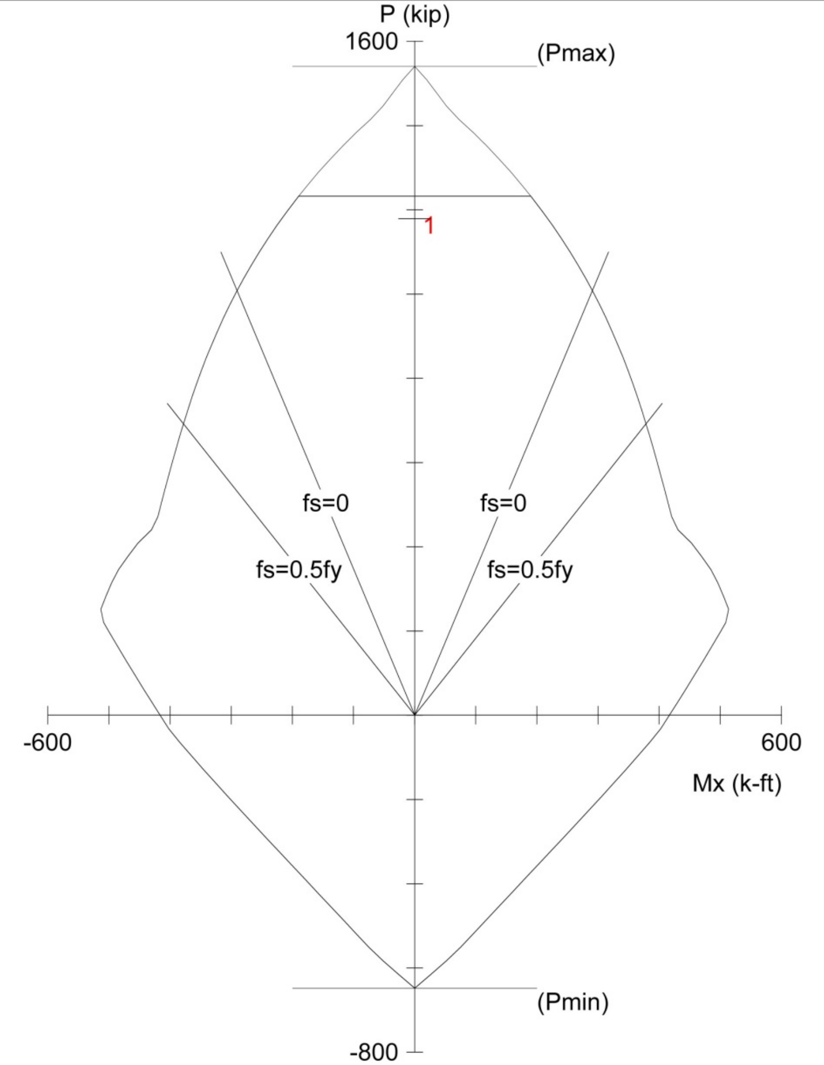
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/21/09

Time: 10:32:22



pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D

File: untitled.col

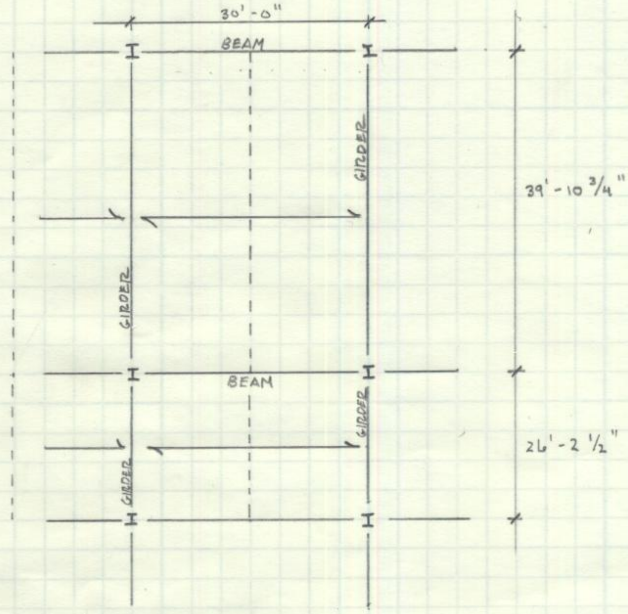
Project:

Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 400 in^2	12 #9 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 12.00 in^2	Rho = 3.00%
fc = 4.25 ksi	fc = 4.25 ksi	Xo = 0.00 in	Ix = 13333.3 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 13333.3 in^4
Beta1 = 0.8		Clear spacing = 3.91 in	Clear cover = 1.88 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

Figure 15: PCA Column Check

Appendix: Pre-Stressed Concrete Plank on Steel

PLANK ON STEEL DESIGN (1) NICK ZIEGLER



BAY 1

DL = 25 - MEP & PARTITIONS
+10 - PCI INCLUDES 15 psf SD IN TABLES
LL = 115 psf
LL = 100 psf

TOTAL SERVICE LOAD = 115 psf

DESIGN OF GIRDERS

DL = 25 + 56* = 81 psf * DL OF PLANK W/ NO TOPPING
LL = 100 psf

LIVE LOAD REDUCTION

$$L = L_o \cdot \left[0.25 + \frac{15}{\sqrt{A_I}}\right] \quad \text{NOT LESS THAN } 0.50\, L_o$$

$$A_I = 60(39.9) = 2394 \text{ ft}^2$$

$$L = L_o\left[0.25 + \frac{15}{\sqrt{2394}}\right] = L_o \cdot 0.56$$

FACTORED LOADS

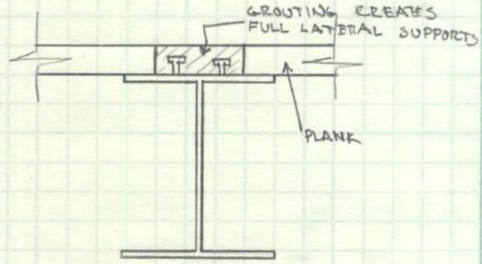
$$1.2(81) + 1.6(56) = 187 \text{ psf}$$

$$\omega_u = \frac{187(30)}{1000} = 5.61 \text{ k/ft}$$

$$M_u = \frac{\omega \cdot l^2}{8} = \frac{5.61(39.9)^2}{8} = 1116^{\text{1K}}$$

PLANK ON STEEL DESIGN (2) NICK ZIEGLER

CONCRETE PLANK PROVIDES
FULL LATERAL SUPPORT



TRY W30 x 108

$\phi M_p = 1300^{1K}$ $\quad I = 4470 \text{ in}^4$

LIMIT DEFLECTIONS TO

$l/360$ - LIVE

$l/240$ - TOTAL

NO NEED FOR CONST. DEFL.
B/C NO WET CONC.

TOTAL LOAD DEFL

$$l/240 = \frac{39.9(12)}{240} = 2.0 \text{ in}$$

$$\Delta_{TL} = \frac{5 \cdot \omega \cdot l^4}{384 \cdot E \cdot I} = \frac{5\left(\frac{181 \times 30}{1000}\right)(39.9)^4 \cdot 1728}{384(29000)(4470)} = 2.38 \text{ in} \therefore \text{NO GOOD}$$

$$I_{REQ'D} = \frac{5(5.43)(39.9)^4(1728)}{384(29000)(2.0)} = 5339 \text{ in}^4$$

TRY W33 x 118

$I = 5900$ $\quad \phi M_p = 1560^{1K}$

LIVE LOAD DEFL.

$$l/360 = \frac{39.9(12)}{360} = 1.33 \text{ in}$$

$$I_{REQ'D_{LL}} = \frac{5\left(\frac{100 \times 30}{1000}\right)(39.9)^4(1728)}{384(29000)(1.33)} = 4436 \text{ in}^4 < 5900 \text{ in}^4 \text{ OK}$$

A W 33 x 118 WILL WORK

$d = 32.9 \text{ in}$

PLANK ON STEEL DESIGN (3)

BAY 2

DESIGN OF GIRDERS

DL = 81 psf

LL = 100 psf

LIVE LOAD REDUCTION

$L_o = L\left[0.25 + \frac{15}{\sqrt{796}}\right] = L \cdot 0.785$

FACTORED LOADS

$1.2(81) + 1.6(78.5) = 223 \; psf$

$\omega_u = 6.69 \; k/ft$

$M_u = \frac{6.61(26.2)^2}{8} = 567^{1k}$

TRY W24 x 62

$\phi M_p = 574^{1k}$ $I = 1550 \; in^4$

TOTAL LOAD DEFLECTION

$\ell/240 = \frac{26.2(12)}{240} = 1.31 \; in$

$\Delta_{TL} = \frac{5(5.43)(26.2)^4(1728)}{384(29000)(1550)} = 1.29 \; in < 1.31 \; in \therefore$ OK GOOD

