UNION STATION EXPANSION AND RESTORATION

WASHINGTON DC

TECHNICAL REPORT II

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Structural Option

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EXECUTIVE SUMMARY

For this technical report, a study of alternative floor systems for the expansion to Union Station was done. A total of four systems, three new and the existing, were designed and compared to determine the viability for each one. Currently in Union Station, a post-tension design is used due to the long spans required throughout the building for the lower floors and the weight limit allowed on the soil. While this system is adequate to handle the criteria for the building, the author of the technical report looked at the following alternative floor systems for Union Station:

1) Pre-Cast Double Tee
2) Composite Floor Deck
3) Flat Plate with Drop Panels

During discussions between Professor M.K. Parfitt and the author, a different approach will be used for this technical report. Instead of using each floor system throughout all levels of Union Station, each new floor system was viewed at different levels of the building. This leads to the option of having a transfer level in Union Station where two of the new systems would meet and transfer the loads from one to the other. Since designing the transfer level is not part of the requirements for this technical report, the author and M.K. Parfitt concluded this could be a topic of interest to explore as the depth option for the thesis proposal.

Based on the typical bays used and redesigning the bay layouts to make the alternative floor systems to work, the author believes that the use of the pre-cast double tee would be beneficial on the lower floors and the composite system would work on the upper levels. Starting on page 8, descriptions of each system with advantages, disadvantages and how each system could work in Union Station. To view why the author selected pre-cast double tees and the composite system for the two floor systems, refer to the conclusion on page 16 of this report.

EXISTING STRUCTURAL SYSTEM

Foundation:

Union Station's expansion main foundation system consists of concrete piles and supportive columns that rest of spread footers. On the Track Level, the foundation is visible for passengers traveling on a locomotive or waiting on the platforms to notice.

All the columns and piles are located between the eight locomotive rail ways that are part of Union Station. Typical diameter size of the columns and the piles are 1 ½' and are spaced 22'-0" from each other (in a straight line between the rails).

The net soil bearing capacity for the site is 1000 PSF and each column and pile was designed to carry a typical load of 250 kips. Fine to coarse sandy clay fill is the typical soil located on the site for Union Station according to the geotechnical report. The columns and piles rest upon spread footers which either have a dimension of 6'-0" x 6'-0" x 2'-0" or 12'-0" x 12'-0" x 2'-0" (l x w x h).

Lateral System:

Union Station's lateral load system is composed of an ordinary reinforced concrete moment frame. Lateral loads, as well as the gravity loads, reach the foundation of Union Station by first traveling through the beams, then carry through the girders which connect to the columns. From there, all loads travel down in the columns to the ground level and then the piles and columns take all the loads into the spread footers. Not all beams and girders take part of the lateral system in Union Station. To view the beams and girders which do not act as part of the lateral system, refer to Appendix A, Figure 1.

It is important to note that the existing structure and the addition of Union Station do not share a lateral system. Steel Chevrons are used as the lateral system for the existing structure of Union Station. Since the expansion and the existing structure do not share a column line, an expansion joint was placed between column lines 7 and 7-1 (Refer to Appendix A, Figure 1).

Since the author will be looking at different floor systems in this technical report, the lateral system for each system would change. For this report, the author realizes will not take into account a new lateral system. In future technical reports and part of a thesis proposal, the author would investigate the design of a new lateral system with the selection of the new floor system(s).

Existing Floor System:

The typical floor system for the expansion to Union Station is a two-way post-tension cast-in-place concrete slab with a thickness of 7”. All the beams and girders are post-tension cast-in-place as well. In Union Station, the beams span a length of 63’-0”. The girders located in the expansion, carry the load from the beams to the columns and have a typical span of 24’-4” throughout the expansion. The concrete compressive strength for the slabs, beams, and girders is f'_c = 5000 psi. It is to be noted that the floor systems for the expansion and the existing structure for Union Station do not connect with each other.

For the Ground Level, a 6 ½” concrete slab was used for majority of the floor. A composite design located along the west elevation was utilized to help reduce the weight within the weakest are of the site. A 5” light weight concrete slab over 1 ½” gage LOK-Floor was used which makes the ground floor total thickness to be 6 ½”. Shear studs sized at ¾” x 4 ½” were used in the composite floor design. The typical member size for the beams is W27x84 which span 63’-0” and tie into a W33x118 girder. The girders tie into the concrete columns that are part of the foundation system.

There are two typical bay sizes located in the expansion of Union Station, 63’-0” x 27’-6” and 63’-0” x 40’-0”. Since the tracks running through Union Station were the major consideration in the design as well as the bus terminal, the use of long spans was concluded as the best approach for the design. For this report, the bay size of 63’-0” x 40’-0” will be analyzed in order to obtain results that can be applied throughout the rest of the structure. Figure 2 in Appendix A shows the area used to analyze the existing structure.

Structural Plan Layout:

As mentioned in the executive summary, each new floor system will be analyzed for different levels throughout Union Station. For the levels consisting of the bus terminal, mezzanine level, and the first floor, the use of the pre-cast double tee floor system can be utilized due to the high floor-to-floor levels (Refer to Figure 3, Appendix A). For the remaining levels, the use of either the composite metal deck or flat plate with drop panels can be used with the use of office space and parking.

For the pre-cast double tee floor system, the existing spans in the east/west direction are used, but the spans in the north/south direction will be reduced in half to 31'-6". The decision in the reduction of the span is to ease the double-tees from being overloaded from the required loads. This decision will also help in the location of columns and walls for the floor system.

To achieve a layout for the composite floor deck and the flat plate with drop panels, revisions of the upper floors were done by the author. Since parking and office space are the main uses on the upper floors, smaller spans can be achieved in both situations. In Appendix A, Figures 4 and 5 show a basic typical structural floor plan for each of the two systems. It should be noted that the plans do not include any areas for the elevators and stairs. If the author decides to take the route of designing the transfer level for the proposal, more detailed plans in architecture and structural design would be looked at.

CODE AND DESIGN REQUIREMENTS

The following two tables represent codes used for the design of Union Station by the engineers in practice and the codes used by the author of this technical report. Since Union Station was designed with older edition of codes, values for loads and member sizes could be off depending if any significant changes were made for the new codes.

CODES & REFRENCES (USED BY DESIGN TEAM)
"DC Building Code 2003"
"International Building Code 2000" (as amended) – International Code Council
"DC Building Code Supplement 2000" (DCMR 12A)
"Building Code Requirements for Structural Concrete (ACI 318-02)" – American Concrete Institute
"ACI Manual of Concrete Practice 2003" – American Concrete Institute
"CRSI Handbook", 2002 Edition – Concrete Reinforcing Steel Institute
"PCI Design Handbook, Fifth Edition" – Precast/Prestressed Concrete Institute
"PTI Design Manual, Fourth Edition" – Post Tensioning Institute
"Manual of Steel Construction" – American Institute of Steel Construction, Inc.
"ASCE 7-05", Minimum Design Loads for Buildings and Other Structures – American Society of Civil Engineers

Table 1: Codes & References Used by Design Team

CODES & REFRENCES (USED IN TECHNICAL REPORT II)
"International Building Code 2006"
"PCI Design Handbook, Sixth Edition" – Precast/Prestressed Concrete Institute
"Manual of Steel Construction, Thirteenth Edition" – American Institute of Steel Construction, Inc.
"ASCE 7-05", Minimum Design Loads for Buildings and Other Structures – American Society of Civil Engineers
"Building Code Requirements for Structural Concrete (ACI 318-08)" – American Concrete Institute
"Vulcraft Steel Roof and Floor Deck"

Table 2: Codes & References Used in Technical Report II

Deflection Criteria:

Total Deflection: l/240

Live Load Deflection: l/360

Construction Load Deflection: l/360

GRAVITY LOADS

The following chart shows the gravity loads were determined from ASCE 7-05 by the engineers in practice and by the author of this technical report. All loads were used by the author that the engineers used and the author used additional loads that felt were important in include in the calculations. Since additional loads were used by the author, loads and members sizes could have increased in some areas of the structure.

GRAVITY LOADS			
Dead Loads:	Weight	Used By Design Team	Used By Author
Lightweight Concrete	120 pcf	Yes	Yes
Steel	490 pcf	Yes	Yes
M.E.P.	10 psf	Yes	Yes
Fiishes & Misc.	5 psf	No	Yes
Live Loads:			
Office	50 psf	Yes	Yes
Stairs	100 psf	Yes	Yes
Landings	100 psf	Yes	Yes
Lobbies	100 psf	Yes	Yes
Mechanical	150 psf	Yes	Yes
Parking	50 psf	Yes	Yes
Partition	10 psf	No	Yes

Table 3: Gravity Loads

EXISTING FLOOR SYSTEM I: POST-TENSIONING

Description:

Post-tensioning is a method of reinforcing (strengthening) concrete with high-strength steel strands or bars, typically referred to as tendons. A typical tendon is composed of 7, ½" Ø with a strength of 270 ksi. Each tendon is placed prior to the concrete is poured in the form work and one side is anchored. Once the concrete is placed and reaches a certain strength, each tendon is jacked from the lose end until it becomes a tight strand. Additional reinforcement is used with the tendons to prevent the concrete from failing if any problems arise within the concrete. Figure 1 shows tendons and other reinforcement resting in place before concrete is poured.

Figure 1: Image of Tendons

Provided by Suncoast-PT

Advantages:

Post-tensioning allows longer clear spans, thinner slabs, and fewer beams throughout a building. Thinner slabs mean less concrete is required, which can reduce the cost of the building's structure significantly. Reduction in a building's weight versus a conventional concrete building can be achieved with the use of post-tensioning. This reduces the foundation load and can be a major advantage in seismic areas or in places where the soil cannot support a heavy building.

Disadvantages:

Unless a building's design has long spans or needs to be lighter than normal, post-tensioning should not be considered as the design. When jacking the tendons to meet the required strength, it is important to jack at a consistent rate. If jacked improperly or not placed correctly before the concrete is poured, a tendon can snap and rupture through the concrete. This problem can not only cause a delay in the completion of the building, but can also be dangerous to a life around the tendon when it snaps.

