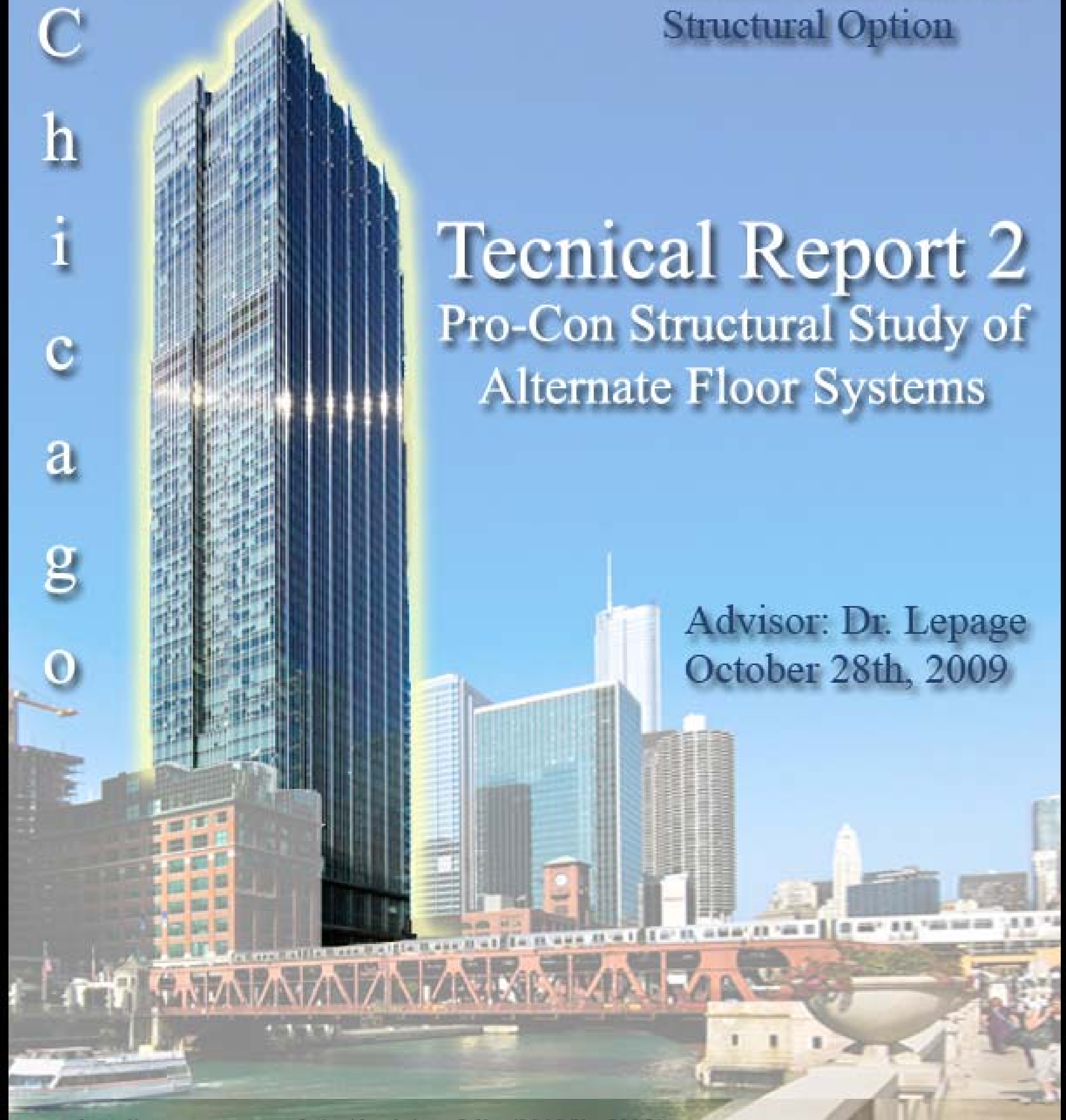
300 North La Salle

Liam McNamara
Structural Option

Chicago

Tecnical Report 2

Pro-Con Structural Study of Alternate Floor Systems



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Executive Summary

The second technical report for 300 North La Salle, a 60 story office building in Chicago, Illinois, is a structural study of the existing floor system as well as three alternative floor systems. A typical bay of 28.5' x 45', spanning lengthwise between exterior steel columns and an interior concrete bearing wall core, was designed and analyzed for each floor system. The floor systems were compared in structural, architectural, and construction categories. The structural categories important for comparisons are self weight, deflection, foundation effects, fire rating, and lateral system effects. The architectural categories were bay size, system depth, architectural impact, and vibration. The construction categories of importance are cost, constructability, and fireproofing. The existing floor system is composite beam with steel decking; it effectively spans the 45' length with four W18x35 beams per bay, and a total structural system depth of 24". The other three systems designed and analyzed in this report include:

- Open Web Steel Joist with Composite Deck
- Two-way Flat Plate
- Two-way Post-Tensioned slab with wide-shallow slab beams

The design of the open web steel joist system resulted in a 3" cast-in-place concrete slab over 3" metal decking supported by a combination of 45' long 28LH05 and 30K9 open web steel joists spaced 2' on center. The system had nearly identical cost and weight as the existing system. A possible advantage of this system is the ability to run MEP through the open webs reducing the floor to floor height in the building. The system has more severe deflection and vibration problems than the existing structure. The possible benefits of reducing the floor to floor height, which could provide additional floors without increasing the overall building height, and the system's ability to be used with various lateral systems, make it a viable option worth further studying.

The design of the two-way flat plate system required a 12" thick slab with the typical bay being divided into two equal bays sized 28.5'x 22.5'. This weight of this system was three times that of the original system which would cause a substantial change to the current foundation. Ultimately it was eliminated as a viable option. It required an interior colonnade diving the rentable space and columns in the corner of the buildings where there previously were none. This is unacceptable because the open, column free, floor plan is of major importance to the owner as a selling point to future renters.

The two-way post-tensioned slab was investigated because of its ability to span long distances while maintaining a thin slab thickness. The design resulted in an 8" thick slab with 16" thick x 4' wide slab beams spanning the 45' between the exterior columns and interior concrete core wall. Despite its additional weight, 114psf vs. the original 50psf, and it's difficultly to construct, the post-tensioned slab remains a feasible option for further evaluation. This is because it fits into the typical bay and works with the existing shear wall core lateral system, while achieving the goal of reducing the floor to ceiling depth.

Introduction

300 North La Salle is a 60-story high rise office building located on the north bank of the Chicago River in Chicago Illinois. It offers 25,000 gsf of rentable, column free floor space per level, with a total square footage of 1.3 million. Construction on the building began in 2006 and was completed in February of 2009 at a cost of $230 million. It is owned and managed by Hines developers and was designed by Pickard Chilton Architects. The primary tenant is Kirkland & Ellis, Chicago's largest law firm, occupying between 24 and 28 floors.

300 North La Salle rises elegantly above the Chicago River with a subtle set back above the 42nd floor. Its "fin-like" steel outriggers and aluminum mullions emphasize verticality. The appearance of structural members on the façade as well as the large open floor plans allude to Mies van der Rohe and the international style he helped make famous in Chicago. His international style incorporated open "universal" spaces that were easily adaptable with clearly arranged structural framework.

The structural engineers for the design were Magnusson Klemencic Associates. The superstructure is composed of a bearing concrete core and exterior steel W-shape "outrigger" columns. The bearing concrete core wall also acts as a shear wall core to carry lateral forces to the foundation. There is a "belt" of trusses spanning from the 41st to 43rd floors which aide in controlling lateral deflection of the structure and rotation within the shear wall core. The concrete strength of the core varies between 6,000 and 10,000 psi and the wall thicknesses vary between 1'6" and 2'3".

The typical floor system is composite beam with steel decking. It is composed of a 3" cast-in-place concrete slab on a 3" steel deck, and W-shape steel beams. The composite decking is typically 4,000 psi light-weight concrete. The steel members are Fy = 50 Ksi except for select columns on the lower level that are high strength Fy = 65 Ksi steel. The typical bay size is 28.5' x 45'. The system was chosen to efficiently span the 45' length creating a column free floor plan between the core and exterior of the building.

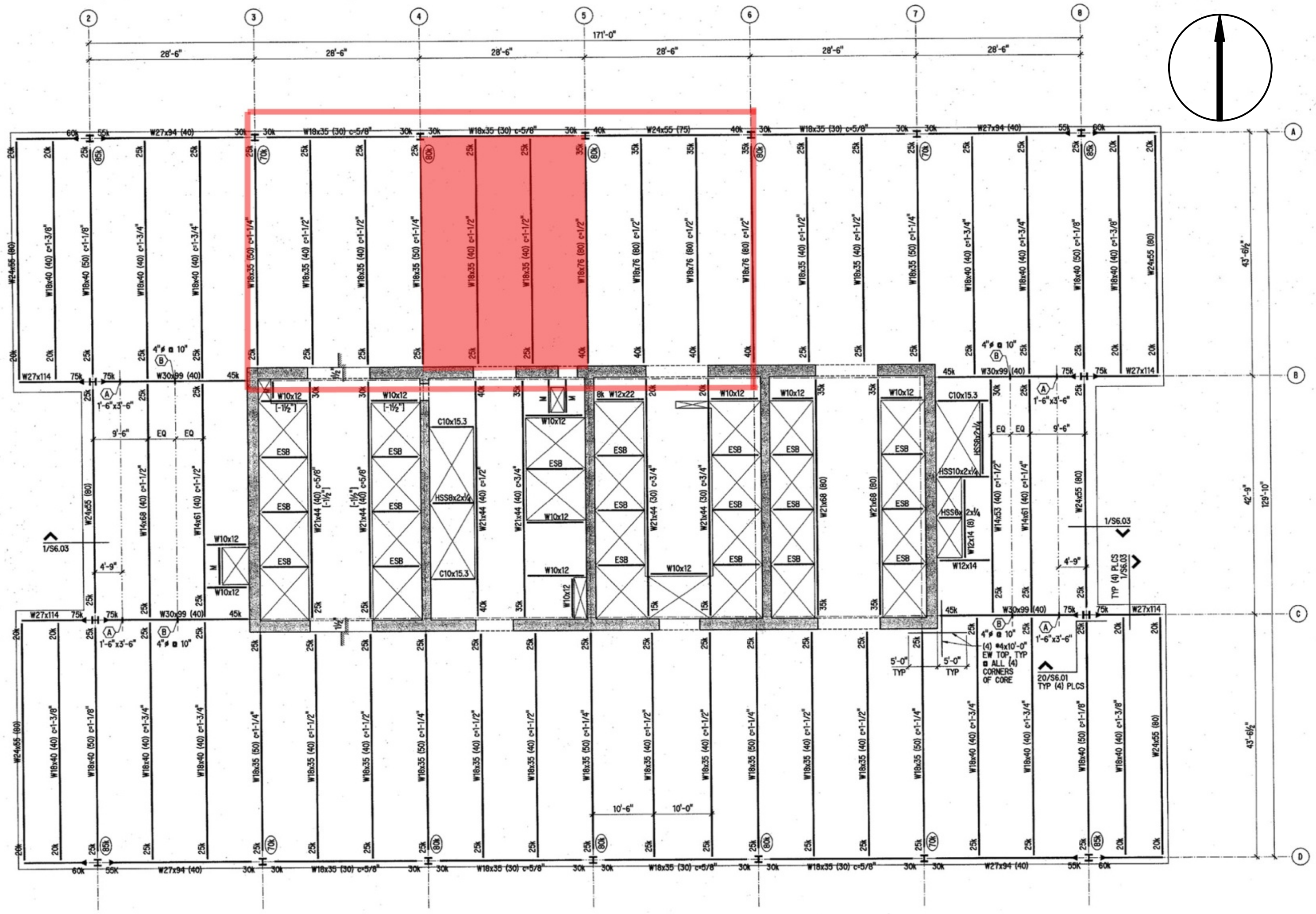


Figure 1 : Typical Bay located on 25th Floor

This report will be a study of the existing floor framing system as well as three alternative possibilities. The designs are all schematic based on the typical bay called out on the floor plan above. Multiple variables will be compared to analyze the feasibility of the systems such as; weight, architectural impact, structural system depth, constructability, foundation impact, lateral system impact, vibration, cost, and fireproofing. The three alternative floor systems to be discussed in this report will be open web steel joist with composite deck; two-way flat plate; and two-way post-tensioned slab with wide-shallow slab beams.

Existing Structural System

Foundations:

The foundation of the building is a combination of poured concrete piers and driven steel H-Piles with a 12" concrete slab sloping away from the core. The foundation slab is 28'-3" below grade and the foundation walls are 18" thick cast-in-place concrete around 3 levels of sub grade parking. The piers are drilled to approximately 72' below grade from top depths of 27'-41' below grade and have a bearing pressure of 40ksf. The piles are driven to refusal in bedrock at approximately 110' below grade and have a design bearing strength of 270 tons.