LIVE LOAD DEFLECTION

$\ell/360 = \frac{26.2(12)}{360} = 0.873$

$I_{REQ'D} = \frac{5(3.0)(26.2)^4(1728)}{384(29000)(0.873)} = 1266 \; in^4 < 1550 \; in^4 \therefore$ OK

A W24 x 62 WILL WORK

$d = 23.7 \; in$

Strand Pattern Designation
76-S

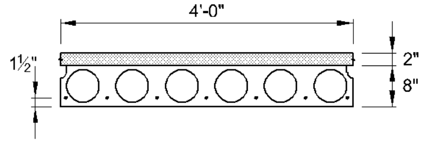
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
458 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 8"
Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi

Section Properties

	Untopped	Topped
A =	215 in.^{2}	311 in.^{2}
I =	1,666 in.^{4}	3,071 in.^{4}
y_b =	4.00 in.	5.29 in.
y_t =	4.00 in.	4.71 in.
S_b =	417 in.^{3}	581 in.^{3}
S_t =	417 in.^{3}	652 in.^{3}
wt =	224 plf	324 plf
DL =	56 psf	81 psf
V/S =	1.92 in.	

4HC8

Table of safe superimposed service load (psf) and cambers (in.) **No Topping**

Strand Designation Code	Span, ft																													
	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	458	415	378	346	311	269	234	204	179	158	140	124	110	98	87	77	69	61	54	48	43	38	33	29						
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	0.0	–0.1	–0.2	–0.3	–0.5	–0.6						
	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.4						
76-S	470	424	387	355	326	303	276	242	213	188	167	149	133	119	106	95	86	77	69	62	55	50	44	39	35	31	26			
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9			
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.6	–0.8	–1.1	–1.4	–1.7	–2.0			
58-S	464	421	384	352	323	300	280	260	244	229	211	194	177	160	144	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	–0.4	–0.3	–0.5	–0.7	–0.9
	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4
68-S	476	430	393	361	332	309	286	269	253	235	223	209	200	180	165	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	–0.3
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	–0.2	–0.5	–0.8	–1.1	–1.5
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	–0.3	–0.7

4HC8 + 2

Table of safe superimposed service load (psf) and cambers (in.) **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft																											
	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	489	445	394	340	294	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	–0.0	–0.1	–0.2	–0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	–1.4								
76-S	498	457	420	387	347	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	–0.0	–0.1	–0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.4							
58-S	492	451	414	384	357	333	310	293	274	245	219	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	–0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.9	–1.2	–1.5	–1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.7	–2.2

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

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Table 6: PCI Hollow-Core Design Table

Appendix: Two-Way Post-Tensioned Slab

PT DESIGN (1)

LOADS

SD = 25 psf
LL = 100 psf

MATERIALS

CONCRETE: NWC 150 psf
f'_c = 5000 psi
f'_{ci} = 3000 psi

REBAR: f_y = 60,000 psi
PT: UNBONDED TENDONS
½"ϕ, 7-WIRE STRANDS, A = 0.153 in²
f_{pu} = 270 ksi
ESTIMATED PRESTRESS LOSSES = 15 ksi (ACI 18.6)
$f_{se} = 0.7(270) - 15 = 174$ ksi (ACI 18.5.1)
$P_{eff} = A \cdot f_{se} = (0.153)(174) = 26.6$ kips/tendon

DETERMINE PRELIMINARY SLAB THICKNESS

START WITH L/h = 45
LONGEST SPAN = 39.9 ft
$h = \frac{(39.9)(12)}{45} = 10.64 \text{ in} \longrightarrow 11 \text{ in}$

LOADING

DL = $\left(\frac{11}{12}\right)(150)$ = 138 psf
SD = 25 psf
LL = 100 psf

LIVE LOAD REDUCTION
BAY 1: A_T = 1197 ft²
K_{LL} = 1
LL = 0.68 L_o = 68 psf
BAY 2: A_T = 786 ft²
K_{LL} = 1
LL = 0.79 L_o = 79 psf