Design for Union Station:

In the design phase for Union Station, realizing there are trains traveling through and stopping, a bus terminal on the ground floor, and parking on the upper levels, post-tensioning is considered a good choice for a floor system for the entire building. Taking advantage of the long spans, reduction of amount of columns, and slab thickness post-tensioning can offer, the use of this slab system was a fine call by the designers. The typical bay in Union Station has around 20 tendons spanning in the east-west direction of the building. Since long spans exist in the structure, post-tensioned beams and girders were used as well to help the slab from reaching a critical deflection. One main disadvantage with using this system within Union Station is how difficult the system is to install. The time for a post-tension system for erection is longer than most of the other structural systems used in practice and more labor is required to install this floor system.

Appendix B contains calculations for the existing post-tension structure. The author calculated the balanced load and the effective force used in the existing bays. From the calculations, the results are within 15% from the designer. One major cause of the difference in results could the assumptions the author used. Another is the possibility of a calculation error somewhere in the process. Since there are post tension beams and girders that tie into the slab and columns, the author recognizes that the knowledge at the time of this technical report is not sufficient enough to continue on with checking the system. For the third technical report, the author will use this report to check the existing structure.

ALTERNATE FLOOR SYSTEM I: PRECAST DOUBLE TEES

Description:

Double tees come in variety of sizes and can span long distances. Typically the most common sized used is 12'-0". A double tee is supported by either an inverted "T" beam or "L" beam, which is used for an edge. From either the "T" or "L" beam, the loads travel to columns which can be spaced further than what most typical layouts are designed for. Typically, ½ ø, 270k tendons are used as the reinforcement bars for all the precast members. Figure 2 shows a general layout of a double tee connection to an inverted "T" beam.

Figure 2: Double Tee System

Provided By FRS

Advantages:

Using precast concrete double tee beams can give you several advantages for a floor system. The most beneficial use of precast is its quick and steady installation. There is no down time on the job site required for concrete to be formed, poured, finished and set. Double tee products arrive on-site and can be placed immediately. Precast products provide the consumer with a high quality product that is fabricated in a controlled working environment and can be installed year round. No additional fire protection is required for a double tee since it is incorporated by the plant that creates the members.

Disadvantages:

While using precast double tees can save you time in erection, specialized labor must used to install the products. Depending on where you project is located and the time given for erection, finding the correct group of installers might be difficult. The depths of the stems on each precast member can cause problems for a project that requires a high floor to floor height. Also, a topping slab might be required as well. Not only does this reduce the floor height, but you also must account for continuity in the topping as well.

Design for Union Station:

Using PCI Design Handbook, 6th Edition, the designed floor system incorporates the use of precast 8'-0" wide double tees spanning a length of 40'-0". The selected tees utilize lightweight concrete and are 24" deep with a 2" normal weight concrete topping for continuity of the floor surface. This gives an overall depth of 26" with a 4" finished slab depth. The tees are reinforced using (4) ½ ø, 270k tendons within 5,000 psi concrete. The 4" thick slab depth is adequate for the 2 hour required fire rating. The inverted "T" beams are designed as 34IT36 beams and the edge "L" beams are designed as 20LB32 beams. Refer to Appendix C for calculations regarding the double tee floor system.

Double Tees can be a good choice for a floor system for the ground level, mezzanine level, and first floor since the floor heights are higher than normal. This will not create problems with the depths of the double tees for each of the floors. Since double tees can span a long length, ground floor can still achieve the architecture of the bus terminals as well for the track level. The only major issue that can be noticed as of right now is that the placement of the columns and walls for the loads that come from the double tees beams. A further investigation in the future would be necessary to see where issues would arise.

ALTERNATE FLOOR SYSTEM II: COMPOSITE FLOOR DECK

Description:

Composite floor deck is complied of three main components; metal deck, concrete, and a steel member. The load path for this system starts with the concrete and the metal deck. To have the loads travel from the floor to the steel members that act as the beams in the composite design, steel shear studs are used as transfer points in the system. Once the loads reach the beams, they travel through the girders and then to the columns. The amount of shear studs used on each beam is determined by the thickness of the slab (concrete and metal deck) as well as the span used in a bay. The size of the metal deck, beam and girders depends on the load used in a single bay. Figure 3 shows a section of a composite floor deck.

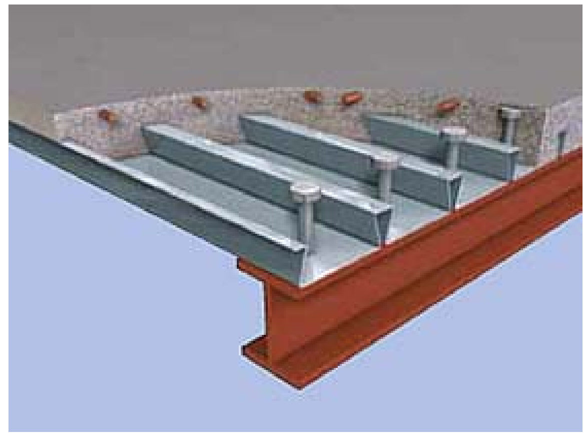

Figure 3: Composite Deck

Provided by EPIC

Advantages:

A composite floor system is one that can be erected quickly and easy to construct in the field. The system also comes with a fire rating that can either be sprayed on or if the engineer designs according to ANSI/UL 263, no additional fire proofing is required. In office areas, this is ideal for open column free tenant spaces and also works well as an acoustical barrier. Construction for this floor system is quick which helps reduce the cost the time to complete the structure down.

Disadvantages:

One major concern when using a composite system is possible lower floor to ceiling heights. If a beam or a girder becomes deeper than expected, the use of the system would not be practical. In the case that the steel members are deep, then the structural system would be heavier than other systems. A heavy building could cause problems in a seismic region or a site that has weak soil.

Design for Union Station:

Using the Vulcraft floor deck catalog, a 2VL16 metal deck with a total thickness of 5 ¼" was determined as the adequate design. The author took advantage of using ANSI/UL 263 by using 3 ¼" of lightweight concrete (110 pcf) on top of a 2" thick metal deck. This results in no additional fire proofing required for the floor system. Each beam designed in the bay looked are a W16x31 and span a length of 30'-0". A total of twenty ¾" Ø shear studs will be used to transfer the load from the floor to the beams. The beams connect to a steel girder which was sized as a W21x62, span a length of 39'-0" and uses 38 shear studs. Refer to Appendix D for calculations regarding the double tee floor system.

For the upper levels of Union Station, this would be an adequate system to use for both the office spaces and the parking. A thin slab works for the upper levels which increases the floor height. The composite design also has low transfer of vibrations between levels. This would be beneficial for the levels with parking above the office spaces to prevent sound transfer from each level. The floor system is easy to construct and is time saving during construction. One concern for this floor system to consider is areas that would have a significant deep member. When the author redesigned the floors as a preliminary design, there is an area that spans a length of 42'-0". This bay has the potential of the girder being deeper than normal (Refer to Appendix A, Figure 4). For future investigation, the author would consider redesigning the layout to reduce the length of the bay to prevent the chance of a deep member.

ALTERNATE FLOOR SYSTEM III: FLAT PLATE WITH DROP PANELS

Description:

A flat plate floor system is essentially a flat slab floor with no beams in the structure. The drop panels are a thicken portion of the slab which can either be in a rectangular or circular region centered on the columns. Each drop panel helps increase the shear strength of the floor system in the critical region around the column and provide increased effective depth for the steel in the region of high negative bending moment over the support. Typically flat slabs are used for live loads of 100 psf or more and for spans up to 30 feet.

Figure 4: Flat Plate W/ Drop Panels

Provided by Univ. of Cal. Berkeley

Advantages:

Flat plate with drop panels can manage a significant amount of live load with a relatively small slab thickness. The thin section of flooring allows for a higher floor-to-floor dimension. A flat plate also fits well with a grid of columns and bays. Since the concrete is dense, no additional fireproofing is needed for the floor system. The floor system has above average as an acoustical barrier for vibration in the floors. Concrete needs minimal formwork and only basic field labor. Columns can also be made of reinforced concrete which would lead to the use of shear walls to handle the lateral forces.

Disadvantages:

Use of a concrete flat slab needs a rather exact ratio of column spans, which doesn't always guarantee an open plan. In fact, this ratio requires a smaller sized bay which could mean more columns, which can lead to a heavier building. The increased amount of concrete requires an increased amount of reinforcement. Since the columns are also reinforced concrete there is a complicated construction of intermingled reinforcement where columns and floors meet which can extend construction time.

Design for Union Station:

Using the design requirements from ACI 318-08, a 7 ½" slab composed of lightweight concrete (120 pcf) with a 3" drop panel was determined to be sufficient to carry the required loads. No additional fire proofing for the floor system is required because the required depth coverage is used from ACI. Number 5 bars were used as the reinforcement steel throughout the 30'-0" x 19'-6" bay analyzed in this technical report. The assumption of using an 8'-0" x 8'-0" drop panel and a 24" x 24" column was used and the author realizes the dimensions of the panel could be smaller. Since (16) #5 bars were determined as the reinforcement for the middle strip of the slab, the author believes there are sufficient bars in the slab. Looking at a higher bar size can result in a decrease of bars used. Further investigations would be needed to determine the most efficient flat slab with drop panel system for Union Station. Refer to Appendix E for calculations regarding the double tee floor system.

For the upper levels of Union Station, a flat plate with drop panels would work efficiently in the parking areas. The columns could be placed accordingly to allow for the maximum amount of spaces. The system can work for the office areas, but the office areas would not have an open floor area. Vibrations would not be transferred between levels since the slab acts as an acoustical barrier. Since there are more columns for this system, the weight of the building has the potential of becoming heavier. Due to the weakness of the soil on the site, this system on the upper levels could cause problems with the foundation. As stated in the above paragraph, further investigation would have to be conducted in order to determine the efficiency of the flat plate with drop panels.