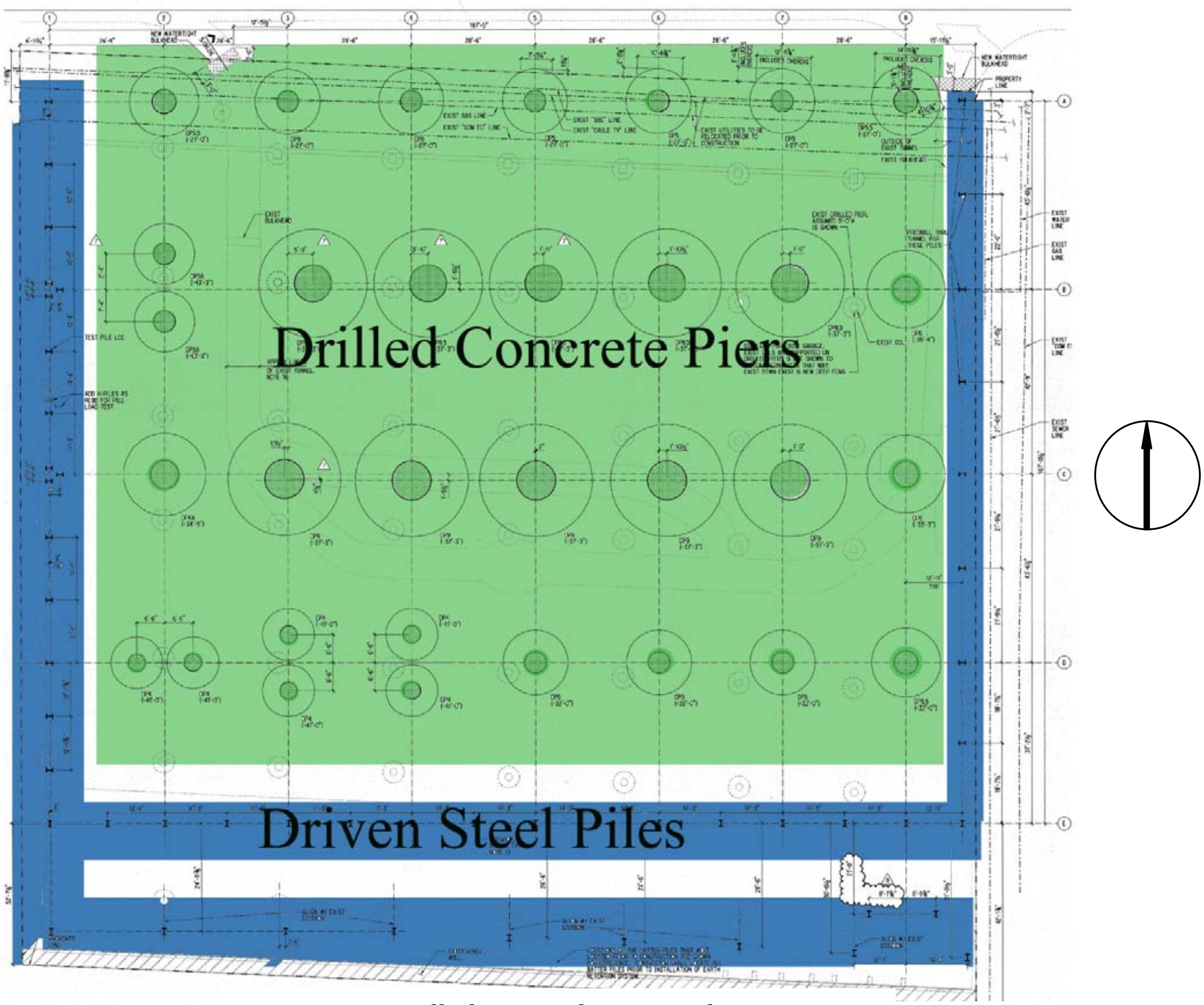


Figure 2 – Drilled Pier and Driven Pile Locations

Gravity System:

The main gravity-load is carried to the ground by exterior steel columns and an interior concrete core wall. The floor system on every floor is poured concrete slab over composite decking. While the slab varies from 3" light-weight concrete, on the office floors, to as thick as 8" normal-weight concrete in the mechanical area, the deck is a consistent 3" Type W minimum 20 gage galvanized steel. The composite decking transfers its loads onto 50ksi steel Wide flange beams typically spanning between 42'-9" and 43'-6½" spaced at 9.5' o.c. Below the elevator pits and Com Ed rooms on Lower Levels 1-4 the slab changes to normal weight 2-way flat concrete slab between 12" and 14" deep. The thickened two way flat slab is used to more readily carry the large live loads in these areas to the core. The roof system is also a light-weight concrete slab on 3" decking, however the beam size is increased to carry the additional weight from the green roof around the core of the building.

Lateral System:

Wind and seismic forces are resisted by a concrete shear wall core, strengthened by a series of outrigger and belt trusses between the 41st and 43rd floors. The shear wall core is cast-in-place normal weight concrete of 6,000; 8,000; and 10,000 psi strength depending on location. The wall reduces in thickness and plan as it rises through the building. The thickness reduces from 2'-3" to 2'-0" and then to 18" on the north and south walls at levels 9 and 43 respectively. The core has four 28'-6" bays running east-west as it rises from Lower Level 4 to Level 42, at Level 43 the core drops its outer two bays and continues through the penthouse with the inner two bays. The shear wall's step back to two bays corresponds to a 10' reduction in east-west width, at the top of the two story "belt" truss system. The floor and roof diaphragms carry the lateral loads to the shear wall core. The shear walls in the core then transfer the base shear, overturning moment, and rotational forces to the foundation.

The belt truss system is comprised of two multi-bay braced frames running east-west on the north and south exteriors, and three braced frames spanning north-south to the concrete shear wall on the interior of the building. The truss members are varying sizes of steel Wide flanges. The purpose of this "belt" truss system is to create a couple moment, from the outrigger steel columns in the event of lateral loading. This couple moment is applied on the shear wall core to fight rotation within the core, and therefore reduce the deflection of the building.

Structural Materials

Structural Steel:

W-Shapes………………………………………….ASTM A992 or A913, Fy=50 KSI
Angles……………………………………………….ASTM A36, Fy=36 KSI
Square of Rectangular
Structural Tube………………………….ASTM A500, Grade B, Fy=36 KSI
Steel Pipe d ≤ 12”……………………………….ASTM A53, Type E or S, Grade B, Fy=35 KSI
Material called out on
as (Fy= 65 KSI)…………………………..ASTM 913, Fy=65 KSI
All other steel…………………………………….ASTM A572, A588, A441, Fy=50 KSI

Metal Decking:

3” Composite Deck…………………………….Verco W3 - 20 gage minimum

Welding Electrodes:

E70 XX……………………………………………..70 KSI minimal tensile strength

Cast-in-Place Concrete:

Misc. Concrete, Curbs,
Sidewalks……………………………………..f’c = 4,000 psi – Normal Weight
Slab on Grade……………...………………………f’c = 4,000 psi – Normal Weight
Foundation Walls…………………………………..f’c = 5,000 psi – Normal Weight
Concrete on Steel Deck…………………………..f’c = 4,000 psi – Normal Weight
f’c = 4,000 psi – Light Weight
Columns, Reinforced Beams,
and Slabs……………………………………..f’c = 5,000 psi – Normal Weight
Shear Walls…………………………………………..f’c = 6,000 psi – Normal Weight
f’c = 8,000 psi – Normal Weight
f’c = 10,000 psi – Normal Weight
Grade Beams, Elevator Pits,
Caissons, Caps………………………………..f’c = 8,000 psi – Normal Weight

Reinforcement:

Reinforcing Bars……………………………………….ASTM A615, Grade 60
Welded Wire Fabric………………………………….ASTM A185

Masonry:

Hollow Concrete Units………………………………ASTM C90, $f'c_{min}$ = 1,900 psi

Codes and References

Design Codes:

National Model Code:

Chicago Building Code 2005

Design Codes:

American Concrete Institute (ACI), ACI 530-92, Requirements for Masonry Structures

ACI 318-83, Requirements for Structural Concrete

American Institute of Steel Construction (AISC), LRFD-86," Load and Resistance Factor Design Specification for Steel Buildings"

AISC-2000, "Specification for Structural Joints using ASTM A325 or A490 Bolts"

American Welding Society (AWS), AWS D1.1-2000, "Structural Welding Code- Steel"

AWS D1.3-98, "Structural Welding Code- Sheet Steel"

AWS D1.4-98, "Structural Welding Code-Reinforcing Steel"

AWS A2.4-98, "Symbols for Welding and Nondestructive testing"

American Iron and Steel Institute (AISI), "Specifications for the Design of Cold Formed Steel Structural Members," 1996 with supplement No.1 July 30, 1999

Structural Standards:

American National Standards Institute (ANSI), ANSI A58.1-1982

Thesis Codes:

National Model Code:

2006 International Building Code

Design Codes:

Steel Construction Manual 13th edition, AISC

ACI 318-05, Building Code Requirements for Structural Concrete

Structural Standards:

American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Design Loads and Deflection Limits

Superimposed Dead Loads		
Load Description	Load Location	Design Load (psf)
Office	Levels 9-40, 43-57	15 - Mech/Elec/Ceiling
Curtain Wall	All Levels	15 - vertical surface

Floor Live Loads			
Load Description	Load Location	Design Load (psf)	ASCE 7-05 Load (psf)
Office	Levels 9-40, 43-57	50 20 - Partitions	50
Corridors	Levels 2-58	--	80
Note - * Denotes a non-reducible live load as specified on load diagrams			

Live Load deflection will be limited to L/360.

Service Load deflection will be limited to L/240.

Construction Load deflection will be limited to L/180.

Note: When designing all of the floor systems a live load of 80psf will be used. This will allow the future tenants the freedom to layout the floor plans with corridors in any location.

Existing Floor System

Composite Beam and Deck

The existing floor system, Figure 3, was analyzed as a control to compare each of the alternate floor designs against. The bay is 28.5' x 45'; the floor composition is 3" light weight cast-in-place concrete slab over 3" composite decking supported by W18x35 beams spaced at 9.5' on center. The increase to W18x76 beams in the span adjacent to the typical span is due to a provided area for tenant filing which requires a larger live load capacity. This span was not analyzed because it is not in a typical location throughout the building's office floors.

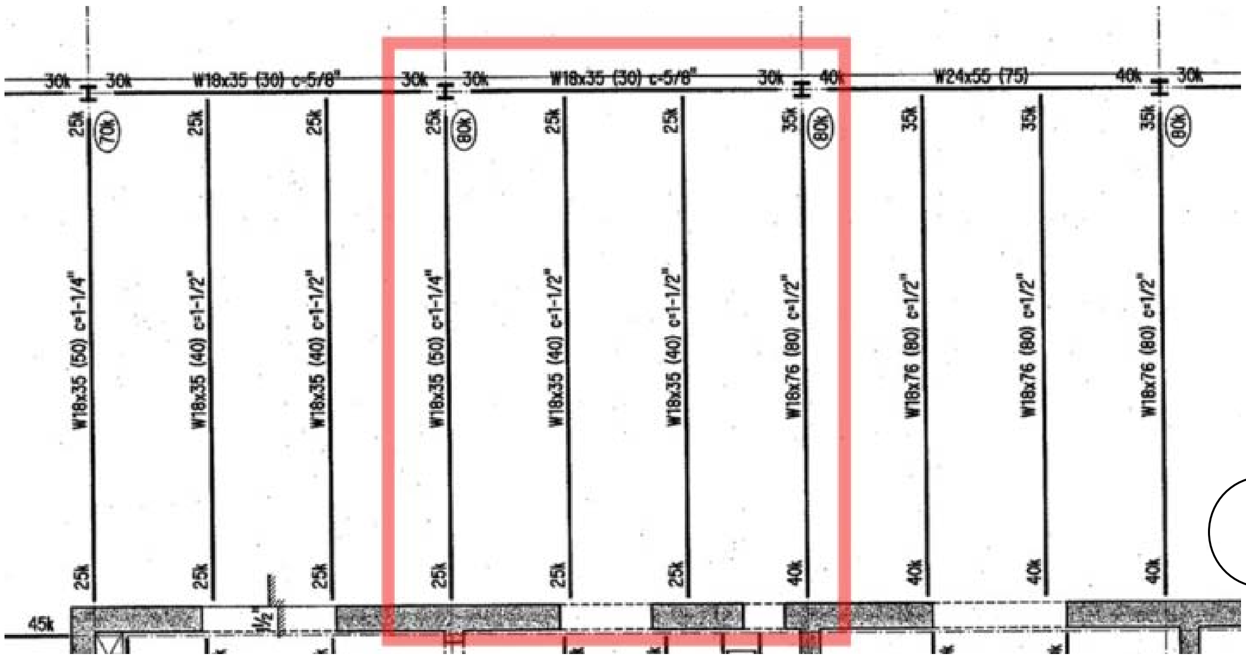


Figure 3 : Existing Steel Framing for typical bay

The floor system was modeled using *RAM Structural System*. During the RAM analysis the bays were modeled to the exact dimensions of the existing bay and are supported by columns on the exterior and a 2' wide concrete bearing wall on the interior. RAM designed beams that were just smaller than those of the original design but which required larger cambers and more shear studs. Figure 4 illustrates the RAM output for the existing bays.

While the new model uses smaller sizes it required larger cambering, 2" vs. 1.5", within the beams to meet the deflection criteria. The larger camber is required because the W16x31's have a lower Moment of Inertia than the designed W18x35's; a lower moment of inertia reduces the stiffness of the beam and in turn increases its mid-span deflection.

As a high rise building 300 North La Salle is exposed to large wind loads, this lateral load was determined to control the design in Technical Report 1. The existing design may be larger than designed by RAM to provide a stiffer floor system. A stiffer floor system can carry lateral loads more efficiently to the shear wall core. This can be reexamined upon further investigation and the inclusion of lateral loading in analysis.