PT DESIGN (2)

DESIGN OF FRAME A

CALCULATE SECTION PROPERTIES

$A = b \cdot h = 360(11) = 3960 \text{ in}^2$

$S = \frac{bh^2}{6} = \frac{300(11)^2}{6} = 6050 \text{ in}^3$

SET DESIGN PARAMETERS

ALLOWABLE STRESSES: CLASS U (ACI 18.3.3)

AT TIME OF JACKING (ACI 18.4.1)

$f'_{ci} = 3000$ psi

COMPRESSION $= 0.60 f'_{ci} = 0.6(3000) = 1800$ psi

TENSION $= 3\sqrt{f'_c} = 3\sqrt{3000} = 164$ psi

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

$f'_c = 5000$ psi

COMPRESSION $= 0.45 \cdot f'_c = 0.45(5000) = 2{,}250$ psi

TENSION $= 6 \cdot \sqrt{f'_c} = 6 \cdot \sqrt{5000} = 424$ psi

AVERAGE PRECOMPRESSION LIMITS

$P/A = 125$ psi. MIN (ACI 18.2.4)

$= 300$ psi MAX.

TARGET LOAD BALANCES

60% TO 80% OF DL (SELF WEIGHT)

$0.75\, w_{DL} = 0.75(138) = 104$ psf

COVER REQUIREMENTS (2 HR FIRE RATING, ASSUME CARBONATE AG.)

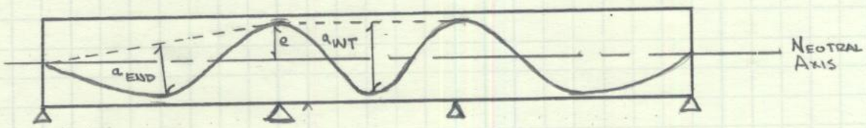
RESTRAINED SLABS = 3/4" BOT.

UNRESTRAINED SLABS = 1 1/2" BOT.

= 3/4" TOP

PT DESIGN (3)

TENDON PROFILE
HIGHEST @ INT. COLS
LOWEST @ MID SPAN



TENDON ORDINATE	TENDON LOCATION (MEASURE FROM BOT. SLAB)
EXT. SUPP. - ANCHOR	5.5"
INT. SUPP. - TOP	10"
INT. SPAN - BOT.	1"
END. SPAN - BOT	1.75"

$a_{INT} = 10 - 1.0 = 9.0"$
$a_{END} = (5.5 + 10)/2 - 1.75 = 6$ in
ECCENTRICITY, e, IS THE DISTANCE FROM THE CENTER TO TENDON TO THE NEUTRAL AXIS; VARIES

PT DESIGN (4)

PRESTRESS FORCE REQUIRED TO BALANCE 80% OF SELFWEIGHT DL

THE EXTERIOR SPAN (BAY 1) WILL GOVERN THE MAXIMUM POST-TENSIONING FORCE

$$w_b = \frac{0.80\,(138)(30)}{1000} = 3.31\ ^{k}/_{ft}$$

FORCE NEEDED =

$$P = \frac{w_b L^2}{8\,a_{END}} = \frac{3.31\,(39.9)^2}{8\left(\frac{6}{12}\right)} = 1317^{k}$$

CHECK PRECOMPRESSION ALLOWANCE

DETERMINE # OF TENDONS

$$\# = \frac{1317}{26.6} = 49.5 - \text{MAX OF } 44 \text{ TENDONS}$$ → IF LARGER THAN 44 TENDONS THE PRECOMPRESSION STRESS WILL EXCEED 300 psi. THIS ALSO LIMITS THE PERCENTAGE OF DL THAT CAN BE CARRIED BY THE TENDONS

ACTUAL FORCE

$$P = 44(26.6) = 1170^{k}$$

ADJUST BALANCED LOAD

$$w_b = \frac{1170}{1317}(3.31) = 2.94\ ^{k}/_{ft}$$

ACTUAL PRECOMPRESSION STRESS

$$\frac{P_{ACT}}{A} = \frac{1170(1000)}{3960} = 295\,psi \quad < 300\ psi_{MAX}\ \text{OK} \quad > 125\ psi_{MIN}\ \text{OK}$$

CHECK INTERIOR SPAN

$$P = \frac{(2.94)(26.2)^2}{9\left(\frac{9}{12}\right)} = 336^{k} < 1170^{k} \quad \text{OK}$$