CONCLUSION

Throughout this technical report, each system was looked by the advantages, disadvantages, and how each system could work for Union Station. While the existing post-tension system works adequately in the entire structure, the author believes it is not the best system for Union Station. Since the major concern for Union Station was the location of the tracks, the use of long spans was determined to be used throughout the building. Respecting the concept used by the design team, the author suggests using two different systems in Union Station.

Using the integration of double tees for the lower levels and a composite steel system for the upper levels, the author believes this would be a valuable alternative design approach to Union Station. Both of the systems are lighter than the existing and have a lower cost for the material and installation. The double tees would not have an effect on the floor heights since they will be used on the ground floor, mezzanine level, and first floor due to the high floor heights already. All the uppers with the composite steel system would have a reduced thickness of the slab allowing for an increased floor height.

The author believes the use of flat plate with drop panels would not be the best system for the upper floors in Union Station. While the cost of the system is the least expensive of all four systems and the slab thickness is the close to the original, the weight is higher than the others. This is a concern for the site because the soil is not capable of carry a heavy structure.

Below, Table 4 summarizes the comparisons for all four systems. As mentioned before in this technical report, if this is decided upon the author to investigate these two systems as a proposal, more detailed drawings, location of columns, bays, transfer level, and new lateral system would have to be designed.

COMPARISON OF FLOOR SYSTEMS				
	Post-Tension (Existing)	Double Tee	Composite Steel	Flat Plate w/ Drop Panels
Location of System	Entire building	Lower Levels	Upper Levels	Upper Levels
Depth of Slab	7"	4"	5 1/4"	7 1/2"
Depth of Members	Up to 48" For Beams & Girders	26" (Including 2" Topping)	W16x31 For Beams & W21x62 For Girders	3" Drop Panels
Weight	95 psf	90 psf	67 psf	100 psf
Addittional Fire Proofing	No	No	No	No
Floor to Floor Height	Varies with Levels	Increased	Incresead	Decreased
Vibrations	Minimal	Minimal	Minimal	Minimal
Cost of Floor System	$26.17	$20.02	$21.30	$16.50
Fesibility	Existing	Yes	Yes	No