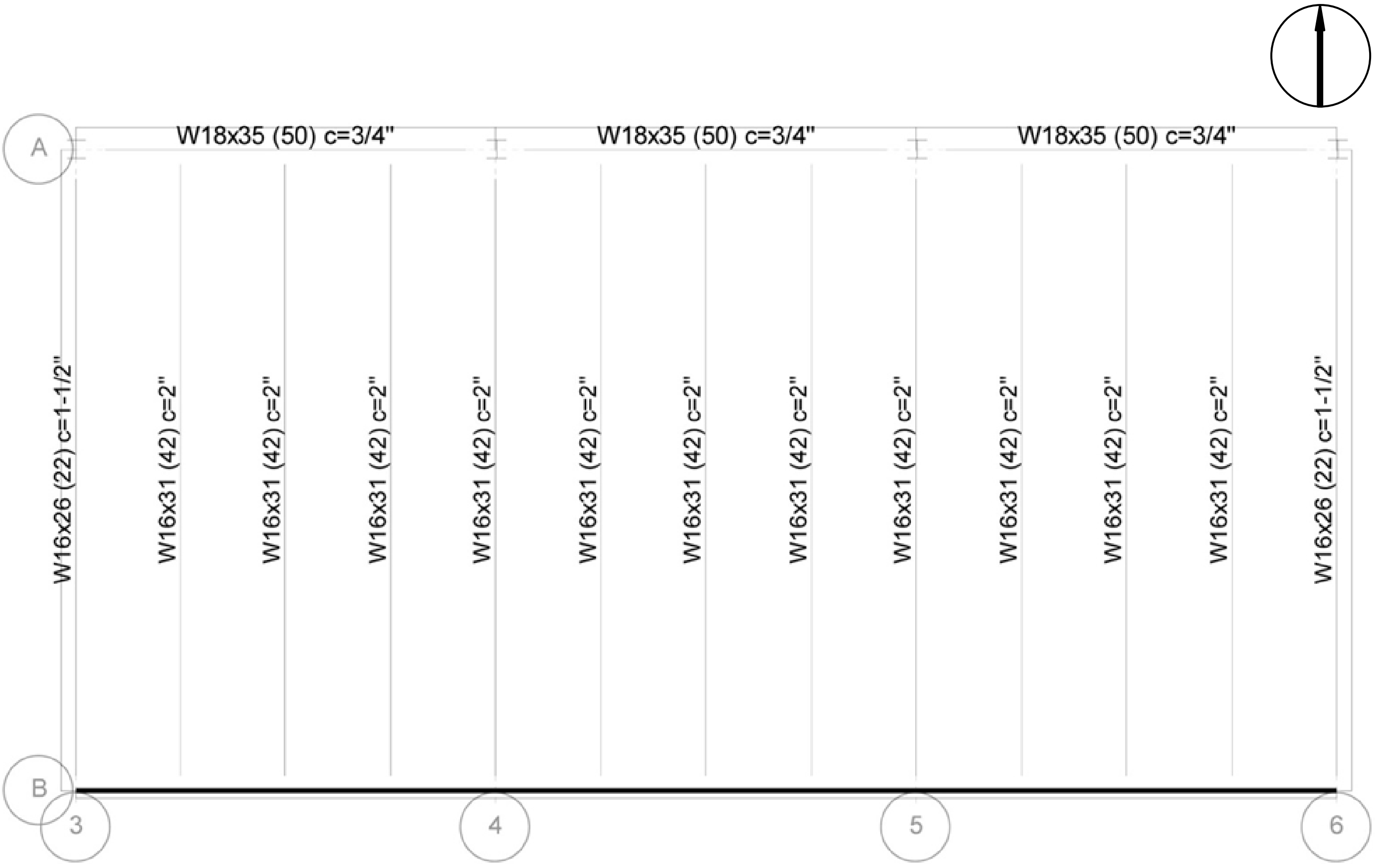


Figure 4 : RAM designed members for existing system and ASCE 7-05 design loads.

Pro-Con Analysis:

After the analysis of the existing system it is confirmed that the design can adequately carry the loads required by ASCE 7-05. One advantage of this system is that it is faster than concrete to erect. Also the steel decking spanning 9.5' on center acts as formwork for the cast-in-place concrete, and the small span does not require shoring. Formwork and shoring add time to construction as well as cost, and avoiding them can be a major benefit. Another advantage is that it can span the long 45' direction while still using relatively light steel 35lb/ft and only having a total depth of 24".

Some of the negatives for the existing structure are its higher cost, and the need for additional spray on fireproofing. While the 3" concrete slab in unison with the 3" composite deck provide the IBC required 2 hours of fireproofing between levels, the supporting steel beams have no inherent fire resistance and require spray on fireproofing, or bituminous paint. Both of these additional forms of fireproofing add cost to the building.

Overall the existing steel framing is a good system for 300 North La Salle. While the materials make the system itself more expensive, it can reduce the overall cost of the building through its relatively light weight. The light weight allows for reduced column sizes as well as smaller foundations.

Alternative Floor Systems

Open Web Steel Joist

This system was again designed using *RAM Structural Systems*. The same Verco W3 Formlok composite deck with 3" light weight concrete slab was used for the model. The shear wall and column locations remained the same as in the existing bay. The joists have a typical spacing of 2' on center, with an increase to 2.25' on either side of the column lines. This increase is due to the bay width of 28.5' which could not be divided evenly. The resulting joist depths varied between 28" and 30". The increase in depth to 30" occurs where the spacing of the trusses is increased, because the trusses must now carry the load from a larger tributary area.

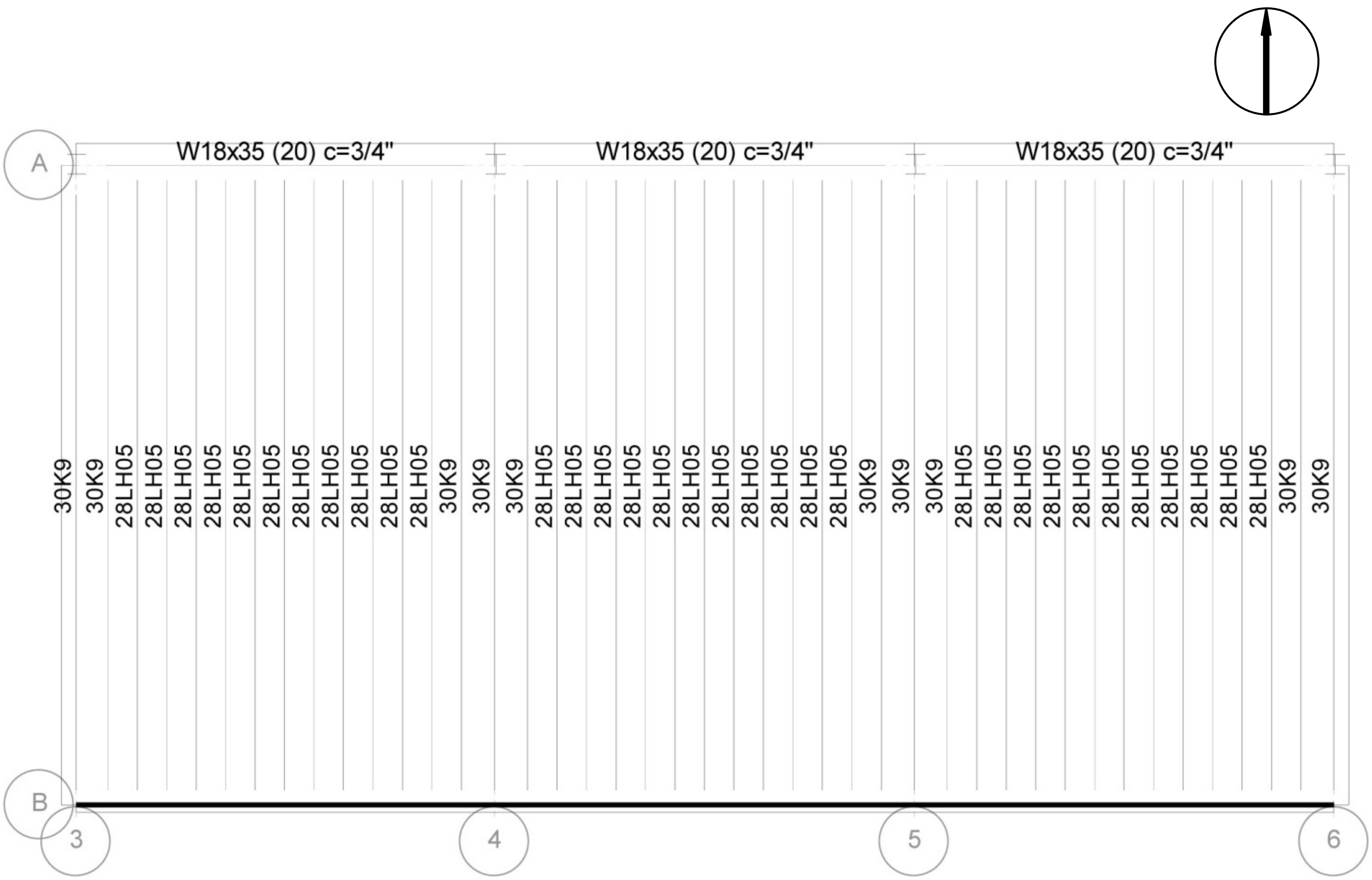


Figure 5: RAM design for open web steel joists with composite deck and shear wall.

Additionally RAM was run replacing the shear wall with steel framing. The second analysis was done to examine the size of the members in the absence of the concrete core. If the lateral system were to be designed as something other than a concrete shear wall core, the joists would no longer have a 2' wide concrete wall to bear upon. The new interior beams can be used as a reference when examining other lateral systems. The beams could potentially be part of a steel braced or moment frame lateral system during redesign. These sizes are the minimum needed to carry the gravity loads, and can be the initial size in the event of a possible redesign.

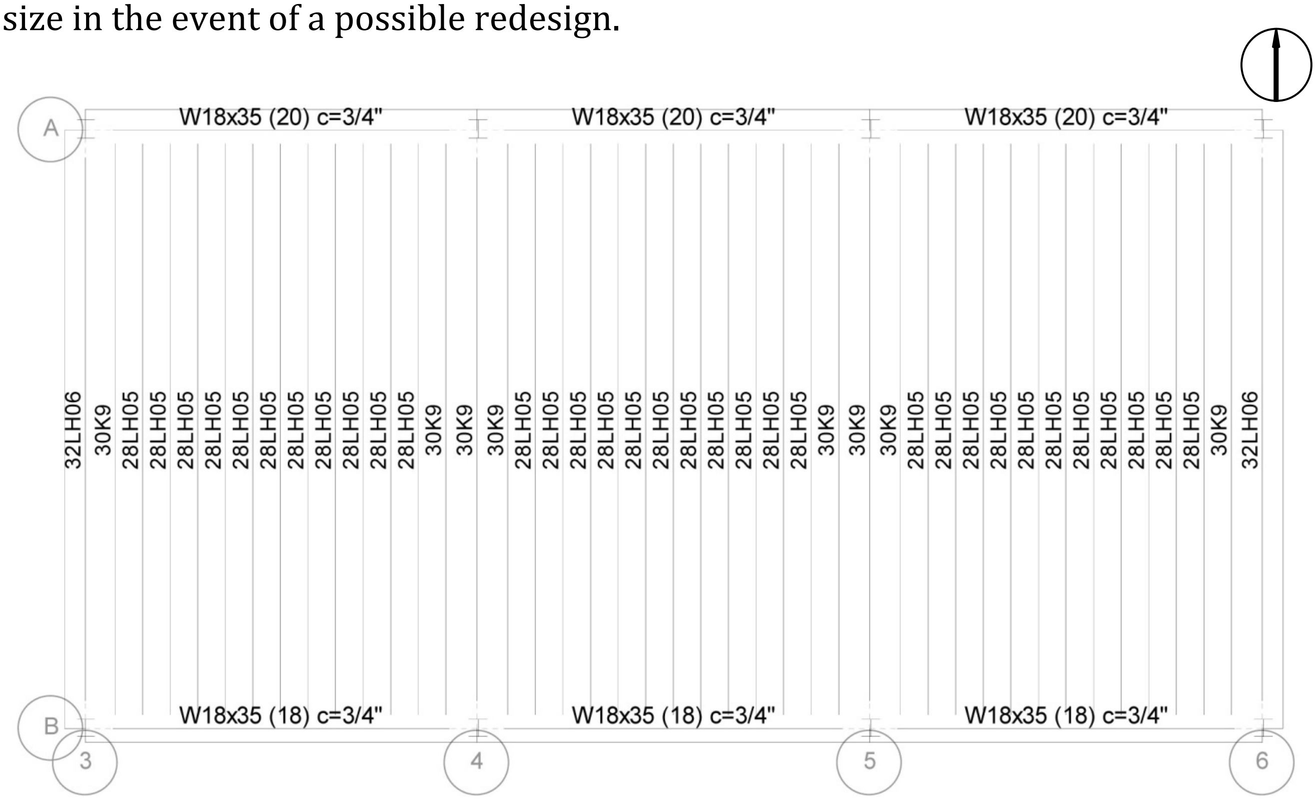


Figure 6 : RAM design for open web steel joists with composite deck on beams.

Pro-Con Analysis:

The composite steel joist system was initially analyzed in the hopes of reducing the weight of the system. It was also a benefit that steel joists can easily span long distances such as the 45' span in the typical bay. The steel joists were also investigated because they work well with the current lateral shear wall system, but also work well with other lateral systems such as steel braced and moment frames.

Another advantage of steel joists is the fairly easy construction required for installation. They are light; each joist weighs approximately 600 lbs compared to the 1600 lb steel W-shapes currently being used. This makes them easier to be moved around the site and lifted into place. Also they are easier to connect to the supporting members; they require a specified bearing length to rest on but do not require the larger bolted and welded shear connections that the existing steel beams require. One of the largest advantages of using steel joists is that the mechanical and electrical systems may be able to run through the open webs. Currently the MEP systems and ceiling add two feet to the structural framing creating a 4' deep "sandwich" between the floor above and the ceiling below. The ability to run these systems through the structure could reduce the "sandwich" and allow for the addition of more floors without increasing the overall building height.

However, upon analysis it can be seen that even with a small spacing such as the specified 2' on center, the joists require large depths between 28" and 30". With this spacing, the joist system ended up with essentially the same weight as the existing steel framing system. To try and reduce the weight would require a larger spacing of the joists. This was not done because it would require even deeper joists, and could cause deflection issues as the deflection is already much larger than the other systems. Another negative is the difficulty to fireproof the joists. The steel joists have no material resistance to fire, much like the existing system, however with their open webs it is difficult to ensure that all the members are adequately fireproofed. Lastly, the steel joists are more prone to have vibration issues as they are the least stiff of all the systems being explored.

While the steel joists can have vibration and deflection issues, the option of running the MEP through the open web as well as the ease of construction makes this a feasible system. The flexibility of the steel joist framing system to work with various lateral systems leaves this option open to further investigation.

Two-way Flat Plate

The two-way flat plate design was performed using pcaSlab. In order to examine a viable two-way flat plate design the bay size was reduced from 28.5' x 45' to two identical bays sized 28.5' x 22.5'. This was a major alteration because it creates an interior colonnade, breaking up the open floor plan. However, by basic design rule of thumb a flat plate design would not be used to span the original bay. The floor would have to be very deep and would require a large quantity of tensile reinforcement steel to carry the moments over the 45' length without large deflections.