AMOUNT OF LOAD BALANCED IN INTERIOR BAY (BAY 2)

$$w_b = \frac{(1170)(8)\left(\frac{9}{12}\right)}{26.2^2} = 10.2\ ^{k}/_{ft}$$ ADJUST INTERIOR DRAPE TO SUPPORT ≈ 95% OF SELF DL

$$a = 12\left[\frac{3.93(26.2)^2}{1170(8)}\right] = 3.45\ in \rightarrow 3.5\ in\ \text{DRAPE}$$

$$w_b = \frac{(1170)(8)\left(\frac{3.5}{12}\right)}{26.2^2} = 3.97\ ^{k}/_{ft}$$

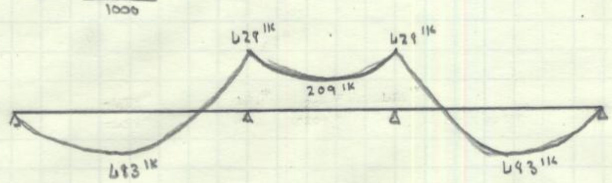
96% OF DL OK

EFFECTIVE PRESTRESS FORCE = 1170^{K}

PT DESIGN (5)

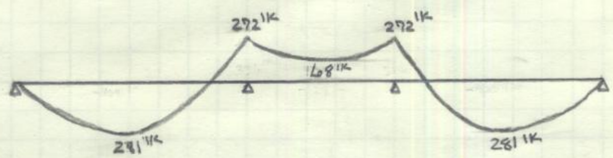
DEAD LOAD MOMENTS

$\omega_D = \frac{163(30)}{1000} = 4.89\ ^{K}/_{ft}$



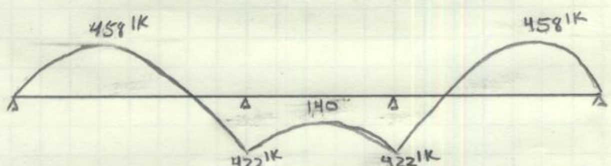
LIVE LOAD MOMENTS

$\omega_L = \frac{79(30)}{1000}$ 2.37 $^{K}/_{ft}$ - INTERIOR BAY $\qquad \omega_L = \frac{68(30)}{1000} = 2.04\ ^{K}/_{ft}$ EXTERIOR



BALANCING MOMENT

$\omega_b = 3.28\ ^{K}/_{ft}$ - AVERAGE OF 3 BAYS



STRESSES IMMEDIATELY AFTER JACKING (DL + PT)

MIDSPAN

INTERIOR (BAY 2)

$f_{TOP} = \frac{\left[(209 - 140)(12)(1000)\right]}{6050} - \frac{1170(1000)}{3960}$

$= 137 - 295 = -158$ psi COMP $< 0.6 f'_c = 1800$ psi $\underline{\underline{OK}}$

$f_{BOT} = -137 - 295 = -432$ COMP $< 0.6 f'_c = 1800$ psi $\underline{\underline{OK}}$

PT DESIGN (6)

END (BAY 1)

$$f_{TOP} = \frac{[(-683 + 458)(12)(1000)]}{6050} - 295 =$$

$$-446 - 295 = -741 \text{ psi} < 0.60 f'_c = 1800 \text{ psi} \quad \underline{\underline{OK}}$$

$$f_{BOT} = 446 - 295 = 151 \text{ psi} \leq 3\sqrt{f'_{ci}} = 164 \text{ psi} \quad \underline{\underline{OK}}$$

SUPPORT STRESSES

$$f_{TOP} = \frac{[(628 - 422)(12)(1000)]}{6050} - 295$$

$$= 409 - 295 = 114 \text{ psi} < 3\sqrt{f'_{ci}} \quad \underline{\underline{OK}}$$

$$f_{BOT} = -409 - 295 = -704 \text{ psi} < 0.66 f'_{ci} \quad \underline{\underline{OK}}$$

STRESSES AT SERVICE LOAD (DL + LL + PT)