Table 4: Comparison of Floor Systems

APPENDIX A: PLANS & SECTIONS

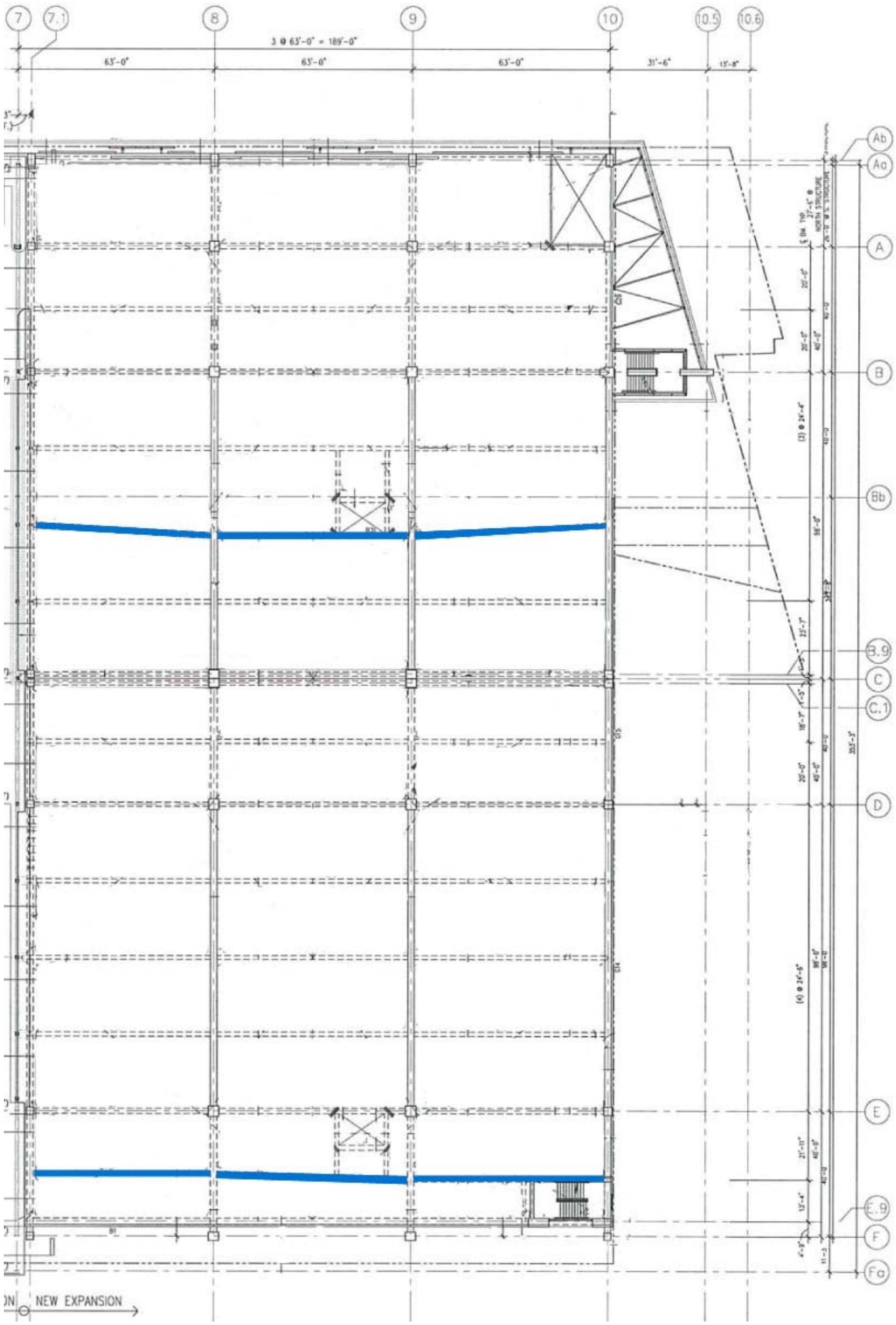

Figure 1: Non-Lateral Members on Typical Floor

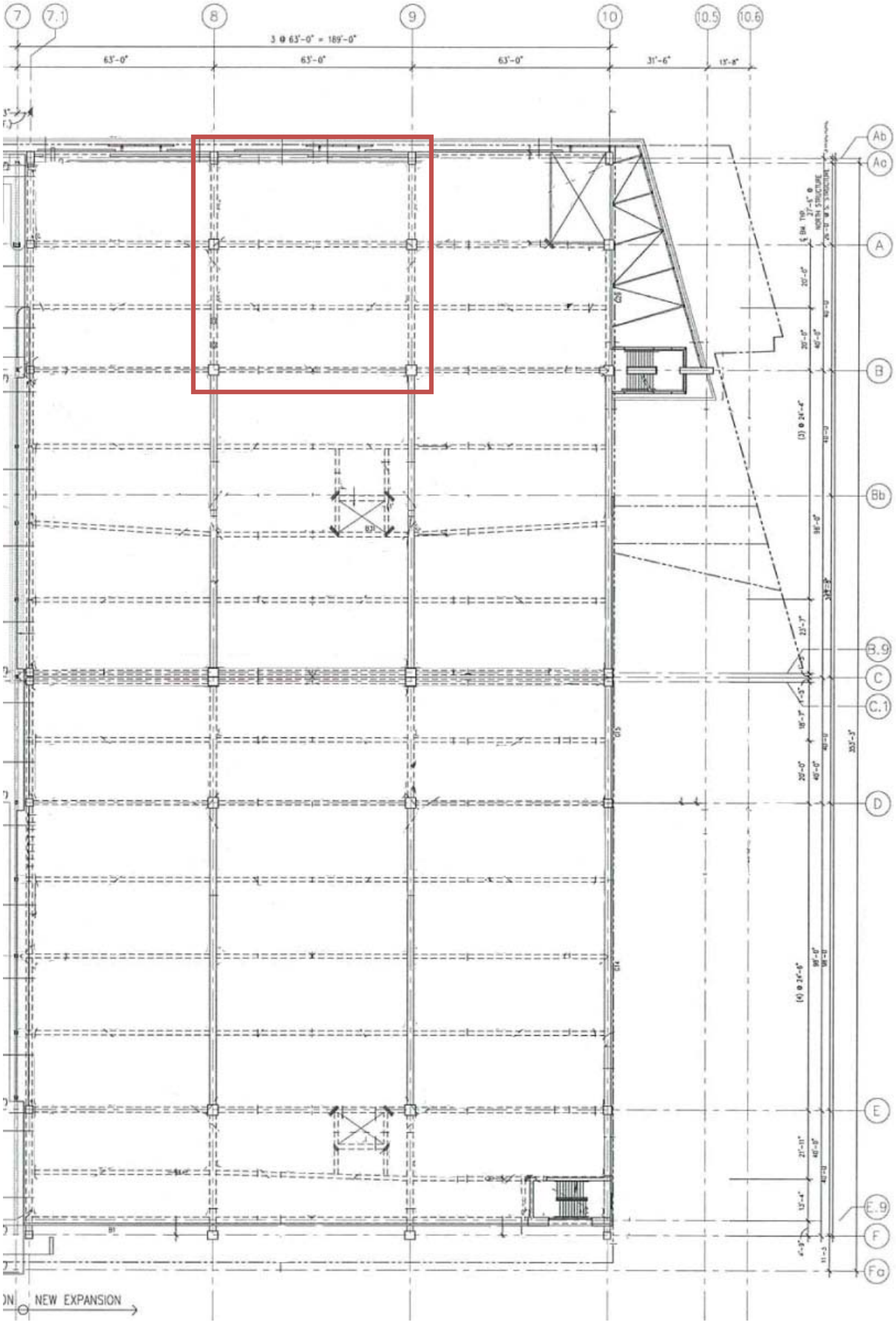

Figure 2: Area Used To Design Existing Post-Tension Design

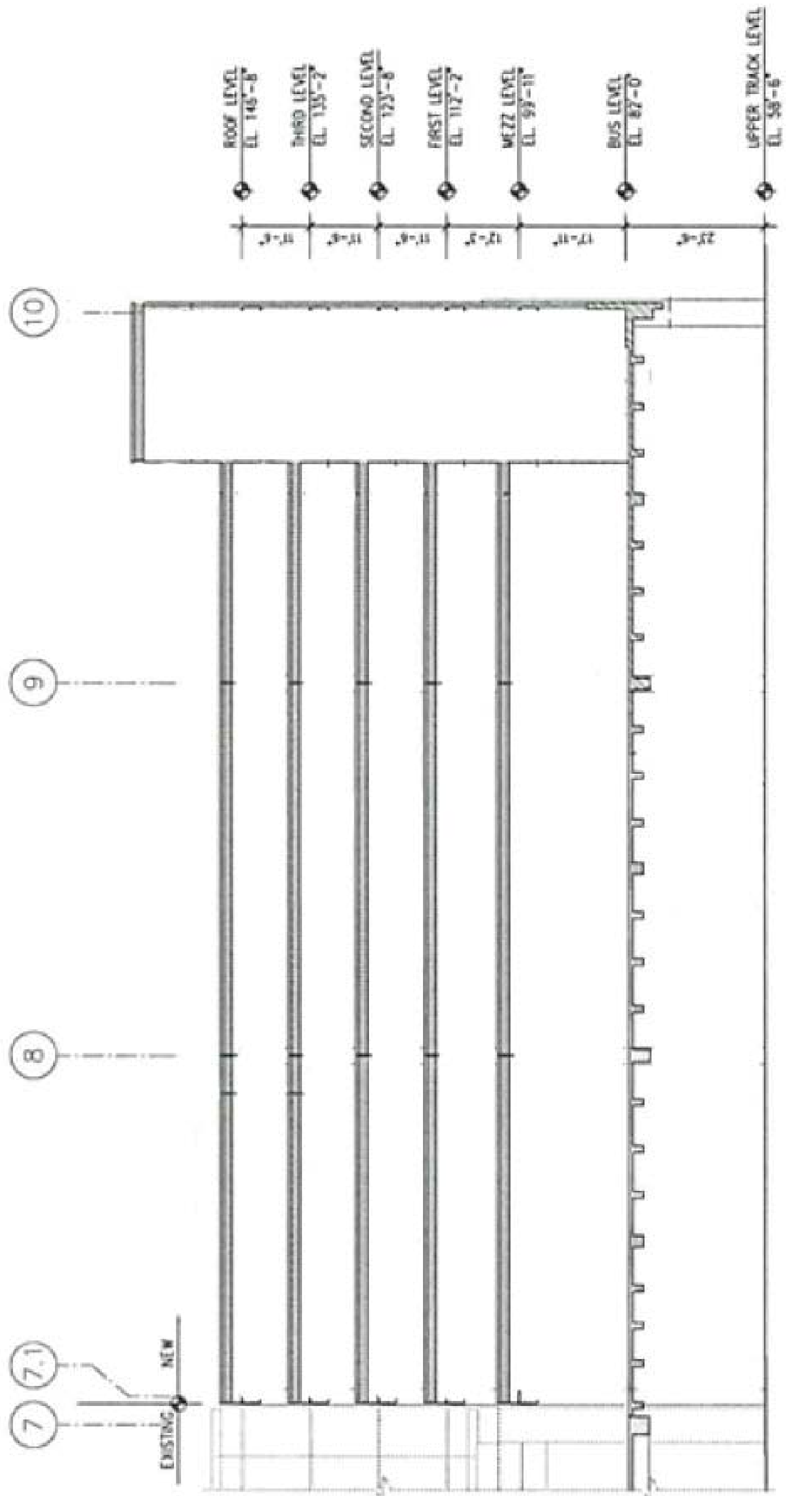

Figure 3: Section of Union Station

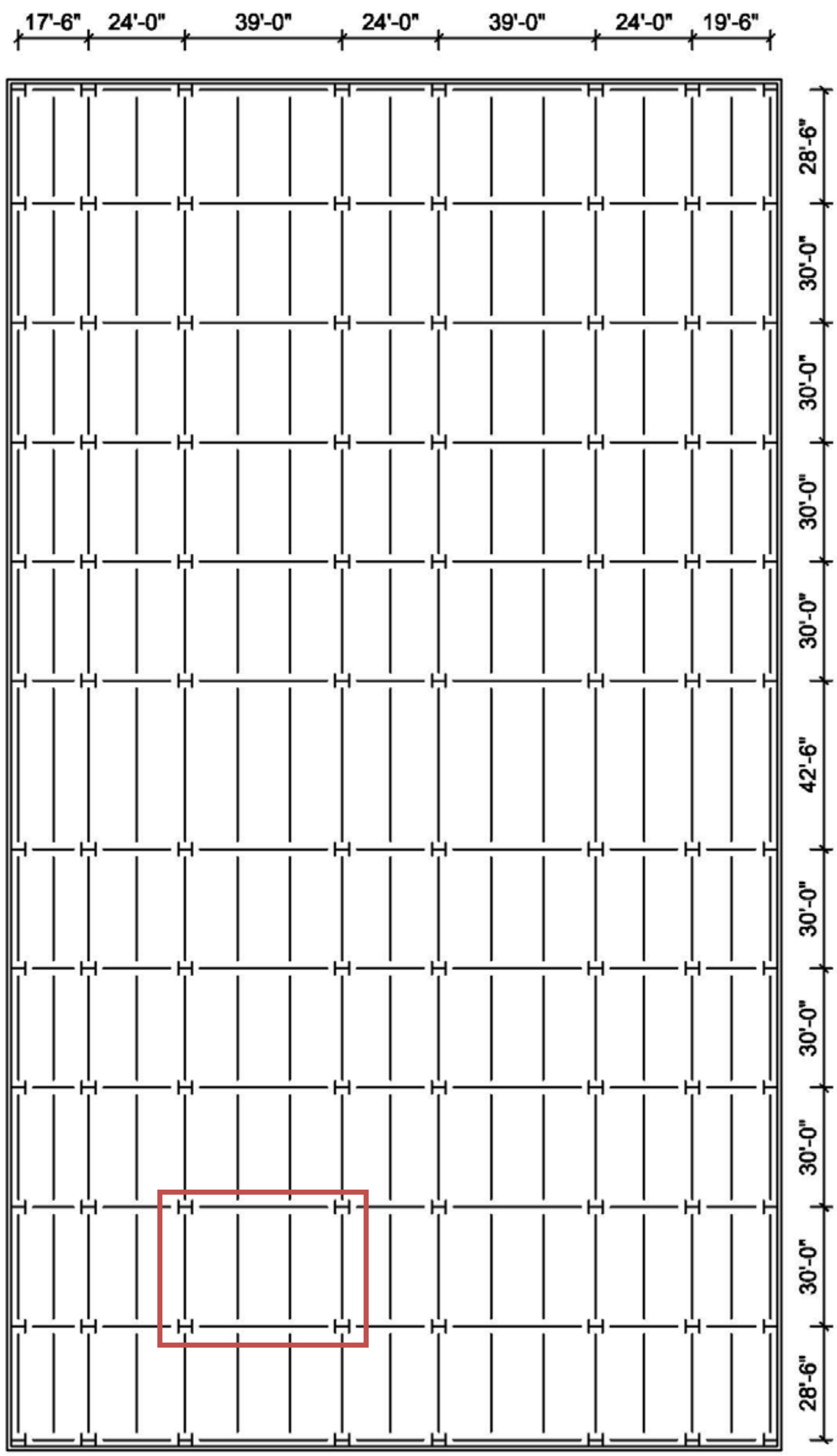

Figure 4: Composite Floor Deck Layout

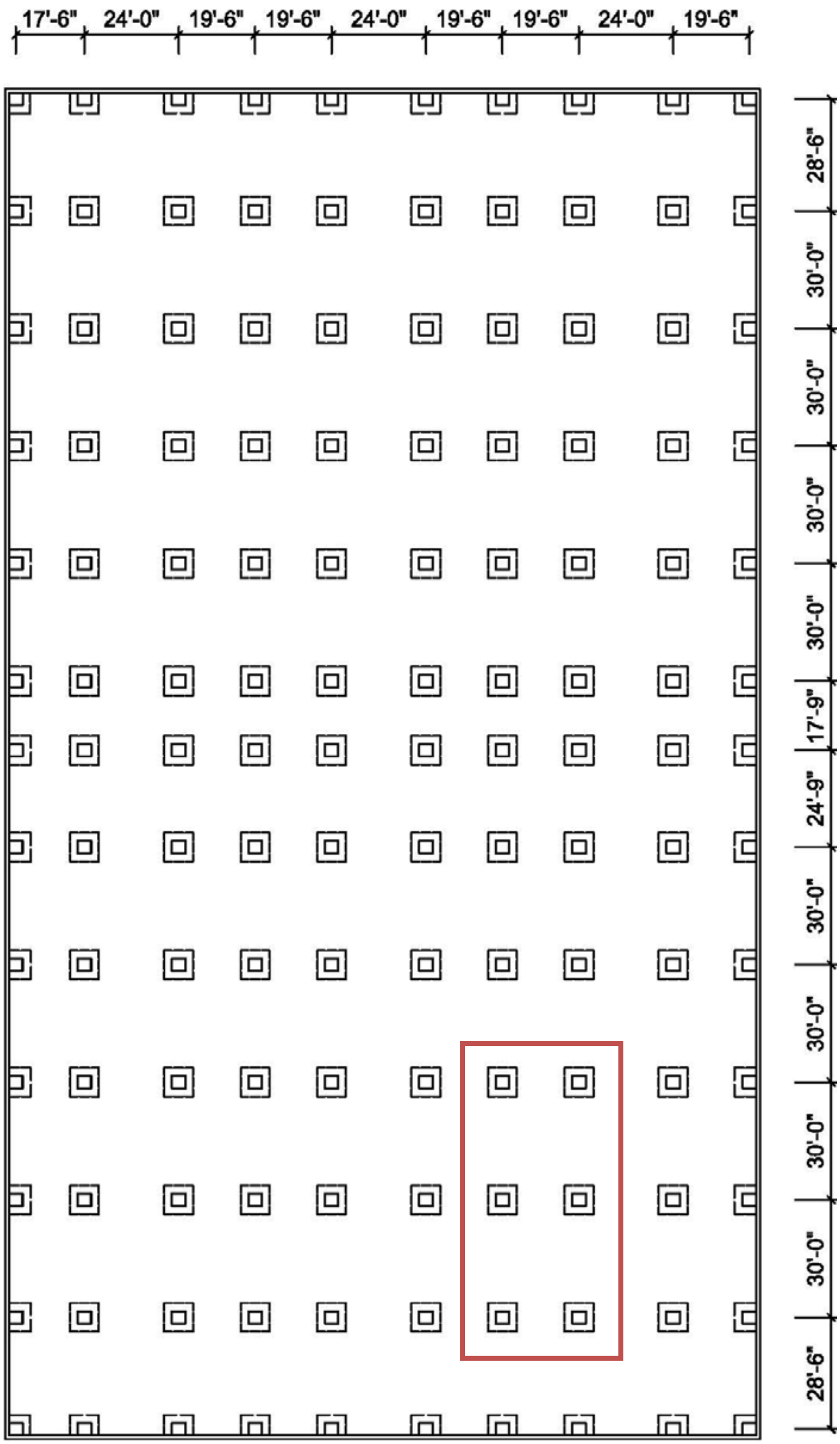

Figure 5: Flat Plate w/ Drop Panel Layout

APPENDIX B: POST-TENSION CALCULATIONS

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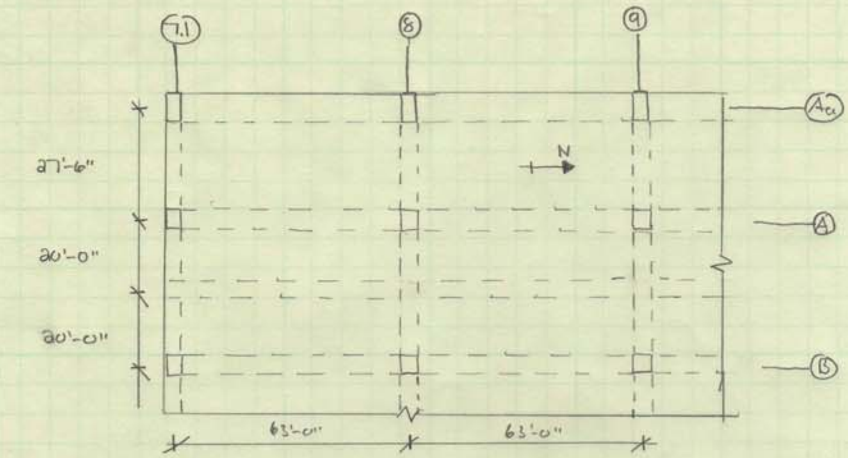

COLUMNS => STORY HEIGHT : 11'-6"

LOADS: FRAMING DEAD LOAD = SELFWEIGHT
SUPERIMPOSED DEAD LOAD = 15 psf [FINISH, MEP]
LIVE LOAD = 50 psf [PARKING]

* 2 HOUR FIRE-RATING

MATERIALS: CONCRETE => LIGHTWEIGHT CONCRETE 120 pcf
f'_c = 5,000 psi
f'_{ci} = 3,000 psi

REBAR => f_y = 60,000 psi

PT => UNBONDED TENDONS
1/2" ϕ, 7-WIRE STRANDS, $A = 0.153\ in^2$
f_{pu} = 270 ksi

$f_{se} = 0.7 f_{pu} = 0.7(270\,ksi) - 14\,ksi \approx 175\,ksi$ [ACI 18.5.1]

$P_{eff} = A(f_{se}) = 0.153\,in^2(174\,ksi) = 26.8^k/\text{tendon}$

DETERMINE SLAB THICKNESS

$\frac{\text{SPAN}}{\text{DEPTH}} = \frac{L}{45}$

$\frac{27.5ft(12\ in/ft)}{45} = 7.33in \therefore$ **USE 7in SLAB**

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DESIGN OF EAST-WEST INTERIOR FRAME

LOADING: DL = SELFWEIGHT = $\frac{7in(ft)\ 120\ pcf}{12in}$ = 70 psf

SIDL = 15 psf
LL = 50 psf

$A = bh = 20ft(12in/ft)(7in) = 1680\ in^2$

$S = \frac{bh^2}{6} = \frac{20ft(12in/ft)(7in)^2}{6} = 1960 in^3$

• At Time Of Jacking => $f'_{ci} = 3{,}000$ psi
Compression $= 0.6f'_{ci} = 0.6(3000\ psi) = 1800$ psi
Tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi

• At Service Loads => $f'_c = 5000$ psi
Compression $= 0.45f'_c = 0.45(5000\ psi) = 2250$ psi
Tension $= 6\sqrt{f'_c} = 6\sqrt{5000\ psi} = 424$ psi

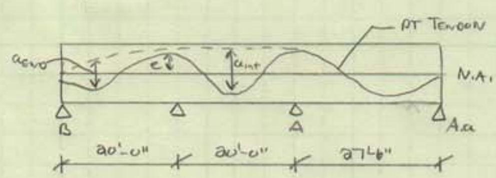
• According To ACI 318-08, 18.12.4, $P/A > 125$ psi

Target Load Balance: $0.75\ w_{DL}$

$0.75 w_{DL} = 0.75\left[\frac{7in(120pcf)}{12in/ft}\right] = 52.5$ pcf

• To Achieve 2Hr Fire Rating, Carbonate Aggregate
↳ Restrained Slabs = 3/4" Bottom
Unrestrained Slabs = 1 1/2" Bottom
= 3/4" Top

Tendon Profile:



Tendon Ordinate	Tendon (CG) Location*
Exterior Support - Anchor	4.0 in
Interior Support - Top	7.0 in
Interior Span - Bottom	1.0 in
End Span - Bottom	1.75 in

$a_{int} = 7.0 - 1.0\,in = 6.0\,in$

$a_{end} = \left[\frac{4\,in + 7\,in}{2}\right] - 1.75\,in = 3.75\,in$

$w_b = 0.75\,w_{DL} = 0.75(70\,psf)(63\,ft) = 3310\,plf = 3.31\,^k/ft$

$P = \frac{w_b L^2}{8[a_{end}]} = \frac{3.31\,^k/ft\,(20\,ft)^2}{8[3.75/12]} = 530^k$

$\#\,\text{Tendons} = \frac{530^k}{26.8\,^k/\text{tendon}} = 19.7 = \underline{\underline{20\ \text{Tendons}}}$

$P_{ACTUAL} = 20\ \text{Tendons}\,(26.8^k/\text{tendon}) = 536^k$

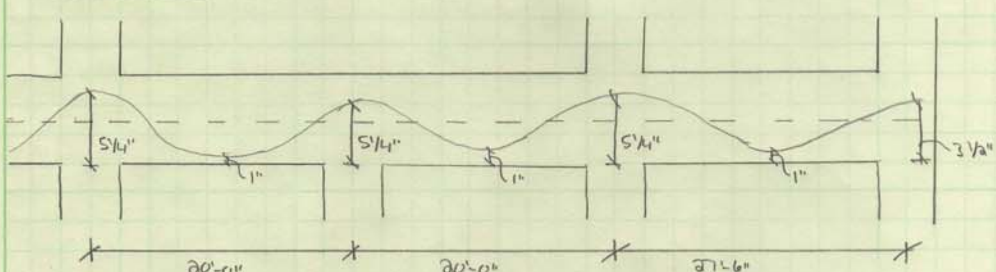
$w_b = \left(\frac{536^k}{530^k}\right)(3.31^k/ft) = 3.35\,^k/ft$

$\frac{P_{actual}}{A} = \frac{536^k\,(1000)}{1680\,in^2} = 319\,psi > 125\,psi \therefore$ OKAY

CHECK INTERIOR SPAN FORCE:

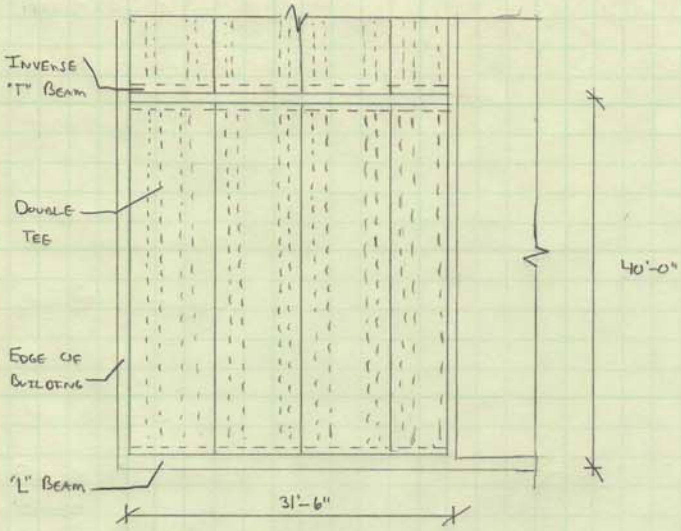
$P = \frac{3.31\,^k/ft\,(20\,ft)^2}{8[6/12]} = 331 < 536 \therefore$ LESS FORCE REQUIRED IN INTERIOR SPAN

$P_{eff} = 536^k$



APPENDIX C: DOUBLE TEE CALCULATIONS

Union Station Expansion | Technical Report II | Double Tee Design | 1



+ Using PCI Handbook, 6th Edition

Assumptions: $f'_c = 5000$ psi
$f_{pu} = 270{,}000$ psi
1/2" ϕ Strands, $A = 0.153 \text{ in}^2$
Lightweight Concrete