The columns were each sized as 30" squares; this is slightly larger than the thickness of the shear wall core at the selected level. The initial slab thickness of 12" was determined from ACI Table 9.5(c), referenced in Appendix D. The pcaSlab punching shear check confirmed that all of the thicknesses and reinforcement were adequate. Punching shear was not checked along the length of the core wall, as it is not a failure mechanism for walls, only columns.

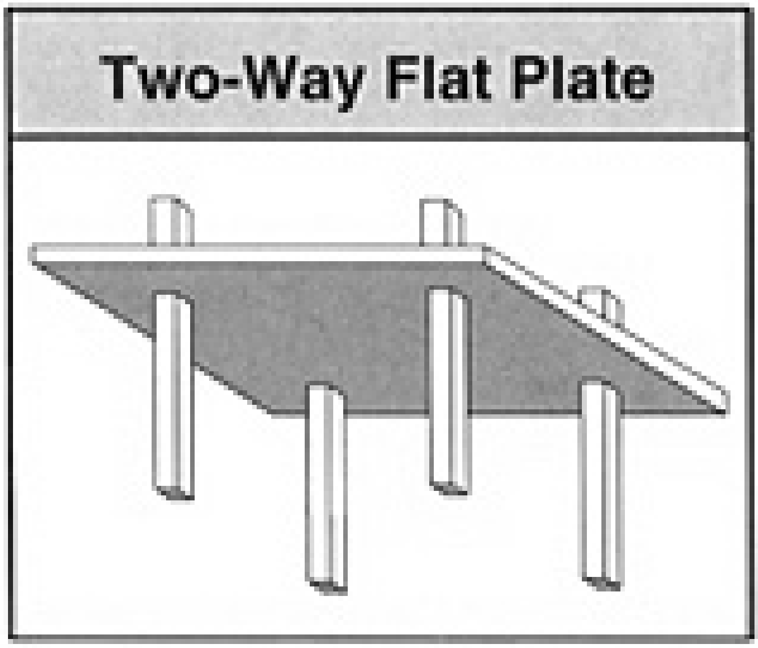


Figure 7: Typical Two-Way Flat Plate (www.crsi.org)

Pro-Con Analysis:

The cast-in-place two way flat plate design's largest advantage is that it reduces the total structural thickness from the existing 24" to 12". The saving of 1' per floor would reduce the floor to floor height to 12' and over 60 stories this could provide 5 more occupiable floors without increasing the height of the building. The flat plate system also works well with the existing shear wall core system and could be integrated fairly easily. Another benefit is its inherent fireproofing, the bottom clear cover provides the required 2 hour fire rating, and the system does not require the additional labor of spray-on fireproofing. The smaller bays also have a larger stiffness and therefore the system is the least susceptible to vibration and deflection, and also carries lateral loads efficiently to the shear walls.

A major disadvantage of the two-way flat plate system is that it would require an interior colonnade through the middle of the current 45' long span. It would also require columns in the corners of the building. Currently the building boasts that it offers large column free corner offices as a selling point to renters. The smaller bays and corner columns would have negative impacts on the flexibility currently available for interior office layout. The interior columns also require transfer girders or trusses to pick up their loads and carry them to the bearing wall core and exterior columns adding cost and weight. The increase in weight of the floor system by three times the current weight would have a major impact on the foundation, and also make the building more susceptible to seismic forces.

With such a large increase in floor weight the seismic effects would need to be reexamined. Also while the lower cost of the floor system may look like an advantage, the overall weight of the system would require much larger columns and foundations. The increase in size of these members, extended time of construction, and need for formwork and shoring, could ultimately make the building more costly and requires further investigation.

The two-way flat plate system is not feasible. It would result in too detrimental a change to the interior rentable space. The large interior columns it would require as well as the corner columns break up two of the main selling points for this office building.

Two-way Post-tensioning with Wide-Shallow slab beams

The post-tensioned slab was designed by hand using a Portland Cement Association (PCA) time saving design aid, as well as ACI 318-05, and Post-Tensioning Institute's Technical Notes by Dr. Bijan O. Aalami. The bay size was again the same as the existing bay 28.5' x 45'. A two way slab was designed due to the geometry of the bay (L2/L1 < 2). A wide-shallow slab beam was included between the columns running North-South along the 45' length of the bay. This wide-shallow slab beam allows the post-tensioning tendons to have an increased drape over its width. The increased tendon drape and slab thickness stiffens the slab in the long direction. To further incorporate the wide-shallow slab beams, the post-tensioning tendons draped in the long direction are banded together and lie solely in the beam, while the tendons for the short direction are distributed through the entire width.

The final floor system design consists of an 8" thick slab with 16" thick wide-shallow beams spanning 45', located at every column line.

Figure 8: Two-way Post-tensioned tendons prior to casting of concrete. (www.suncoast-pt.com)

Pro-Con Analysis:

The post-tensioned floor system was initially investigated because it allows large spans with thin slab thicknesses. The major advantage of this system is that it can span the existing bay while reducing the structural system thickness by 6" per floor. Over the 60 stories this reduction could provide for two more floors without increasing the height of the building. It also has inherent fireproofing like the two-way flat plate provided by its clear cover. The increased stiffness of the system from the weight and wide-shallow beams also makes it less prone to vibration problems than the existing structure. Post-tensioning also allows for cantilevered slabs and does not require the columns at the corners of the building that two-way flat plate does. The post-tensioning floor system also works well with the existing shear wall core and outrigger lateral system.

A disadvantage is that post-tensioning construction is difficult and requires specialized and experienced contractors. Also openings in the slab must be predesigned to adjust the tendon layout around them. This is a disadvantage as some renters will be renting multiple floors and plan on installing interior stairwells. This option would be restricted by post-tensioning design and could be a negative for future renters. Also the increased weight of the floor system and the concrete columns it would now need to support it, while not as heavy as the two-way flat plate, would have a large impact on the foundation.

Overall the ability of the post-tensioned floor system to provide the same typical bays while decreasing floor depth makes this a viable option worth further investigation in the future.

Conclusion

	Structural Floor Systems			
Considerations	Existing Steel Framing	Composite Steel Joists	Two-way flat plate	Post-Tensioned w/ wide shallow beams
Total Structural Depth (in.)	24	36*	12	16
Constructability	Easy-Medium	Medium- see fireproofing	Medium	Difficult
Foundation Impact	N/A	No	Greatly increases capacity requirements	Increases Capacity Requirements
Lateral System	No	No	No	No
Weight (psf)	49.64	50.40	150	114.0
Deflection (in.)	0.67	2.107	0.2178	N/A
Relative Vibration	Average	Above Average	Lowest	Low
Fireproofing	Easy-spray on	Difficult- Spray on	No	No
Fire rating (hrs)	2	2	2	2
Cost ($/ft^2)	27	25	11	16
Bay size	28.5' x 45'	28.5' x 45'	28.5'x22.5'	28.5' x 45'
Architectural Impact	N/A	No	Yes	No
Feasibility	N/A	Yes	No	Yes

* Signifies that the increase in total structural depth does not directly effect floor to ceiling depth.

In the second technical report for 300 North La Salle, alternate floor systems were analyzed and compared to the existing floor system. This was performed by studying the design of these systems within a selected typical bay. Major factors in determining alternative floor systems were floor depth, ability to span long distances, and required lateral systems. To benefit the design of 300 North La Salle a floor system must be able to span the long open interior space between the shear wall core and the exterior columns while also reducing the floor the floor height. The ability to do this would allow the owner to add additional floors without increasing the building height.

The existing composite beam and deck system efficiently carries the gravity load across the 45' span and maintains the lightest weight of all the floor systems studied. It also works as a fairly stiff diaphragm carrying the lateral load to the shear wall core. The flexibility of steel composite beam construction allows for this floor system to be used with various other lateral systems such as steel braced or moment frames.

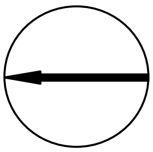
Due to the requirement of maintaining the long column free span the two-way flat plate system is not feasible for 300 North La Salle. By reducing the bay size from one 28.5'x 45' bay into two 28.5'x22.5' bays a colonnade is placed through the center of the open office floor plans. The heavy weight of the system would also add a large amount of loading into an already deep foundation system which could cause problems and is another reason the system is not feasible.

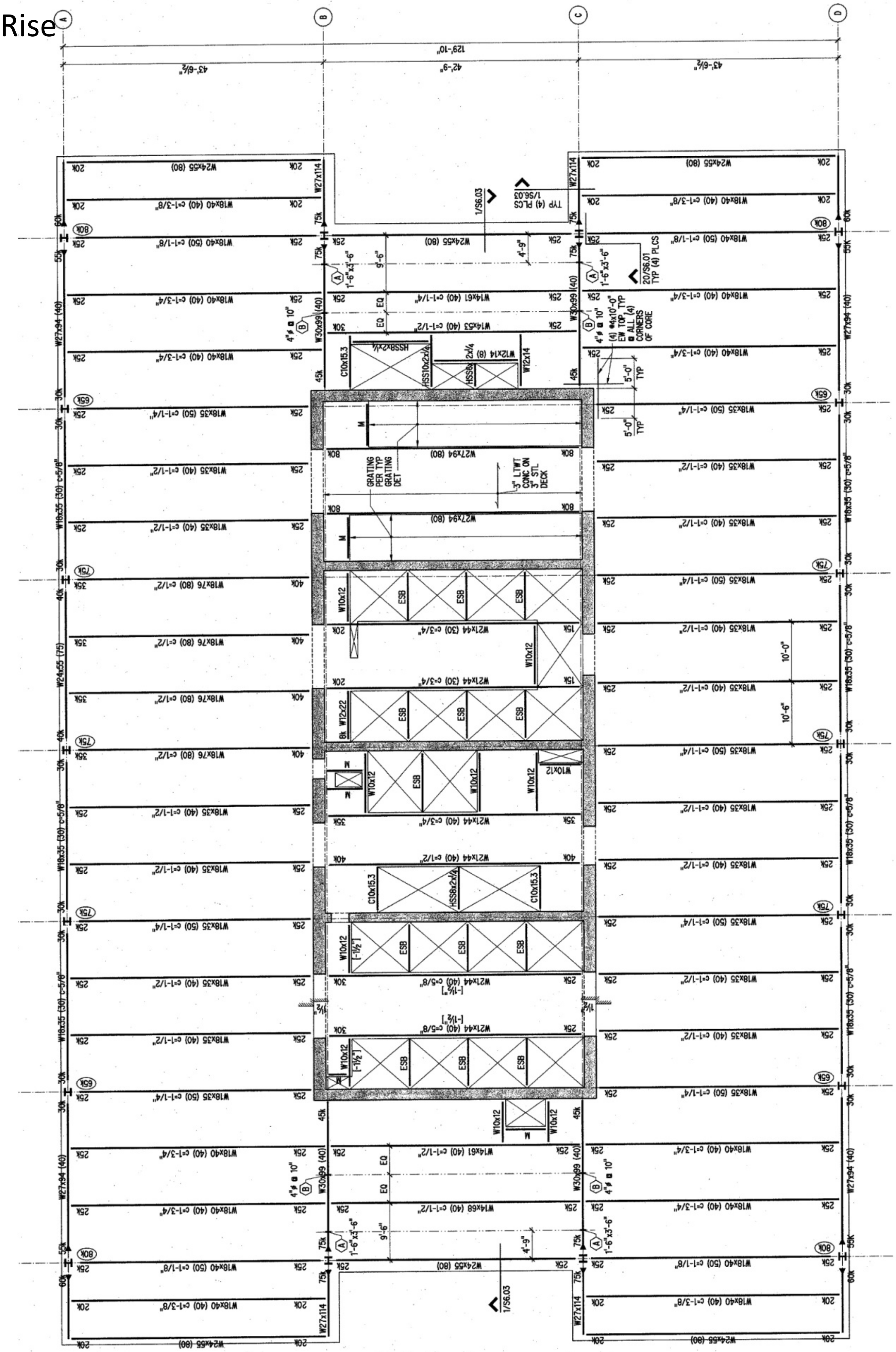
The steel composite joist system successfully spans the 45' length with a negligible increase in system weight. While the 30K and 28LH joists themselves are deeper than the current steel W-shapes, their open webs could provide space to run the mechanical, electrical, and plumbing systems, effectively reducing the floor to ceiling depth. The possibility to reduce the floor to ceiling depth as well as the system's flexibility in regards to different lateral systems make it a viable candidate for further study and a feasible system for 300 North La Salle.

The post-tensioned with wide-shallow slab beams system also successfully spans the 45' length while reducing the structural system depth by 8". Even though the increase in weight will have an effect on the foundation, the post-tensioned systems ability to work with the shear wall lateral system and its reduced cost of $16/sqft make it a feasible system for further consideration. Further study must go into the foundation effects as well as the possible increase in labor costs due to the inherent difficulty of post-tensioned construction.

Appendix A – Typical Floor Plans

Typical Low/Mid-Rise





Appendix B: Existing Floor System

Typical Slab Spot Check

Typical Slab Spot Check

- Typical Floor = 3" slab of light weight concrete on 3" composite steel deck
  - * Based on minimum steel deck = 20 GA
  - * Max Dead load deflection = 3/4" or L/190.