MIDSPAN

INTERIOR (BAY 2)

$$f_{TOP} = \frac{[(209 + 68 - 140)(12)(1000)]}{6050} - 295$$

$$= 272 - 295 = -23 \text{ psi} \quad \text{COMP} \quad < 0.45 f'_c = 2250 \text{ psi} \quad \underline{\underline{OK}}$$

$$f_{BOT} = -272 - 295 = -567 \text{ psi} \quad \text{COMP.} \quad < 0.45 f'_c = 2250 \text{ psi} \quad \underline{\underline{OK}}$$

EXTERIOR (BAY 1)

$$f_{TOP} = \frac{[(-683 - 281 + 458)(12)(1000)]}{6050} - 295$$

$$= -1004 - 295 = -1299 \text{ psi} \quad \text{COMP} < 0.45 f'_c = 2250 \text{ psi} \quad \underline{\underline{OK}}$$

$$f_{BOT} = 1004 - 295 = 709 \text{ psi} \quad \text{TENSION} > 6\sqrt{f'_c} = 424 \text{ psi} \quad \underline{\underline{\text{NO GOOD}}}$$

NEED TO EITHER INCREASE THE SLAB THICKNESS OR THE CONCRETE STRENGTH

PT DESIGN (7)

CHANGE f'_c

$$f'_{c_{REQ'D}} = \left(\frac{709}{6}\right)^2 = 13963 \rightarrow 14000 \text{ psi} \rightarrow \text{NOT PRACTICAL}$$

CHANGE SLAB THICKNESS

$h_{REQ'D}$ =

$$295 + 424 = 719$$

$$\frac{[(683 + 281 + 458)(12)(1000)]}{S} = 719$$

$$S = 8445$$

$$\frac{360(h)^2}{6} = 8445$$

$$h = 11.86 \text{ in} \rightarrow h = 12 \text{ in}$$

$$A = 360(12) = 4320 \text{ in}^2$$
$$S = \frac{360(12)^2}{6} = 8640 \text{ in}^3$$

$$\frac{P}{A} = \frac{1170(1000)}{4320} = 271 \text{ psi}$$

RECHECK STRESSES

@ JACKING

BAY 2

$f_{TOP} = 95.8 - 271 = -175.2$ psi (C) < 1800 psi OK

$f_{BOT} = -95.8 - 271 = -366.8$ psi (C) < 1800 psi OK

BAY 1

$f_{TOP} = -313 - 271 = -548$ psi (C) < 1800 psi OK

$f_{BOT} = 313 - 271 = 42$ psi (T) < 164 psi OK

@ SUPPORTS

$f_{TOP} = 286 - 271 = 15$ psi (T) < 164 psi OK

$f_{BOT} = -286 - 271 = -557$ psi (C) < 1800 psi OK

PT DESIGN (8)

@ SERVICE LOADS

BAY 2

$f_{TOP} = 186 - 271 = -85$ psi (c) < 2250 psi <u>OK</u>

$f_{BOT} = -186 - 271 = -457$ psi (c) < 2250 psi <u>OK</u>

BAY 1

$f_{TOP} = -703$ psi $- 271 = -974$ psi (c) < 2250 psi <u>OK</u>

$f_{BOT} = 703 - 271 = 432$ psi (T) > 424 psi <u>NO GOOD</u>

USE $h = 12$ in BUT INCREASE f'_c

$f'_c = \left[\frac{432}{6}\right]^2 = 5148$ psi $\rightarrow$ 6000 psi

$f_{BOT} = 432$ psi (T) < 465 psi <u>OK</u>

SUPPORTS

$f_{TOP} = 664 - 271 = 393$ (T) < 465 psi <u>OK</u>

$f_{BOT} = -664 - 271 = -935$ (c) < 2700 psi <u>OK</u>

SUMMARY

12 in SLAB
$f'_c = 6000$ psi
44 TENDONS

PT DESIGN (9)

CHECK PUNCHING SHEAR

SIMILAR FLOOR DEPTH AS TWO-WAY
SO USE 20" × 20" COLUMNS

$\omega_u = 1.2\left(\frac{12}{12} \times 150 + 25\right) + 1.6(100) = 370$ psf

INTERIOR COLUMN

$V_u = 370\left[33.1 \times 30 - 1.67 \times 1.67\right] = 366^k$

ASSUME # 5 BARS

$d = 12 - 0.75 - \frac{0.625}{2} = 10.9$ in

$b_o = (20 + 10.9)\,4 = 124$ in

$V_c = 4 \cdot \sqrt{4000} \cdot 124\,(10.9) = 419^k$

$\phi V_c = 0.75\,(419) = 314^k < V_u$ ∴ NEED DROP PANELS

EXTERIOR COLUMN

$V_u = 370\left[19.95 \times 30 - 1.67^2\right] = 220^k$

$b_o = (20 + 10.9) \cdot 3 = 92.7$

$\phi V_c = 0.75\,(4)\sqrt{4000}\,(92.7)(10.9)$
$= 235^k > V_u$

∴ DO NOT NEED DROP PANELS @ EXT. COLS.