Loads: Superimposed Dead: 25 psf [MEP, Finish, Partition]
Live Load = 50 psf [Office]

$W_u = 1.2D + 1.6L$

$W_u = 1.2(25\text{psf}) + 1.6(50\text{psf})$

$W_u = 110$ psf

+ Select 8LDT24+2 [2" Topping Results In Total Depth Of 26"]
88-S Strand Pattern
40'-0" Span

151 psf > 110 psf ∴ Okay

Use 8LDT24+2 For Double Tee

Design Precast Inverted "T" Beam

Weight of Double Tee => 65 psf

Dead Load: 25 psf
+65 psf
90 psf

Live Load: 50 psf

$W_u = 1.2D + 1.6L$

$W_u = 1.2(90\,psf) + 1.6(50\,psf)$

$W_u = 188\,psf$

$188\,psf(38.67\,ft) = 7270\,plf$

* Select 34IT36
  24½-S Strands
  32'-0" Span

8016 plf > 7270 plf ∴ Okay

Use 34IT36 As Inverted "T" Beam

Design Edge "L" Beam

$188\,psf\left(\dfrac{38.67\,ft}{2}\right) = 3634\,plf$

* Select 20LB32
  14½-S Strands
  32'-0" Span

4001 plf > 3634 plf ∴ Okay

Use 20LB32 As "L" Beam

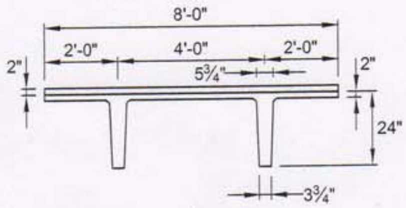
Strand Pattern Designation

1 2 8 - D 1
- No. of strand (12)
- S = straight D = depressed
- No. of depression points
- Diameter of strand in 16ths

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.

Key
196 – Safe superimposed service load, psf
1.2 – Estimated camber at erection, in.
1.5 – Estimated long-time camber, in.

DOUBLE TEE
8'-0" x 24"
Lightweight Concrete



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

Section Properties

	Untopped		Topped	
A =	401	in.²	–	
I =	20,985	in.⁴	29,857	in.⁴
y_b =	17.15	in.	19.94	in.
y_t =	6.85	in.	6.06	in.
S_b =	1,224	in.³	1,497	in.³
S_t =	3,064	in.³	4,927	in.³
wt =	418	plf	520	plf
DL =	40	psf	65	psf
V/S =	1.41	in.		

8LDT24

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

Strand Pattern	y_s(end) in. / y_s(center) in.	Span, ft																								
		32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80
68-S	4.00	196	170	149	131	115	102	90	80	72	64	57	51	45	40	36	32	28								
	4.00	1.2	1.3	1.4	1.5	1.6	1.6	1.7	1.8	1.8	1.9	1.9	1.9	1.9	1.8	1.7	1.6	1.4								
		1.5	1.6	1.7	1.8	1.9	1.9	2.0	2.0	2.0	2.0	1.9	1.8	1.7	1.5	1.2	0.8	0.4								
88-S	5.00			194	171	152	135	121	108	97	87	79	71	64	58	52	47	43	38	35	31	28				
	5.00			1.8	1.9	2.1	2.2	2.3	2.5	2.6	2.7	2.7	2.8	2.8	2.9	2.9	2.8	2.8	2.6	2.5	2.3	2.0				
				2.3	2.4	2.6	2.7	2.8	2.9	2.9	3.0	3.0	2.9	2.9	2.8	2.6	2.5	2.2	1.9	1.5	0.9	0.3				
108-S	6.00					183	164	147	132	119	107	97	87	78	70	64	58	53	48	44	40	36	33	29	26	
	6.00					2.4	2.5	2.7	2.9	3.0	3.2	3.3	3.4	3.5	3.6	3.6	3.7	3.7	3.6	3.6	3.5	3.3	3.1	2.9	2.5	
						3.0	3.2	3.4	3.5	3.6	3.7	3.8	3.8	3.8	3.7	3.6	3.5	3.3	3.1	2.8	2.5	2.0	1.5	0.8	0.0	
128-S	7.00											110	99	89	80	72	65	59	53	49	44	40	37	34	31	28
	7.00											3.7	3.8	3.9	4.0	4.1	4.2	4.2	4.2	4.2	4.1	4.0	3.9	3.7	3.5	3.2
												4.3	4.4	4.4	4.4	4.3	4.2	4.0	3.8	3.6	3.3	2.9	2.4	1.9	1.3	0.6
128-D1	11.67																83	76	69	62	57	51	46	42	38	34
	3.25																4.8	4.9	5.0	5.1	5.1	5.1	5.0	4.9	4.6	4.3
																	5.1	5.0	4.9	4.7	4.4	4.0	3.6	3.0	2.3	1.4
148-D1	12.86																							51	46	42
	3.50																							5.9	5.8	5.7
																								4.6	4.1	3.4

8LDT24 + 2

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

Strand Pattern	y_s(end) in. / y_s(center) in.	Span, ft																						
		28	30	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74
48-S	3.00	178	150	126	107	90	76	64	54	45	38	31	25												
	3.00	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	1.0	0.9												
		0.6	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	-0.2	-0.6												
68-S	4.00			198	170	147	127	111	96	84	73	63	55	47	40	34	29								
	4.00			1.2	1.3	1.4	1.5	1.6	1.7	1.7	1.8	1.9	1.9	1.9	1.9	1.9	1.8								
				1.2	1.2	1.2	1.3	1.2	1.2	1.1	1.0	0.9	0.7	0.4	0.1	-0.3	-0.8								
88-S	5.00					197	172	151	133	117	103	91	80	71	61	52	45	37	31	25					
	5.00					1.8	1.9	2.1	2.2	2.3	2.5	2.6	2.7	2.7	2.8	2.8	2.9	2.9	2.8	2.8					
						1.8	1.8	1.9	1.9	1.9	1.8	1.7	1.6	1.4	1.1	0.8	0.5	0.0	-0.5	-1.1					
108-S	6.00							186	164	146	129	115	102	89	76	65	56	48	41	34	29				
	6.00							2.4	2.5	2.7	2.9	3.0	3.2	3.3	3.4	3.5	3.6	3.6	3.7	3.7	3.7				
								2.3	2.3	2.4	2.4	2.3	2.2	2.1	1.9	1.6	1.3	0.9	0.5	-0.1	-0.7				
128-S	7.00													104	90	78	68	58	49	42	35	30			
	7.00													3.7	3.8	3.9	4.0	4.1	4.2	4.2	4.2	4.2			
														2.6	2.4	2.2	1.9	1.5	1.1	0.5	0.0	-0.7			
128-D1	11.67																		71	62	53	46	39	32	26
	3.25																		4.8	4.9	5.0	5.1	5.1	5.1	5.0
																			1.9	1.5	0.9	0.3	-0.5	-1.3	-2.3

Strength is based on strain compatibility; bottom tension is limited to $12\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.
Shaded values require release strengths higher than 3500 psi.

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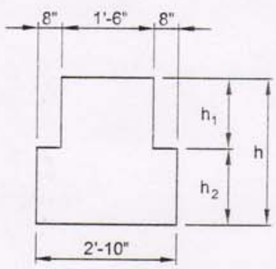
INVERTED TEE BEAMS

Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Section Properties								
Designation	h in.	h_1/h_2 in./in.	A $in.^2$	I $in.^4$	y_b in.	S_b $in.^3$	S_t $in.^3$	wt plf
34IT20	20	12/8	488	16,082	8.43	1,908	1,390	508
34IT24	24	12/12	624	27,825	10.15	2,741	2,009	650
34IT28	28	16/12	696	44,130	11.79	3,743	2,722	725
34IT32	32	20/12	768	65,856	13.50	4,878	3,560	800
34IT36	36	24/12	840	93,616	15.26	6,135	4,514	875
34IT40	40	24/16	976	128,656	16.85	7,635	5,558	1,017
34IT44	44	28/16	1,048	171,157	18.58	9,212	6,733	1,092
34IT48	48	23/16	1,120	221,906	20.34	10,910	8,023	1,167
34IT52	52	36/16	1,192	281,504	22.13	12,721	9,424	1,242
34IT60	60	44/16	1,336	439,623	25.78	17,053	12,847	1,392

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key

7822 – Safe superimposed service load, plf.
0.4 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Desig-nation	No. Strand	y_s(end) in. y_s(center) in.	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
34IT20	148-S	2.29		7822	6253	5092	4209	3522	2977	2537	2177	1879	1629	1417	1237	1081				
		2.29		0.4	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.2				
				0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1				
34IT24	178-S	2.59			9221	7524	6233	5229	4432	3789	3262	2826	2461	2151	1887	1660	1463	1291	1140	1007
		2.59			0.4	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.2
					0.2	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
34IT28	208-S	3.00					8641	7271	6183	5306	4589	3994	3495	3073	2713	2403	2134	1900	1694	1513
		3.00					0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3	1.3
							0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1
34IT32	238-S	3.48						9589	8174	7032	6097	5323	4674	4124	3655	3252	2902	2597	2329	2093
		3.48						0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3
								0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2
34IT36	248-S	3.50								9223	8016	7015	6176	5466	4860	4338	3886	3492	3146	2840
		3.50								0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2
										0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
34IT40	308-S	4.40									9720	8510	7497	6639	5907	5277	4731	4254	3836	3467
		4.40									0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.3
											0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4
34IT44	308-S	4.40											9362	8307	7406	6630	5958	5372	4857	4403
		4.40											0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.1
													0.2	0.2	0.2	0.2	0.2	0.3	0.2	0.2
34IT48	338-S	4.73													8963	8037	7234	6533	5919	5376
		4.73													0.8	0.8	0.9	1.0	1.0	1.1
															0.3	0.3	0.3	0.3	0.3	0.3
34IT52	368-S	5.22														9503	8564	7745	7026	6392
		5.22														0.8	0.9	0.9	1.0	1.0
																0.3	0.3	0.3	0.3	0.3
34IT56	398-S	5.59																	8269	7532
		5.59																	1.0	1.0
																			0.3	0.3
34IT60	408-S	6.00																	9564	8721
		6.00																	0.8	0.9
																			0.3	0.3