Max unsupported length = 9'6"

Floor DL:

15 psf = Mech / Elec / Ceiling SDL

* LWC = 115 pcf

Depth = 3" + 1/2 (Deck = 3") = 4.5"

Conc. PSF = (115 pcf)(4.5"/12"/1') = 43.125 psf

Steel Deck = * Allowable Decks = Verco Type W or ASC Type W

From Vercodeck.com W3 Formlock 20 GA Galv. = 2.3 psf

Total Dead load = 60.425 psf

Total Live load = based on loading diagram

Most severe load = "G" increased office live load

LL = 100 psf

LL (Partitions) = 20 psf

LL = 120 psf

$W_u = 1.2D + 1.6L$

$W_u = 1.2(60.425) + 1.6(120)$

$W_u = 264.51$ psf

From: Verco Decking, INC. Steel Floor Decks Catalog

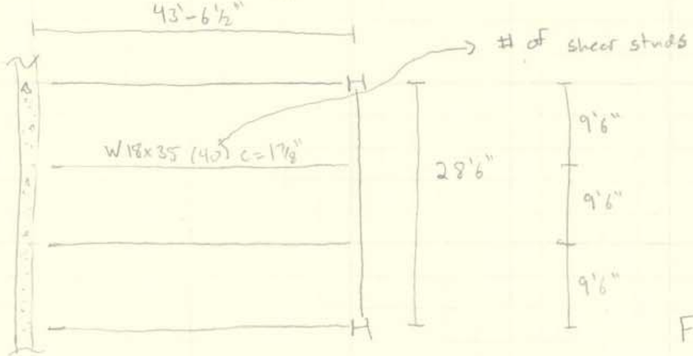
Allowable superimposed loads:

20 GA @ 9'6" span = 268 psf > 264.51 psf (too close bump up deck)

18 GA @ 9'6" span = 338 psf > 264.51 psf ∴ ✓ OK

Typical Beam Spot Check

Beam Spot Check: Typical Floor – Level 27



Floor DL = 45.425 (From typ slab ✓)
SDL from load diag. = 15 psf
Total DL = 60.425 psf

Floor LL =
50 psf LL
20 psf Partitions
Total LL = 70 psf
* From load diagram

Lightweight Concrete
w = 115 pcf
f'_c = 4000 psi

Note: Beam is assumed to be pinned-pinned.

W 18 x 35
F_y = 50 ksi
I_x = 510 in^4
A_s = 10.3 in^2

Live Load Reduction – Reference AISC Steel Construction Manual 13th Ed.

A_I = Influence Area = (9'6" x2)(43'-6½") = 827.3 ft^2 > 400 ft^2 ∴ Reducible

$$LL = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) = 70\text{psf}\left(0.25 + \frac{15}{\sqrt{827.3}}\right) = 54.0\text{ psf}$$

Using LRFD:

Tributary Width = 9'6"

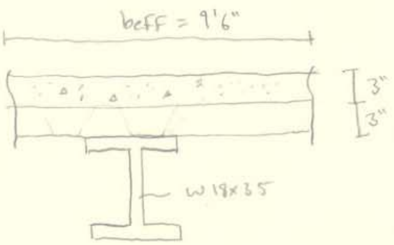
W_u = 1.2D + 1.6L (controlling combination)

W_u = 1.2(60.425) + 1.6(54) = 158.91 psf

W_u = 158.91 psf (9'6")
W_u = 1509.65 plf
+ Self weight of beam = 1.2(35)

$$M_u = \frac{w_u l^2}{8} = \frac{(1,551.65\text{ plf})(43'\text{-}6\tfrac{1}{2}")^2}{8} / 1000 = 366.9 \text{ k-ft}$$

b_{eff} ≤ | Spacing = 9'6" → controls
| span/4 = 43'6½"/4 = 10'11"

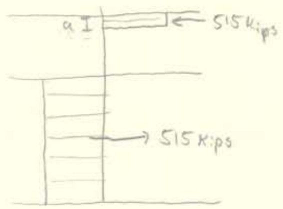


Deck runs perpendicular to beam

$A_{conc} = (3")(9'6" \times \frac{12"}{1'}) = 342 in^2$

$C_c = 0.85 F'_c A_c = 0.85 (4 ksi)(342 in^2) = 1162.8$ Kips

$T_s = A_s F_y = (10.3 in^2)(50 ksi) = 515$ kips



$a = \frac{T_s}{0.85(F'_c)(beff)} = \frac{515 k}{0.85(4)(9'6 \times 12)} = 1.33"$

$Y = 6" - \frac{a}{2} = 6" - \frac{1.33}{2} = 5.34"$ ~> (5" conservative)

From Steel Manual Table 3-19

@ P.N.A = BFL & Y2 = 5"
$M_n = 435$ k-ft > 366.9 k-ft
$\Sigma Q_n = 260$ K

Determine Required # of Shear Studs:

- Drawing notes designate 3/4" "Nelson" or "Tru-Weld" studs @ maximum 2'0" spacing.

Q_n per stud $= 0.5 A_{sc} \sqrt{F'_c E_c} \le R_g R_p A_{sc} F_u = 0.442(65) = 28.73$ ($R_g = 1.0$, $R_p = 1.0$)

$E_c = w^{1.5} \sqrt{F'_c} = 115^{1.5} \sqrt{4} = 2466$ ksi
$A_{sc} = \frac{\pi (3/4)^2}{4} = 0.442 in^2$

$Q_n = 0.5(0.442)\sqrt{4(2466)} = 21.9^k < 28.73"$ ✓

# of studs $= \frac{260^k}{21.9^k} = 11.87 \approx 12$ studs × 2 sides of beam
= 24 studs < 40 studs ✓OK

★ Ribs are spaced every 12" o.c. the 40 studs prescribed follows placement of a stud in all but 3 ribs, most likely for constructibility as all beams on the floor call for 40 studs.

Deflection Check

$\frac{lbs}{ft} \times \frac{1}{12}$

Live Load Deflection (ΔL) $= \frac{5 w_L l^4}{384 EI} = \frac{(5)(54psf)(9'-6")(43'-6\frac{1}{2}" \times 12")^3}{384(29,000)(I)}$

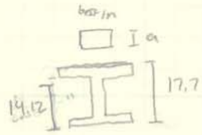
w_L = service Live Load

$I_{TR} = I_o + Ad^2$

$f_c A_c = f_t A_t$

$\frac{f_c A_c}{f_t} = A_t$

$\frac{A_c}{n} = A_t$



$\frac{b_{eff}}{n} = \frac{114"}{11.76} = 9.7"$

$\frac{E_s}{E_c} = \frac{29,000}{2466} = n = 11.76$

$I_T = \frac{bh^3}{12} = \frac{(9.7")(1.33")^3}{12}$

$I_T = 1.9 in^4$

$A_T = (9.7)(1.33) = 12.9 in^2$

$A_s = 10.3 in^2$

$\bar{y} = \frac{A_s(\frac{h}{2}) + A_T(h_s + h_c - \frac{a}{2})}{A_s + A_T} = \frac{10.3(17.7/2) + 12.9(13 + 6 - \frac{1.33}{2})}{10.3 + 12.9}$

$\bar{y} = 14.12"$

$I_{tr} = I_{os} + A_s d^2 + I_t + A_c d^2$

$I_{tr} = 510 in^4 + 10.3(14.12 - \frac{17.7}{2})^2 + 1.9 in^4 + 12.9 in^2 (17.7 - 14.12 + 6 - \frac{1.33}{2})^2$

$I_{tr} = 1823.2 in^4$

$\Delta_{LL} = \frac{5(.513 k/ft)(43'6\frac{1}{2}")^3(1728)}{384(29,000)(1823.2)} = 0.018" \ll \frac{L}{360} = 1.45"$ ✓ok

Deflection During Construction

$W_D = 45.425 psf (9'6") = 431.5 plf + 35 plf$ (weight of beam) $= 466.5 plf$

$W_L = 20 psf (9'6") = 190 plf$

$W_T = 1.2(466.5 plf) + 1.6(190 plf) = .864 k/ft$

$M_u = \frac{(0.864)(43'6\frac{1}{2}")^2}{8} = 204'^k < \phi M_p = 249'^k$ (AISC Manual Table 3-6)

$\Delta_{DL} = \frac{5}{384} \frac{(0.4665)(43'-6\frac{1}{2}")^4(1728)}{(29,000)(510)} = 2.54" - 1\frac{7}{8}"$ camber $= \frac{2}{3}" < \frac{3}{4}" < 2.9"$ ✓ok

Δ_R Max allowed by design docs $= \frac{L}{180}$ or $\frac{3}{4}"$

$L/180 = 2.9"$

★ Beam is adequate to carry loads.

RAM Structural Systems: Beam Design Criteria

UNBRACED LENGTH:

Check Unbraced Length
Do Not Consider Point of Inflection as Brace Point
Noncomposite/Precomposite Beam Design:
Deck Perpendicular to Beam Braces flange
Deck Parallel to Beam does not Brace flange
Calculate Cb for all Simple Span Beams
Use Cb=1 for all Cantilevers

SPAN/DEPTH CRITERIA:

Maximum Span/Depth Ratio (ft/ft): 0.00

DEFLECTION CRITERIA:

Default Criteria	L/d	delta (in)
Unshored		
Initial (Construction Load):	0.0	0.0
Post Composite		
Live Load:	360.0	0.0
Total Superimposed:	240.0	0.0
Total (Init+Superimp-Camber):	240.0	0.0
Shored		
Dead Load:	0.0	0.0
Live Load:	360.0	0.0
Total Load:	240.0	0.0
Noncomposite		
Dead Load:	0.0	0.0
Live Load:	360.0	0.0
Total Load:	240.0	0.0

Alternate Criteria	L/d	delta (in)
Unshored		
Initial (Construction Load):	0.0	0.0
Post Composite		
Live Load:	0.0	0.0
Total Superimposed:	0.0	0.0
Total (Init+Superimp-Camber):	0.0	0.0
Shored		
Dead Load:	0.0	0.0
Live Load:	0.0	0.0
Total Load:	0.0	0.0
Noncomposite		
Dead Load:	0.0	0.0
Live Load:	0.0	0.0

Note: 0.0 indicates No Limit

CAMBER CRITERIA FOR COMPOSITE BEAMS:

Do not Camber Beams with Span < 0.0 ft
Do not Camber Beams with Weight < 0.0 lbs/ft
Do not Camber Beams with Weight > 1000.0 lbs/ft
Do not Camber Beams with Depth < 0.0 in
Do not Camber Beams with Depth > 100.0 in
Percent of Dead Load used for Camber: 80.00
(For Unshored Composite the specified % of Construction DL is used)
Camber Increment (in): 0.250
Minimum Camber (in): 0.750
Maximum Camber (in): 4.000

CAMBER CRITERIA FOR NONCOMPOSITE BEAMS:

Do not Camber Beams with Span < 0.0 ft
Do not Camber Beams with Weight < 0.0 lbs/ft
Do not Camber Beams with Weight > 1000.0 lbs/ft
Do not Camber Beams with Depth < 0.0 in
Do not Camber Beams with Depth > 100.0 in
Percent of Dead Load used for Camber: 80.00
Camber Increment (in): 0.250
Minimum Camber (in): 0.500
Maximum Camber (in): 4.000

STUD CRITERIA:

Stud Distribution: Use Optimum
Maximum % of Full Composite Allowed: 100.00
Minimum % of Full Composite Allowed: 25.00
Maximum Rows of Studs Allowed: 3
Minimum Flange Width for 2 Rows of Studs (in): 5.500
Minimum Flange Width for 3 Rows of Studs (in): 8.500
Maximum Stud Spacing: Per Code

WEB OPENING CRITERIA:

Stiffener Fy (ksi): 36.000
Stiffener Dimensions
Minimum Width (in): 1.000
Minimum Thickness (in): 0.250
Increment of Width (in): 0.250
Increment of Thickness (in): 0.125
Increment of Length (in): 1.000
Do Not Allow Stiffeners on One Side of web
Allow Stiffeners on Two Sides of web

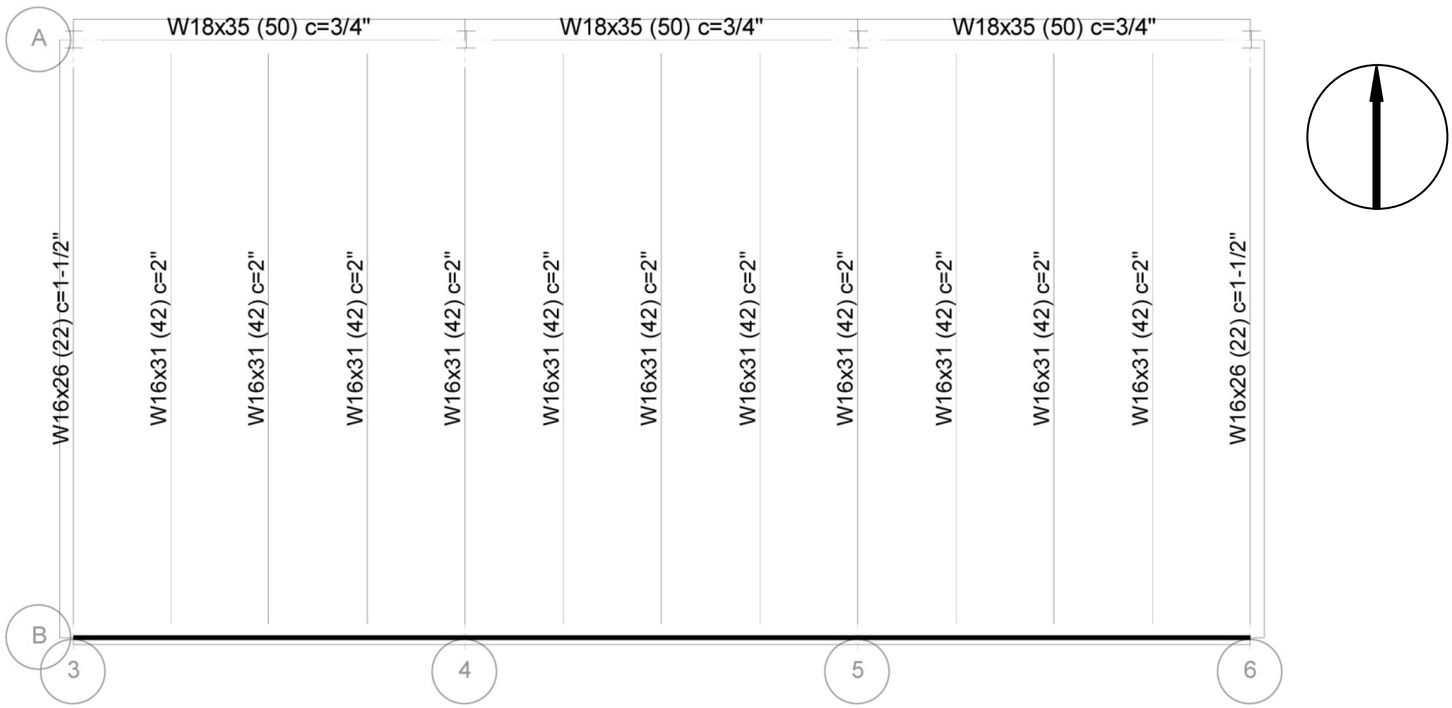
RAM Structural Systems: Required Sizes for Design Loads