DROP PANEL DEPTH = TRY 3 in
EXTEND DROP PANEL 5 in FROM FACE → 2.5' × 2.5' × 3" PANEL

$d = 15 - 0.75 - \frac{0.625}{2} = 13.9$ in

$b_o = (20 + 13.9) \cdot 4 = 136$

$\phi V_c = 0.75 \cdot 4 \cdot \sqrt{4000} \cdot 136\,(13.9) = 439^k > V_u$ OK

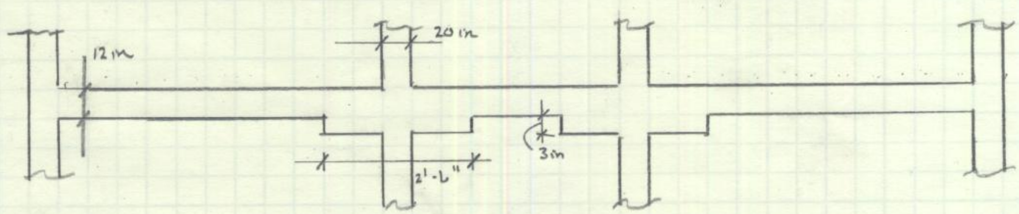


TABLE: Element Forces - Frames								
Frame	**Station**	**OutputCase**	**CaseType**	**M3**	**FrameElem**	**Station**	**M3**	**Location**
Text	in	Text	Text	Kip-in	Text	ft	Kip-ft	
1	0	DEAD	LinStatic	0	1-1	0	0	BEGIN
1	191.52	DEAD	LinStatic	8193.595	1-1	15.96	682.7996	MID*
1	478.8	DEAD	LinStatic	-7541.756	1-1	39.9	-628.48	END
2	0	DEAD	LinStatic	-7541.756	2-1	0	-628.48	BEGIN
2	157.2	DEAD	LinStatic	-2506.719	2-1	13.1	-208.893	MID*
2	314.4	DEAD	LinStatic	-7541.756	2-1	26.2	-628.48	END
3	0	DEAD	LinStatic	-7541.756	3-1	0	-628.48	BEGIN
3	191.52	DEAD	LinStatic	8193.595	3-1	15.96	682.7996	MID*
3	478.8	DEAD	LinStatic	-1.202E-11	3-1	39.9	-1E-12	END
4	0	DEAD	LinStatic	2.274E-13	4-1	0	1.9E-14	BEGIN
4	191.52	DEAD	LinStatic	3373.266	4-1	15.96	281.1055	MID*
4	478.8	DEAD	LinStatic	-3258.557	4-1	39.9	-271.546	END
5	0	DEAD	LinStatic	-3258.557	5-1	0	-271.546	BEGIN
5	157.2	DEAD	LinStatic	-818.263	5-1	13.1	-68.1886	MID*
5	314.4	DEAD	LinStatic	-3258.557	5-1	26.2	-271.546	END
6	0	DEAD	LinStatic	-3258.557	6-1	0	-271.546	BEGIN
6	191.52	DEAD	LinStatic	8193.595	6-1	15.96	682.7996	MID*
6	478.8	DEAD	LinStatic	-9.649E-12	6-1	39.9	-8E-13	END
7	0	DEAD	LinStatic	-4.547E-13	7-1	0	-3.8E-14	BEGIN
7	191.52	DEAD	LinStatic	-5495.908	7-1	15.96	-457.992	MID*
7	478.8	DEAD	LinStatic	5058.683	7-1	39.9	421.5569	END
8	0	DEAD	LinStatic	5058.683	8-1	0	421.5569	BEGIN
8	157.2	DEAD	LinStatic	1681.398	8-1	13.1	140.1165	MID*
8	314.4	DEAD	LinStatic	5058.683	8-1	26.2	421.5569	END
9	0	DEAD	LinStatic	5058.683	9-1	0	421.5569	BEGIN
9	287.28	DEAD	LinStatic	-5495.908	9-1	23.94	-457.992	MID*
9	478.8	DEAD	LinStatic	-4.974E-13	9-1	39.9	-4.1E-14	END
*MID represents the point between supports where the maximum moment occurs.								