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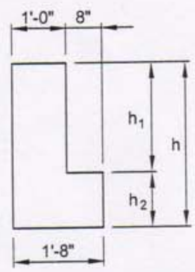
L-BEAMS

Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Designation	h in.	h_1/h_2 in./in.	A in.^{2}	I in.^{4}	y_b in.	S_b in.^{3}	S_t in.^{3}	wt plf
20LB20	20	12/8	304	10,160	8.74	1,163	902	317
20LB24	24	12/12	384	17,568	10.50	1,673	1,301	400
20LB28	28	16/12	432	27,883	12.22	2,282	1,767	450
20LB32	32	20/12	480	41,600	14.00	2,971	2,311	500
20LB36	36	24/12	528	59,119	15.82	3,737	2,930	550
20LB40	40	24/16	608	81,282	17.47	4,653	3,608	633
20LB44	44	28/16	656	108,107	19.27	5,610	4,372	683
20LB48	48	32/16	704	140,133	21.09	6,645	5,208	733
20LB52	52	36/16	752	177,752	22.94	7,749	6,117	783
20LB56	56	40/16	800	221,355	24.80	8,926	7,095	833
20LB60	60	44/16	848	271,332	26.68	10,170	8,143	883

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key
6566 – Safe superimposed service load, plf.
0.3 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Desig-nation	No. Strand	y_s(end) in. / y_s(center) in.	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
			Span, ft																	
20LB20	98-S	2.44	6566	5131	4105	3345	2768	2318	1961	1674	1438	1243	1079							
		2.44	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2							
			0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2							
20LB24	108-S	2.80	9577	7495	6006	4904	4066	3414	2896	2479	2137	1854	1617	1416	1244	1097	969			
		2.80	0.3	0.3	0.4	0.5	0.5	0.6	0.7	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2			
			0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.1	0.0	0.0			
20LB28	128-S	3.33			8228	6733	5596	4711	4009	3443	2979	2595	2273	2000	1768	1567	1394	1243	1110	992
		3.33			0.4	0.4	0.5	0.6	0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.3
					0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	0.0
20LB32	148-S	3.71				8942	7446	6281	5356	4611	4001	3495	3071	2712	2406	2143	1914	1715	1540	1386
		3.71				0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3
						0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1
20LB36	168-S	4.25					9457	7988	6823	5883	5113	4476	3941	3489	3103	2771	2483	2231	2011	1816
		4.25					0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.3
							0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2
20LB40	188-S	4.89						9812	8386	7235	6293	5513	4858	4305	3832	3425	3073	2765	2495	2257
		4.89						0.4	0.5	0.6	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.1	1.2
								0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3
20LB44	198-S	5.05								8959	7803	6845	6042	5363	4783	4284	3851	3474	3143	2850
		5.05								0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.1	1.1
										0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
20LB48	218-S	5.81									9226	8100	7158	6360	5678	5092	4584	4140	3751	3408
		5.81									0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.1
											0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3
20LB52	238-S	6.17										9634	8521	7578	6774	6082	5482	4958	4499	4094
		6.17										0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0
												0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
20LB56	258-S	6.64											9954	8860	7927	7124	6427	5820	5287	4816
		6.64											0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.0
													0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
20LB60	278-S	7.33													9089	8173	7380	6688	6080	5544
		7.33													0.7	0.7	0.8	0.9	0.9	1.0
															0.3	0.3	0.3	0.3	0.3	0.3

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APPENDIX D: COMPOSITE FLOOR SYSTEM CALCULATIONS

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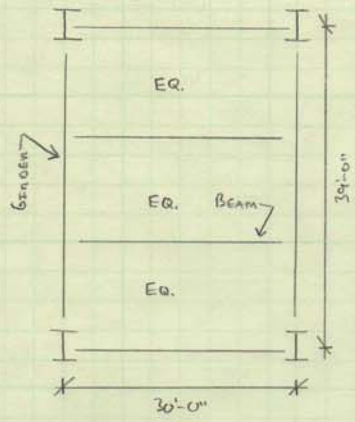

+ Using
+ Using AISC Stee Manual, 13th Edition

Design Of Metal Deck

+ According To ANSI/UL 263, To Achieve A 2-HR Fire Rating While Using Lightweight Concrete, 3¼" Thickness Of Concrete Above The Metal Deck Must Be Used [110 pcf]

Dead Load: 25 psf
Live Load: 50 psf

f'_c = 3,000 psi
f_y = 50 ksi

$W_u = 1.2D + 1.6L$

$W_u = 1.2(25\,psf) + 1.6(50\,psf)$

$W_u = 110\,psf$

+ Try A 5¼" Slab Depth W/ 16 Gage 2" Deck [2VL16]

Allowable Load: 128 psf > 110 psf ∴ Okay

Max Unshored Span: 13.25 ft > 13 ft ∴ Okay

Use A 5¼" Slab Depth W/ 16 Gage 2VL16 Deck

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DESIGN OF BEAM

$M_u = \frac{110\,psf\,(13ft)(30ft)^2}{8\,(1000)}$

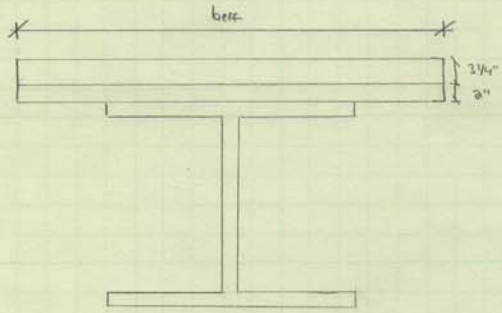
$M_u = 161$ k-ft

+ Assume $a = 1.25$ in

$Y_2 = 5.25in - 1.25in = 4.0in$

+ Using Table 3-19 In The Steel Manual
↳ Try W16x31 ⇒ 313 k-ft > 161 k-ft ∴ Okay [At P.N.A. 6]

$\Sigma Q_n = 164^k$



$b_{eff} \leq \begin{cases} Span/2 = 30ft(12)/2 = 180\,in \\ 1/4\,Span = 7.5ft(12) = 90\,in \end{cases}$

$b_{eff} = 90in$

$a = \frac{164^k}{0.85(3ksi)(90in)}$

$a = 0.71$ in < $a_{assumed} = 1.25$in ∴ A Conservative Approach Was Made

Shear Studs = $\frac{\Sigma Q_n}{Q_n}$ +Assume 3/4" Shear Studs, Deck ⊥, 1 Weak Stud Per Rib

+ From Table 3-21 In Steel Manual
$Q_n = 17.2$

Shea Studs = $\frac{164^k}{17.2^k/studs}$ = 9.53 = 10 (2) = 20 studs

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Design of Beam Cont'd

Check Deflections

W16x31 → Bare Steel ⇒ $\phi M_n = 203^k$

$w_{conc+deck} = 42\,psf$

$w_{beam} = 0.031\ ^k/ft$

$w_{construction\ live} = 20\ psf$

$w_{DL} = (0.042\ ksf)(13\ ft) + 0.031\ ^k/ft = 0.575\ ^k/ft$

$w_{LL} = 0.020\ ksf(13\ ft) = 0.26\ ^k/ft$

$w_u = 1.2w_D + 1.6w_L = 1.2(0.575) + 1.6(0.26)$

$w_u = 1.11\ ^k/ft$

$M_u = \frac{w_u l^2}{8} = \frac{1.11\ ^k/ft\,(30\ ft)^2}{8} = 125\ k\text{-}ft < \phi M_n \therefore$ Okay

Deflection During Construction: [Only Look At Dead Weight]

$\Delta < 1\ in$ [$l/360$]

$\Delta = \frac{5 w_D (L_n)^4 (1728)}{384 E I_{required}}$

$1\ in = \frac{5(0.575)(30\ ft)^4(1728)}{384(29{,}000)(I_{required})}$

$I_{required} = 361\ in^4$

$I_{W16x31} = 375 > I_{required} \therefore$ Okay

Live Load Deflection:

$w_{LL} = 50\ psf(13\ ft) = 0.65\ ^k/ft$ [$l/360$]

• From Table 3-20 In The Steel Manual, Use $I_{LB} = 719\ in^4$ For W16x31

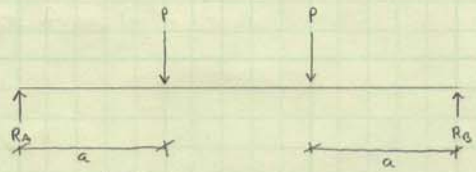
$\Delta = \frac{5(0.65\ ^k/ft)(30\ ft)^4(1728)}{384(29{,}000)(719)} = 0.568\ in < 1\ in \therefore$ Okay

• For Total Deflection: [$l/240$]

$\Delta = \frac{5(1.11\ ^k/ft)(30\ ft)^4(1728)}{384(29{,}000)(719)} = 0.961 < 1.5\ in \therefore$ Okay

Use W16x31 (20) For Composite Beam

DESIGN OF GIRDER



$P = \left[\dfrac{110\,psf(13ft)(30ft)}{2(1000)}\right](2)$

$P = 42.9^k$

* USING TABLE 3-23 IN THE STEEL MANUAL,

$R_A = R_B = P = 42.9^k$

$M_{max} = M_u = Pa$

$M_u = 42.9^k(13ft)$

$M_u = 558$ k-ft

* ASSUME $a = 1.25$ in

$Y_2 = 4.0$ in

* USING TABLE 3-14 IN THE MANUAL
  ↳ TRY W21x62 ⇒ 792 k-ft > 558 k-ft ∴ OKAY [AT P.N.A. #6]