Floor Map

RAM Steel v12.1
DataBase: 3 bays
Building Code: IBC

10/28/09 14:29:27
Steel Code: AISC360-05 LRFD

Floor Type: Type 1



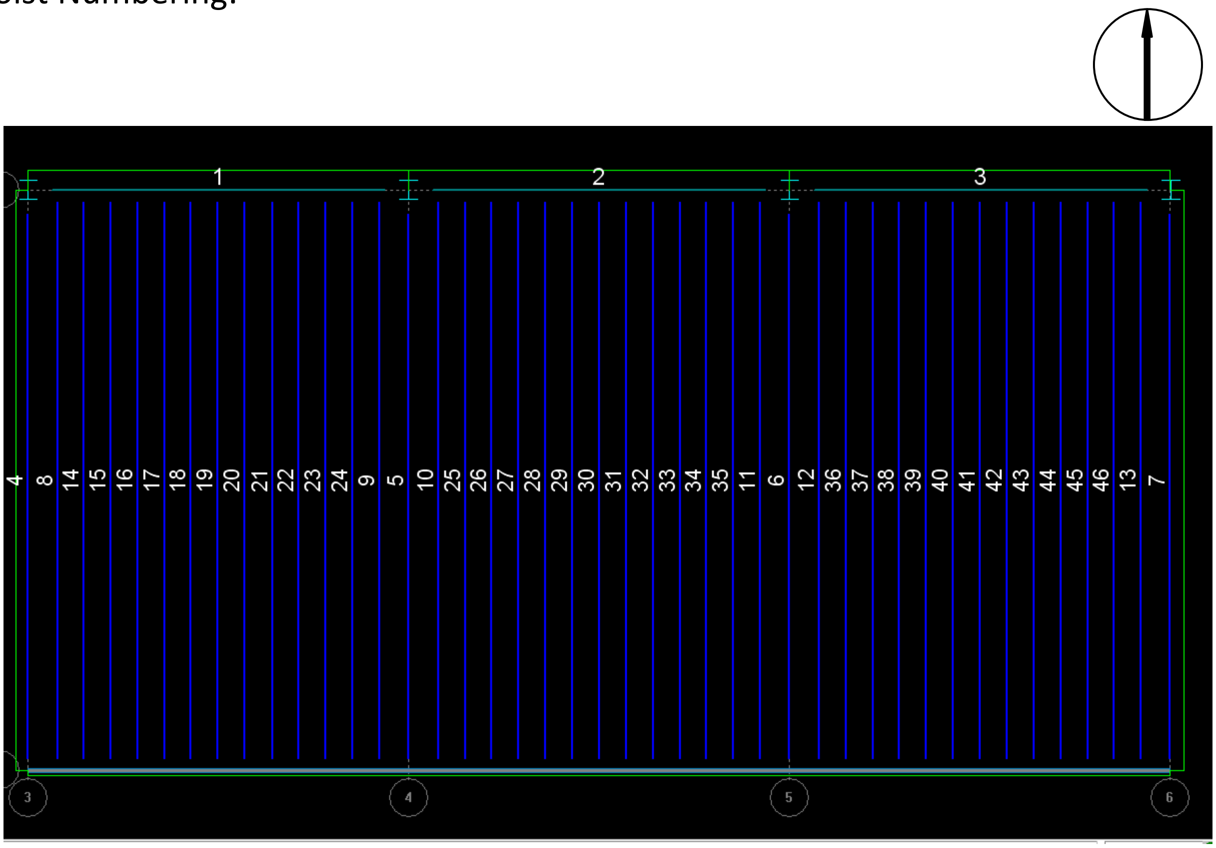
Appendix C: Open Web Steel Joist with Composite Deck

Floor Type: Type 1

Standard Joists

Bm #	Beam Size	Dead in	Live in	Total in
4	30K9	0.874	1.144	2.018
8	30K9	0.874	1.144	2.018
14	28LH05	0.913	1.195	2.107
15	28LH05	0.913	1.195	2.107
16	28LH05	0.913	1.195	2.107
17	28LH05	0.913	1.195	2.107
18	28LH05	0.913	1.195	2.107
19	28LH05	0.913	1.195	2.107
20	28LH05	0.913	1.195	2.107
21	28LH05	0.913	1.195	2.107
22	28LH05	0.913	1.195	2.107
23	28LH05	0.913	1.195	2.107
24	28LH05	0.913	1.195	2.107
9	30K9	0.874	1.144	2.018
5	30K9	0.925	1.211	2.137
10	30K9	0.874	1.144	2.018
25	28LH05	0.913	1.195	2.107
26	28LH05	0.913	1.195	2.107
27	28LH05	0.913	1.195	2.107
28	28LH05	0.913	1.195	2.107
29	28LH05	0.913	1.195	2.107
30	28LH05	0.913	1.195	2.107
31	28LH05	0.913	1.195	2.107
32	28LH05	0.913	1.195	2.107
33	28LH05	0.913	1.195	2.107
34	28LH05	0.913	1.195	2.107
35	28LH05	0.913	1.195	2.107
11	30K9	0.874	1.144	2.018
6	30K9	0.925	1.211	2.137
12	30K9	0.874	1.144	2.018
36	28LH05	0.913	1.195	2.107
37	28LH05	0.913	1.195	2.107
38	28LH05	0.913	1.195	2.107
39	28LH05	0.913	1.195	2.107
40	28LH05	0.913	1.195	2.107
41	28LH05	0.913	1.195	2.107
42	28LH05	0.913	1.195	2.107
43	28LH05	0.913	1.195	2.107
44	28LH05	0.913	1.195	2.107
45	28LH05	0.913	1.195	2.107
46	28LH05	0.913	1.195	2.107
13	30K9	0.874	1.144	2.018
7	30K9	0.874	1.144	2.018

Joist Numbering:



STANDARD LOAD TABLE/LONGSPAN STEEL JOISTS, LH-SERIES
Based on a Maximum Allowable Tensile Stress of 30 ksi

Joist Deisignation	Approx. Wt. in Lbs. per Linear Ft. (Joists Only)	Depth in Inches	SAFELOAD* in Lbs. Between	CLEAR SPAN IN FEET							**								
			28-32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48
24LH03	11	24	11500	342 235	339 226	336 218	323 204	307 188	293 175	279 162	267 152	255 141	244 132	234 124	224 116	215 109	207 102	199 96	191 90
24LH04	12	24	14100	419 288	398 265	379 246	360 227	343 210	327 195	312 182	298 169	285 158	273 148	262 138	251 130	241 122	231 114	222 107	214 101
24LH05	13	24	15100	449 308	446 297	440 285	419 264	399 244	380 226	363 210	347 196	331 182	317 171	304 160	291 150	280 141	269 132	258 124	248 117
24LH06	16	24	20300	604 411	579 382	555 356	530 331	504 306	480 284	457 263	437 245	417 228	399 211	381 197	364 184	348 172	334 161	320 152	307 142
24LH07	17	24	22300	665 452	638 421	613 393	588 367	565 343	541 320	516 297	491 276	468 257	446 239	426 223	407 208	389 195	373 182	357 171	343 161
24LH08	18	24	23800	707 480	677 447	649 416	622 388	597 362	572 338	545 314	520 292	497 272	475 254	455 238	435 222	417 208	400 196	384 184	369 173
24LH09	21	24	28000	832 562	808 530	785 501	764 460	731 424	696 393	663 363	632 337	602 313	574 292	548 272	524 254	501 238	480 223	460 209	441 196
24LH10	23	24	29600	882 596	856 559	832 528	809 500	788 474	768 439	737 406	702 378	668 351	637 326	608 304	582 285	556 266	533 249	511 234	490 220
24LH11	25	24	31200 **	927 624	900 588	875 555	851 525	829 498	807 472	787 449	768 418	734 388	701 361	671 337	642 315	616 294	590 276	567 259	544 243
			33-40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56
28LH05	13	28	14000	337 219	323 205	310 192	297 180	286 169	275 159	265 150	255 142	245 133	237 126	228 119	220 113	213 107	206 102	199 97	193 92
28LH06	16	28	18600	448 289	429 270	412 253	395 238	379 223	364 209	350 197	337 186	324 175	313 166	301 156	291 148	281 140	271 133	262 126	25 120
28LH07	17	28	21000	505 326	484 305	464 285	445 267	427 251	410 236	394 222	379 209	365 197	352 186	339 176	327 166	319 158	305 150	295 142	285 135
28LH08	18	28	22500	540 348	517 325	496 305	475 285	456 268	438 252	420 236	403 222	387 209	371 196	357 185	344 175	331 165	319 156	308 148	297 140
28LH09	21	28	27700	667 428	639 400	612 375	586 351	563 329	540 309	519 291	499 274	481 258	463 243	446 228	430 216	415 204	401 193	387 183	374 173
28LH10	23	28	30300	729 466	704 439	679 414	651 388	625 364	600 342	576 322	554 303	533 285	513 269	495 255	477 241	460 228	444 215	429 204	415 193

Appendix D: Two-way Flat Plate

pcaSlab uses the Equivalent Frame Method to analyze slabs. In order to design the interior bay shown in figure blank, two orthogonal frames were input into pcaSlab. These frames allow for the complete design of the bay providing necessary reinforcing & slab thickness in both the transverse and longitudinal directions.

Office Live Load = 80 psf

- Allows for freedom of layout for corridors
- Partition load is not required by 4.2.2 (ASCE 7-05

Curtain wall = 15 psf (vertical surface)
= 15 psf (13') = 195 plf

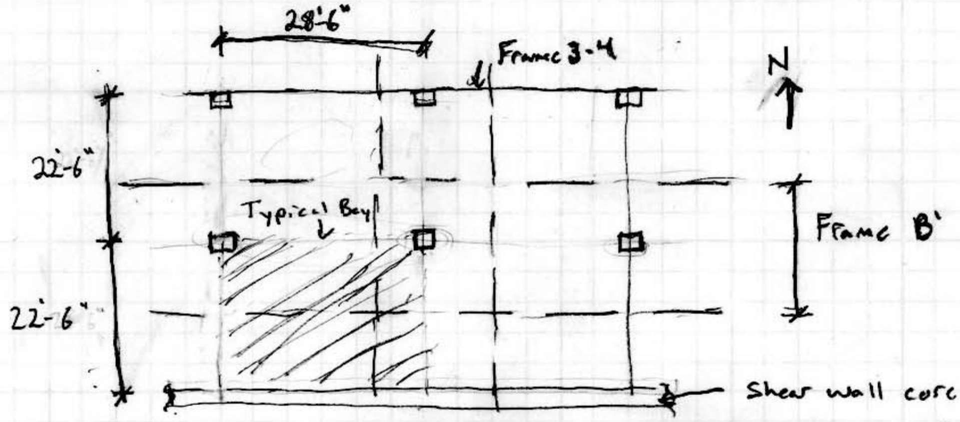
★ Assumed carried by exterior columns ∴ won't apply to typical span.

2-Way Flat Plate

ACI 318-02

	Slab & Beams	Columns
Unit Density	150 pcf	150 pcf
Comp. strength f'_c	4 ksi	4 ksi

Assume col's = 30" x 30"



★ Note: Assume bays continuous in both directions ∴ reinforcing for analyzed frames will apply for entire typical slab.

Minimum slab thickness ACI 318-08 Table 9.5(c)

$f_y = 60{,}000$ psi

Without drop panels or edge beams

$t_{min} = \frac{l}{33}$

Frame B $t_{min} = \frac{28'\text{-}6" - 24"}{33} = 9.63"$

Frame 3-4 $t_{min} = \frac{22'\text{-}6" - 24"}{33} = 7.45"$

Frame B'
Design for 12" ✓ok

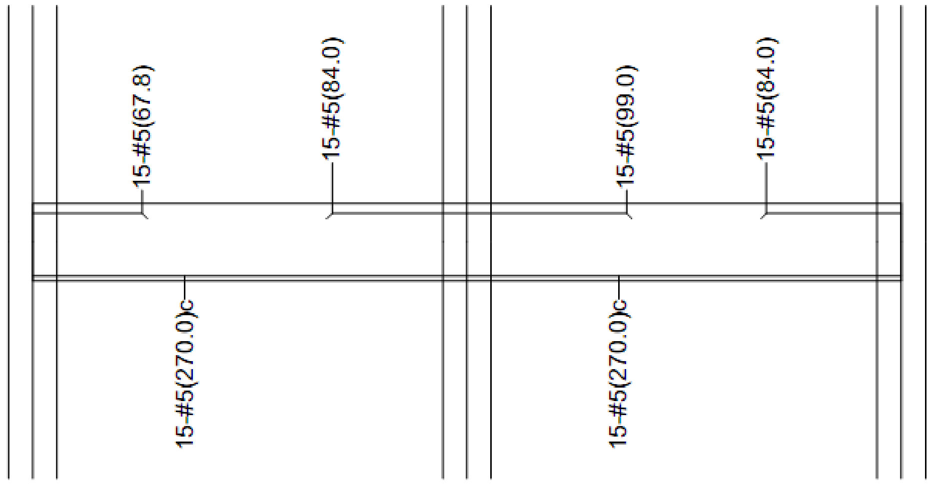
Frame 3-4:
Design for 12" ✓ok

Slab Material Properties:

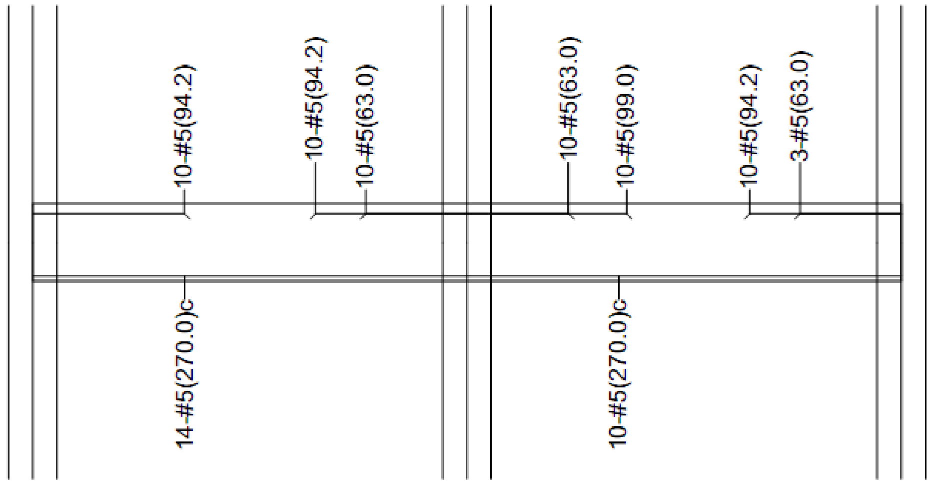
```
Material Properties:
====================
              Slabs|Beams       Columns
             ------------   ------------
  wc     =            150            150 lb/ft3
  f'c    =              4              4 ksi
  Ec     =         3834.3         3834.3 ksi
  fr     =        0.47434        0.47434 ksi

  fy     =             60 ksi, Bars are not epoxy-coated
  fyv    =             60 ksi
  Es     =          29000 ksi
```

Frame 3-4 Reinforcement:



Middle Strip Flexural Reinforcement



Column Strip Flexural Reinforcement

Frame 3-4 Shear Checks & Deflection:

Slab Shear Capacity:

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc	Vu	Xu
1	342.00	10.19	1.000	330.53	101.93	20.40
2	342.00	10.19	1.000	330.53	90.29	20.40

Flexural Transfer of Negative Unbalanced Moment at Supports:

Units: Width (in), Munb (k-ft), As (in^2)

Supp	Width	GammaF*Munb	Comb	Pat	AsReq	AsProv	Additional Bars
1	66.00	82.99	U2	Odd	1.855	1.516	2-#5
2	66.00	67.13	U2	Odd	1.494	3.031	---
3	--- Not checked ---						

Punching Shear Around Columns:

Units: Vu (kip), Munb (k-ft), vu (psi), Phi*vc (psi)

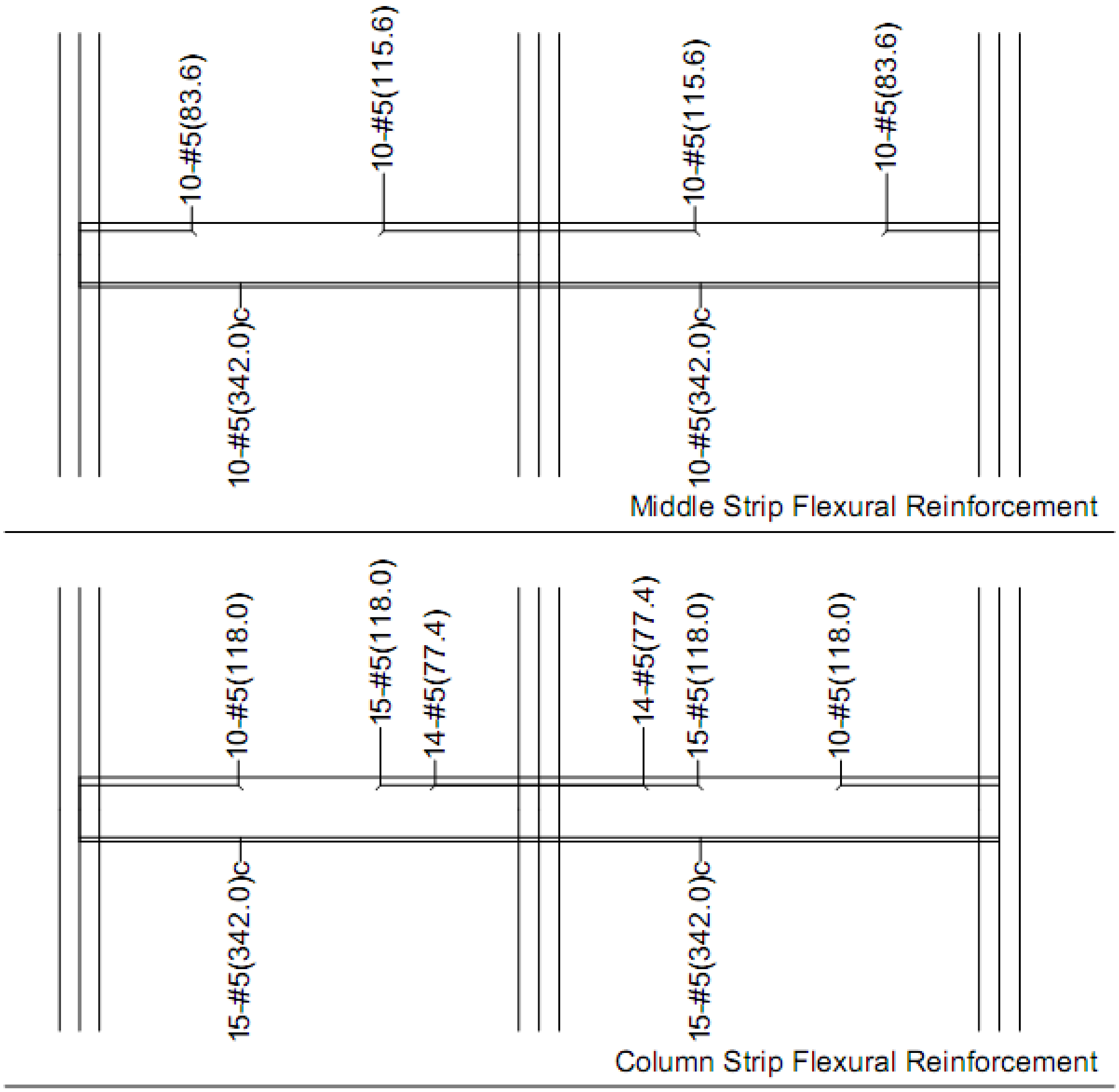
Supp	Vu	vu	Munb	Comb	Pat	GammaV	vu	Phi*vc
1	88.65	110.8	11.30	U2	S1	0.320	116.7	189.7
2	224.08	140.0	-62.57	U2	S2	0.400	153.9	189.7
3	--- Not checked ---							

Maximum Deflections:

Units: Dz (in)

	Frame			Column Strip			Middle Strip		
Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.055	-0.031	-0.086	-0.103	-0.058	-0.161	-0.024	-0.014	-0.037
2	-0.018	-0.009	-0.027	-0.029	-0.014	-0.042	-0.012	-0.006	-0.017

Frame B' Reinforcement:



Frame B' Shear Checks and Deflection:

Slab Shear Capacity:

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc	Vu	Xu
1	270.00	10.19	1.000	260.95	104.16	26.40
2	270.00	10.19	1.000	260.95	104.16	2.10

Flexural Transfer of Negative Unbalanced Moment at Supports:

Units: Width (in), Munb (k-ft), As (in^2)

Supp	Width	GammaF*Munb	Comb	Pat	AsReq	AsProv	Additional Bars
1	66.00	160.92	U2	Odd	3.689	1.516	8-#5
2	66.00	82.66	U2	Even	1.848	4.395	---
3	66.00	160.92	U2	Even	3.689	1.516	8-#5

Punching Shear Around Columns:

Units: Vu (kip), Munb (k-ft), vu (psi), Phi*vc (psi)

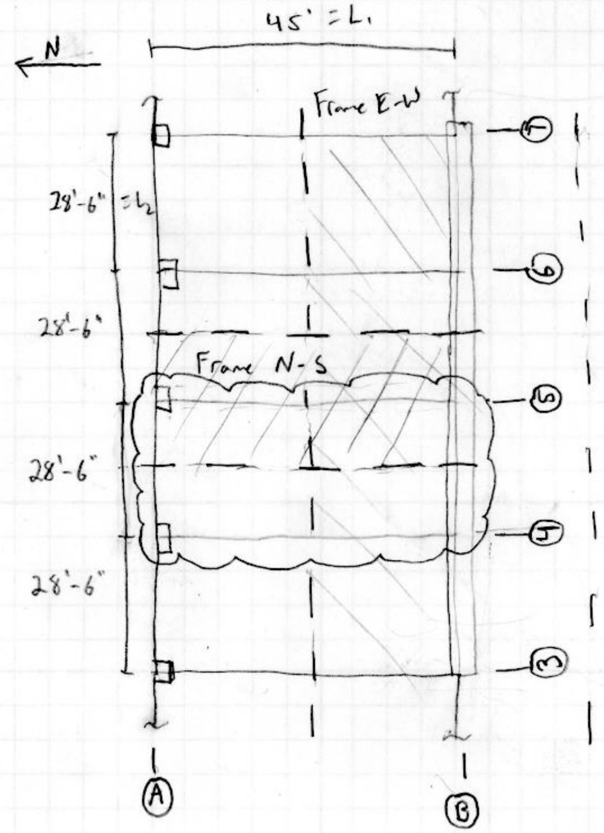
Supp	Vu	vu	Munb	Comb	Pat	GammaV	vu	Phi*vc
1	90.99	113.7	123.03	U2	S1	0.320	178.2	189.7
2	189.97	118.7	137.77	U2	S3	0.400	149.3	189.7
3	90.99	113.7	-123.03	U2	S3	0.320	178.2	189.7

Maximum Deflections:

Units: Dz (in)

	Frame			Column Strip			Middle Strip		
Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.122	-0.070	-0.192	-0.180	-0.103	-0.283	-0.064	-0.037	-0.101
2	-0.122	-0.070	-0.192	-0.180	-0.103	-0.283	-0.064	-0.037	-0.101

Appendix E: Two Way PT with Wide-Shallow slab beams



Loads:
Framing Dead Load = selfweight
Superimposed DL = 15 psf
Live Load = 80 psf office / corridors
2-hr fire rating

Materials: Normal weight = 150 pcf

$f'_c = 5{,}000$ psi
$f'_{ci} = 3{,}000$ psi

Rebar: $f_y = 60{,}000$ psi

PT: Unbonded tendons
½" φ, 7-wire strands, $A = 0.153\text{ in}^2$
$f_{pu} = 270$ ksi
Estimated losses = 15 ksi (ACI 18.6)
$f_{se} = 0.7(270\text{ksi}) - 15 = 174$ ksi (ACI 18.5.1)

$P_{eff} = A^* f_{se} = (0.153)(174) =$
26.6 kips / tendon

Determine Primary Slab Thickness

Long direction $L_1 = 45'$

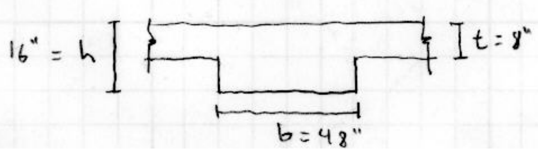
$L/h = 45$
$h = 45'(12)/45 = 12"$

Short direction: $L = 28.6'$
$L/h = 45'$
$h = 28.5(12)/45 = 7.6"$
∴ $h = 8"$

Loading
DL = Selfweight = 12"(150pcf) = 150psf
SIDL = 15psf
LL = 80psf

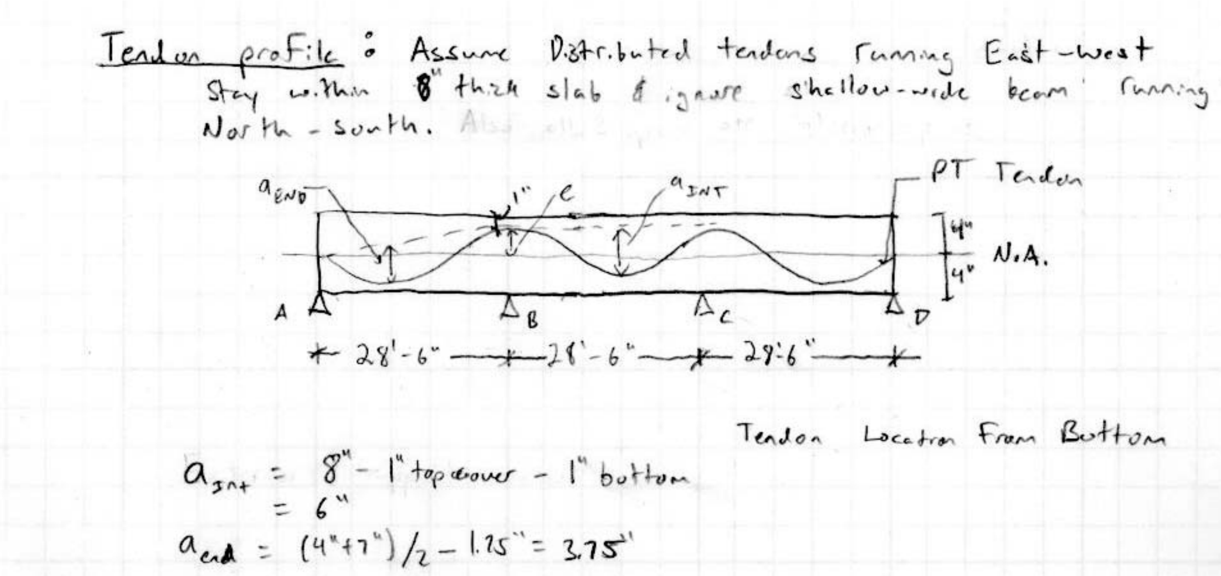
Selfweight = 8"(150) = 100psf
SIDL = 15psf
LL = 80psf

Wide - Shallow beams in Long direction:
dimensions controlled by short direction thickness.



$h \leq 2t$; $h \leq b/3$

$t = 8"$ ∴ $h = 16"$ & $b = 48"$



The post-tensioned floor slab for the typical span was designed using the equivalent frame method for a frame spanning east-west as well as a frame spanning north-south.