Table 7: Abridged SAP Output

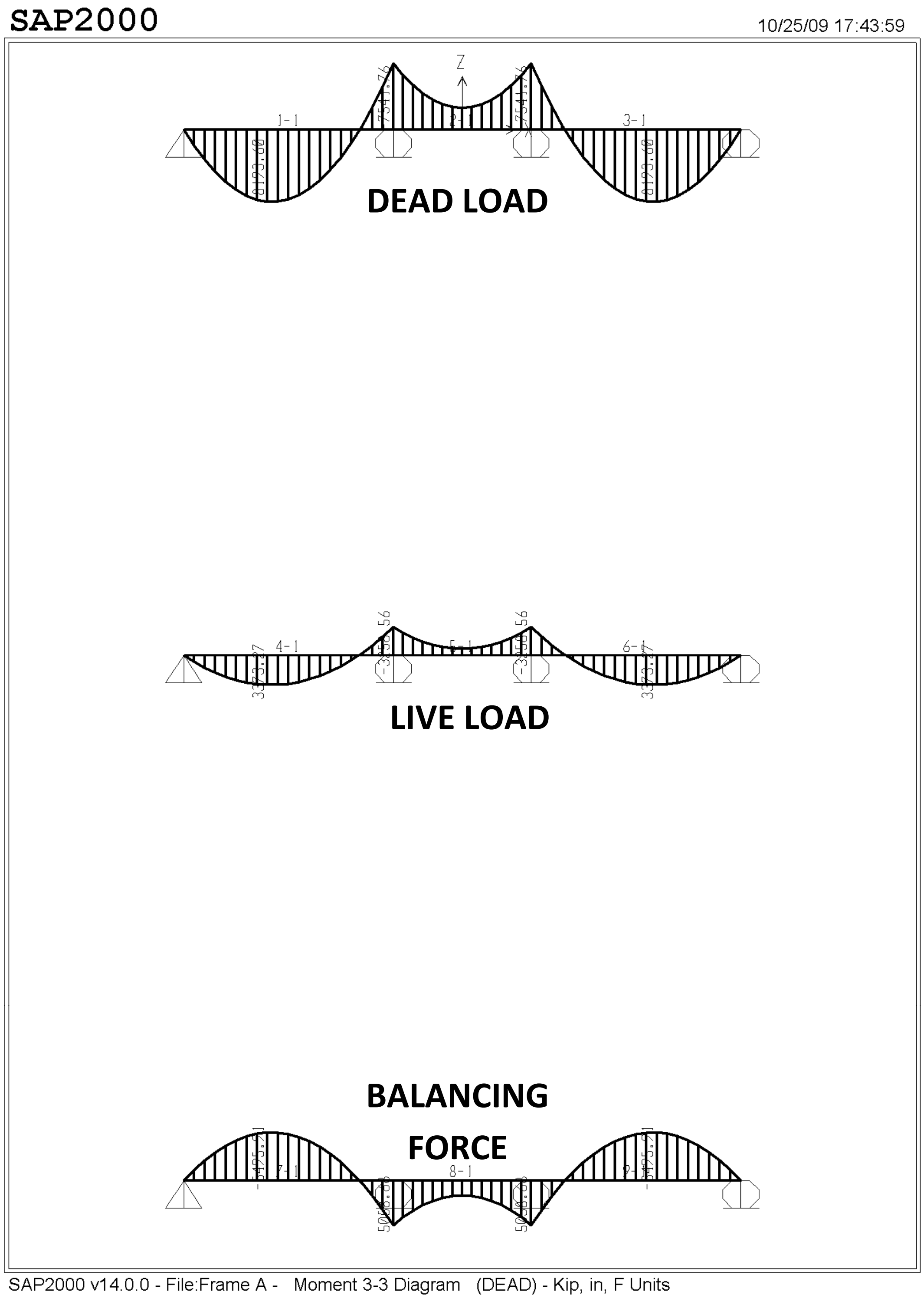


Figure 16: SAP Moment Diagrams