NOTE: It was assumed when designing the East-West frame that there was an adjacent continuous span where the shear wall and core openings are located. This assumption was made to simplify calculations, and because the large stiffness of the shear wall can be assumed to act like an adjacent floor span.

Materials		
Concrete	Normal Weight (pcf)	150
	f'c (psi)	5000
	f'ci (psi)	3000
Rebar	fy (psi)	60000
PT	Unbonded tendons	
	1/2" phi, 7 wire strands	
	Area (in^2)	0.153
	fpu(psi)	270000
	Estimate prestress losses (psi)	15000
	fse (psi)	174000
	Peff (psi)	26622

Design Parameters			
Allowable stresses		Class: U	
	At time of Jacking		
	Compression	1800	
	Tension	164.3	
	At service loads		
	Compression	2250	
	Tension	424.3	
Average precompression limits			
	P/A	125 min	
		300 max	
Cover Requirements		bottom	top
	Restrained slabs	0.75	
	Unrestrained slab	1.5	0.75

Design of East West:

Note: When designing the typical span it was assumed that the distributed tendons to be analyzed would terminate at grid lines 3 and 7 creating a 4 bay system with two exterior bays and two interior bays.

	L1	L2
Length	45	28.5
Preliminary Thicknes	12	7.6
Thickness	12	8
Self-Weight		100
Superimposed DL	15	15
Live Load	80	80
a int	8	4
a end	3.75	3.75

Design of East-West Interior Frame

Calculate Section Properties

Area	4320	
S	5760	

Prestress Force Required to Balance 60% of selfweight DL

wb	2.70	(klf)
P	877.2	

Check Precompression Allowance

# tendons	32

Actual force for banded tendons

Pactual	851.9	kips

Balance load for the end span

wb	2.6220	(klf)

Determine actual Precompression stress

Pactual/ A	197.2	psi > 125psi < 300psi

Check Interior Span Force

* Will work since width of interior is the same, but a int is bigger

wb (klf)	2.797	
wb/wdl	62.2	< 100% therefore ok

Check Slab Stresses

Dead Load			
	wdl	5.175	(klf)
	M-(Support 4)	449.8	(ft-k)
	M-(Support 5)	300.1	(ft-k)
	Mext+	324.5	(ft-k)
	Mint+	153.0	(ft-k)
Live Load			
	wll	2.408	(klf)
	M-(Support 4)	209.3	(ft-k)
	M-(Support 5)	139.6	(ft-k)
	Mext+	151.0	(ft-k)
	Mint+	71.2	(ft-k)
Total Balancing Moment			
	wb	2.732	(klf)
	M-(Support 4)	237.5	(ft-k)
	M-(Support 5)	158.5	(ft-k)
	Mext+	171.3	(ft-k)
	Mint+	80.8	(ft-k)

Stage 1: Stresses immediately after jacking (DL + PT)

Midspan Stresses (psi)

	Interior Span	
	ftop	-347.66
	fbottom	-46.74
	Exterior Span	
	ftop	-516.31
	fbottom	121.91

Support Stresses (psi)		
Support 4&6	ftop	245.09
	fbottom	-639.49
Support 5	ftop	97.93
	fbottom	-492.33

Stage 2: Stresses at service load (DL+LL+PT)

Midspan Stresses

	Interior Span	
	ftop	-496.0
	fbottom	101.6
	Exterior Span	
	ftop	-830.9
	fbottom	436.5
Support Stresses		
Support 4 & 6	ftop	681.1
	fbottom	-1075.5
Support 5	ftop	388.9
	fbottom	-783.3

Ultimate Strength

M1=P*e
e = 0in. At the exterior support
e = 3.0 in at the interior support

M1	212.976
Msec = Mbal-M1	24.488886

Mu=1.2Mdl + 1.6 Mll + 1.0Msec

Mu @ midspan	625.1
Mu @ support 4 & 6	-850.1
Mu @ support 5	-559.1

Determine minimum bonded reinforcement:

Interior span: ft = -496 psi > 2sqrt(f'c) = 141.4
Minimum positive moment reinforcement required!

y= ft/(ft+fc)h	1.359887109
Nc = M_{dl+ll}/ S *.5*y*l_2	363.7192171
As,min = Nc/ 0.5f_y	12.1239739
Distributed uniformly	0.269421642

Use #5 @ 12" oc Bottom (0.31 in^2)

Negative Moment Region:
As,min = 0.00075Acf
Interior supports:
Acf = max. (thickness * (trib length l1, l2)

Acf	4320
As,min	3.24

Use 11-#5 Top (3.41 in^2)

Exterior supports:

Acf=	4320
As,min=	3.24

Use 11-#5 Top (3.41 in^2)

Check minimum reinforcement if it is sufficient for ultimate strength

Interior Supports

Mn =($A_s f_y$+$A_{ps} f_{ps}$)(d-a/2)

	d=	7		
	Aps =	4.896		
	fps=	196867.6471		
	a= (Asfy+ Apsfps)/ (0.85*f'c*b)			
	aint=	0.504690196		
	phi Mn	586.1674318	<	-850.0591
Support 4&6	As, req'd	9.131289761		
	Distributed uniformly	0.20291755		

Use #5 @ 12" oc Bottom (0.31 in^2)

Midspan

Mn =(Asfy+Apsfps)(d-a/2)

d=	6.25		
Aps =	4.896		
fps=	195488.9706		
a= (Asfy+ Apsfps)/ (0.85*f'c*b)			
aint=	1.468019551		
phi Mn	395.9573601	<	625.1
As, req'd	6.712278754		
Distributed uniformly	0.14916175		

Use #5 @ 12" oc Bottom (0.31 in^2)

Design of North- South Frame:

Note: When analyzing the building in the North-South direction it was assumed that the middle bay, which is open for the core, is still there. This is because the large stiffness of the shear wall can be assumed to influence the exterior span's moment distribution much like that of an interior span.

	L1	L2
Length	28.5	45
Preliminary Thickness	7.6	12
Thickness	8	16
Self-Weight		100
Superimposed DL	15	15
Live Load	80	80
a int	4	12
a end	9.75	9.75
height effective		9.12

Design of North-South Interior Frame

Calculate Section Properties

Area (in^2)	3120
S (in^3)	4743.9

Prestress Force Required to Balance 60% of selfweight DL

wb (klf)	1.8300
P (kips)	570.1

Check Precompression Allowance

tendons 21

Actual force for banded tendons

Pactual 559.1

Balance load for the end span

wb 1.795

Determine actual Precompression stress

Pactual/ A 179.2 psi > 125psi
< 300psi

Check Interior Span Force

* Will work since width of interior is the same, but a int is bigger

wb (klf)	2.20864	
wb/wdl	72.4144262	< 100% therefore ok

Check Slab Stresses

Dead Load			
	w_{DL}	3.278	(klf)
	M-	663.7	(ft-k)
	Mext+	531.0	(ft-k)
	Mint+	165.9	(ft-k)
Live Load			
	w_{LL}	1.020	(klf)
	M-	206.5	(ft-k)
	Mext+	165.2	(ft-k)
	Mint+	51.6	(ft-k)
Total Balancing Moment			
	w_b (klf)	1.956	(klf)
	M-	396.1	(ft-k)
	Mext+	316.9	(ft-k)
	Mint+	99.0	(ft-k)

Stage 1: Stresses immediately after jacking (DL + PT)

Midspan Stresses (psi)

Interior Span

ftop	-348.4
fbottom	-10.0

Exterior Span

ftop	-720.6
fbottom	362.3

Support Stresses (psi)

ftop	497.6
fbottom	-856.0

Stage 2: Stresses at service load (DL+LL+PT)

Midspan Stresses (psi)

Interior Span

ftop	-479.0	
fbottom	120.6	

Exterior Span

ftop	-1138.6	
fbottom	780.2	Need Reinforcement

Support Stresses (psi)

ftop	1020.0	Need Reinforcement
fbottom	-1378.4	

Ultimate Strength

M1=P*e
e = 0in. At the exterior support
e = 7.0 in at the interior support

M1	326.1	(ft-k)
Msec = Mbal-M1	70.0	(ft-k)

Mu=1.2Mdl + 1.6 Mll + 1.0Msec

Mu @ midspan	936.5	(ft-k)
Mu @ support	-1056.8	(ft-k)

Determine minimum bonded reinforcement:

Exterior span: ft =1345.6psi > 2sqrt(f'c) = 141.4
Minimum positive moment reinforcement required!

y= ft/(ft+fc)h	3.70945086
Nc = M_{dl+ll}/ S *.5*y*l_2	1117.04166
As,min = Nc/ 0.5f_y	37.2347221
Distributed uniformly	1.30648148

As,min = 0.00075Acf

Interior supports:

Acf = max. (thickness * (trib length l1, l2)

Acf	4926.31579
As,min	3.69473684

Use 12-#5 Top (3.72 in^2)

Exterior supports:

Acf=	4926.31579
As,min=	3.69473684

Use 12-#5 Top (3.72 in^2)

Check minimum reinforcement if it is sufficient for ultimate strength

Interior Supports

Mn = (As*fy+Aps*fps)(d-a/2)

d=	15		
Aps =	3.213		
fps=	210610.644		
a= (Asfy+ Apsfps)/ (0.85*f'c*b)			
aint=	0.61807789		
phi Mn	989.850744	<	1056.84534
As, req'd/ft	0.77737376		

*As based upon Mu since its larger than phi Mn provided by minimum reinforcement

Exterior Supports

Mn = (As*fy+Aps*fps)(d-a/2)

d=	8
Aps =	3.213
fps=	198192.344
a= (Asfy+ Apsfps)/ (0.85*f'c*b)	
aext=	1.18125382
phi Mn	477.05779

Midspan

Mn = (As*fy+Aps*fps)(d-a/2)

d=	6.25		
Aps =	3.213		
fps=	195087.768		
a= (Asfy+ Apsfps)/ (0.85*f'c*b)			
aint=	3.45689161		
phi Mn	212.564028	<	936.5
As, req'd / ft	1.08689817		

*As based upon Mu since its larger than phi Mn provided by minimum reinforcement

Check Punching Shear:

Check Punching Shear:

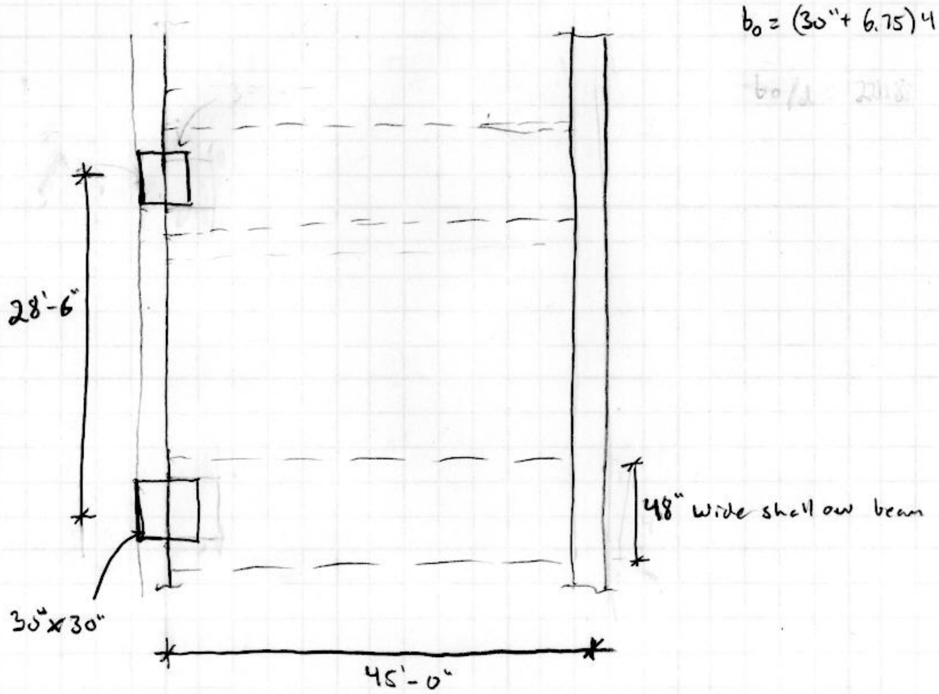
$\beta_c = \frac{30}{30} = 1$

$\alpha_s = 30$

t = 8" w/o slab beam
Exterior support for N-S Frame use #5's.

$d = 8" - 0.625" - 0.625" = 6.75"$

$b_o = (30" + 6.75)4 = 147"$



$W_u = 1.2(\frac{8}{12}(150) + 15) + 1.6(80) = 266$ psf

$V_u = w_u A = 266(28.5 \times 45 - (2.5)^2) = 339.5$ k

$$V_c = \begin{cases} 4\sqrt{f'_c}\, b_o d = 4\sqrt{5{,}000}\,(147")(6.75") = 286.7\text{ k} \\ \left(\frac{30}{\left(\frac{147}{6.75}\right)} + 2\right)\sqrt{5000}\,(147")(6.75") = 236.9\text{ k} \leftarrow \text{Controls} \end{cases}$$

$\phi V_c = 0.75(236.9\text{k}) = 177.7^k < V_u$

Need drop panels.

$$V_u \leq \phi V_c$$

$$339.5 = 0.75\left(\frac{30}{\frac{(30+d)4}{d}} + 2\right)\sqrt{5{,}000}\ \left(4(30+d)\right)\cdot d$$

$$(1000)\ \frac{6.401}{4} = \left(\frac{30d}{120+4d} + 2\right)\left(30d + d^2\right)$$

$$1600 = \frac{900d^2 + 30d^3}{120+4d} + 60d + 2d^2$$

$$192{,}000 + 6400d = 900d^2 + 30d^3 + 7200d + 480d^2 + 8d^3$$

$$0 = 38d^3 + 1380d^2 + 800d - 192{,}000$$

$d = 10.2"$ use $d = 10.5"$

$$\phi V_c = 355^k$$

Requires 11.75" thick drop panel

* Note: Since a 16" thick wide shallow beam is already pre
around column, drop panel isn't necessary.