$\Sigma Q_n = 317^k$

$b_{eff} \le \begin{cases} \text{SPACING}/2 = 30ft(12in/ft)/2 = 180\,in \\ \text{SPAN}/4 = 39ft(12in)/4 = 117\,in \end{cases}$

$b_{eff} = 117\,in$

$a = \dfrac{317}{0.85(3ksi)(117in)} = 1.063\,in < a_{assumed} = 1.25\,in$ ∴ A CONSERVATIVE ASSUMPTION WAS MADE

$\#\text{ SHEAR STUDS} = \dfrac{\Sigma Q_n}{Q_n}$ * ASSUME 3/4" SHEAR STUDS, DECK //, $w_r/h_r < 1.5$

* FROM TABLE 3-21 IN THE MANUAL
  $Q_n = 17.1$

$\#\text{ SHEAR STUDS} = \dfrac{317^k}{17.1^k/\text{STUDS}} = 18.5 \approx 19(2) = 38$ STUDS

DESIGN OF GIRDER CONT'D

CHECK DEFLECTIONS

W21 x 62 ⇒ BARE STEEL ⇒ $\phi M_n = 540\,k\text{-}ft$

$W_{CONC + DECK} = 42\ psf$

$W_{GIRDER} = 0.062\ k/ft$

$w_{DL} = (0.042\ ksf)(30\,ft) + 0.062\ k/ft = 1.32$

$W_{CONSTRUCTION\ LIVE} = 20\ psf$

$w_{LL} = 0.020\ ksf\,(30\,ft) = 0.60\ k/ft$

$w_u = 1.2(1.32) + 1.6(0.60) = 2.54\ k/ft$

$M_u = \frac{w_u l^2}{8} = \frac{2.54\ k/ft\,(39\,ft)^2}{8} = 483\,k\text{-}ft < \phi M_n \therefore$ OKAY

DEFLECTION DURING CONSTRUCTION

$\Delta = l/360 = 39(12)/360 = 1.3\ in$

$\Delta = \frac{5(1.32)(39\,ft)^4(1728)}{384\,(29{,}000)(1330)} = 1.43\ in$

* SINCE CONSTRUCTION LOADS SHOULDN'T CONTROL, HAVE A 3/4" CAMBER WITHIN THE MEMBER

$1.43\ in - 0.75\ in = 0.68\ in < l/360$ and $< 1"$ ∴ OKAY

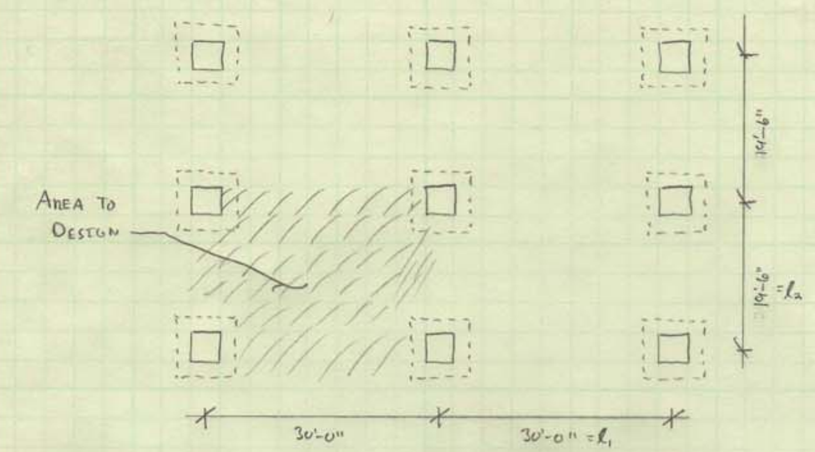
LIVE LOAD DEFLECTION [$l/360$]

$w_{LL} = 50\ psf\,(30\,ft) = 1.5\ k/ft$

$\Delta = \frac{5(1.5\ k/ft)(39)^4(1728)}{384(29{,}000)(1330)} = 2.02\ in - 3/4"$ CAMBER $= 1.27\ in < l/360$ ∴ OKAY

USE W21x62 (35) WITH 3/4" CAMBER

APPENDIX E: FLAT PLATE WITH DROP PANELS CALCULATIONS



ASSUMPTIONS: f'_c = 4,000 psi
f_y = 60,000 psi
LIGHTWEIGHT CONCRETE = 120 pcf => FOR SLAB

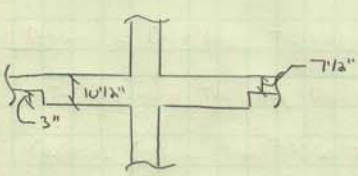
COL. SIZE: 24" x 24"
DROP PANEL SIZE: 8'x8'
DROP PANEL THICKNESS = 3"

DETERMINE SLAB THICKNESS

• ACCORDING TO ACI 318-08, USING THE LIMITS IN TABLE 9.5(c), DEFLECTION CRITERIA WILL BE TAKEN CARE OF

$$t_{min} = \frac{l_n}{36} = \frac{(30ft - 8ft)(12"/ft)}{36} = 7.33 in$$

USE 7½" THICK SLAB



DEAD LOADS: $\frac{7.5 in (120 pcf)}{12 in/ft}$ = 75 psf
25 psf [PARTITION, FINISH, MEP]

LIVE LOADS: 50 psf [OFFICE]

$W_u = 1.2D + 1.6L = 1.2(100 psf) + 1.6(50 psf) = 200 psf$

FRAME A: $M_o = \frac{1}{8} w_u l_2 l_n^2 = \frac{1}{8}(0.200 ksf)(19.5 ft)(30ft - 2ft)^2 = 382$ ft-k

FRAME B: $M_o = \frac{1}{8} w_u l_1 l_n^2 = \frac{1}{8}(0.200 ksf)(30ft)(19.5ft - 2ft)^2 = 230$ ft-k

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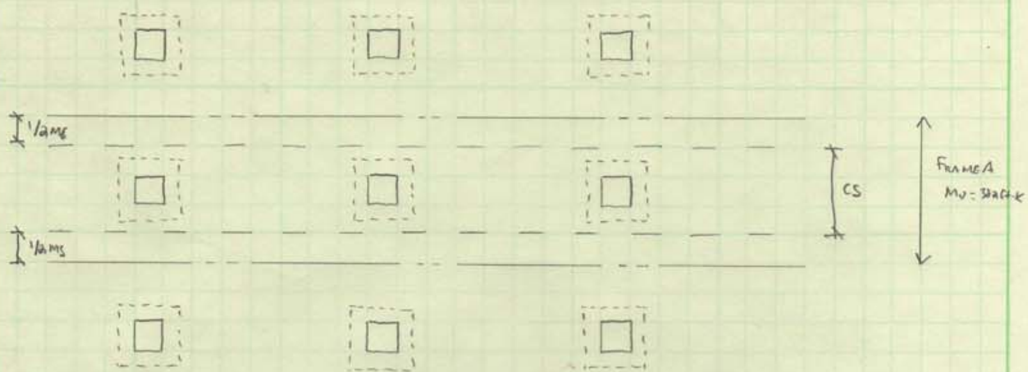
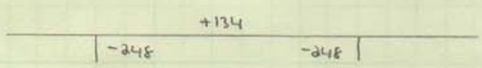


*FROM PAGE 294 OF ACI 318-08

FRAME A:

INTERIOR SPAN
$M^- = 0.65M_o = 0.65(382\text{ ft-k}) = -248\text{ ft-k}$
$M^+ = 0.35M_o = 0.35(382\text{ ft-k}) = +134\text{ ft-k}$



TRANSVERSE OF DISTRIBUTION OF LONGITUDINAL MOMENT

1.) NEGATIVE MOMENT @ INTERIOR SUPPORT

$\ell_2/\ell_1 = 19.5\text{ft}/30\text{ft} = 0.65$
$\beta_t = 0$ } NO BEAMS
$\alpha \ell_2/\ell_1 = 0$

INTERPOLATION

ℓ_2/ℓ_1	0.50	0.65	1.0
$\beta_t = 0$	75%	75%	75%

-248 ft-k → 75% TO CS = $0.75(-248) = -186$ ft-k
→ 25% TO MS = $0.25(-248) = -62$ ft-k

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2) POSITIVE MOMENT @ INTERIOR SUPPORT

l_2/l_1	0.50	0.65	1.0
$\beta_t = 0$	60%	[60%]	60%

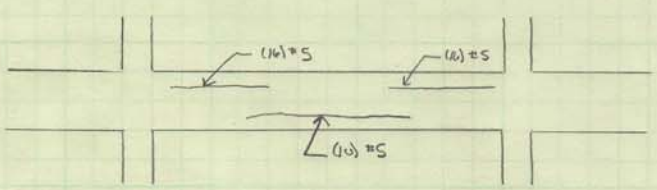
+134 A-K → 60% TO CS = 0.60(+134) = 80 A-K
→ 40% TO MS = 0.40(+134) = 54 A-K

FRAME A: TOTAL WIDTH = 19.5 ft, CS = 9.75 ft, MS = 9.75 ft

TOTAL MOMENT	−248	+134	−248
CS SLAB	−186	+80	−186
MS SLAB	−62	+54	−62

DESIGN OF SLAB REINFORCEMENT IN COL. STRIP [ASSUME #5 BAR]

ITEM NO.	DESCRIPTION	INTERIOR SPAN M^-	M^+
1)	M_u (A-K)	−186	+80
2)	WIDTH b OF DROP PANEL IN COL. STRIP (in)	96	96
3)	EFFECTIVE DEPTH d (in)	8.81	6.44
4)	$M_n = M_u/0.9$ (A-K)	−207	+89
5)	$R = M_n/bd^2$	−333	+268
6)	ρ	0.00585	0.00466
7)	$A_s = \rho bd$ (in²)	4.95 ✓	2.88 ✓
8)	$A_{s,min} = 0.002bt$	3.77	1.76
9)	$N = \frac{\text{GREATER 7 OR 8}}{0.31}$	16 ✓	10 ✓
10)	$N_{min} = \frac{\text{width strip}}{2t}$	5	7



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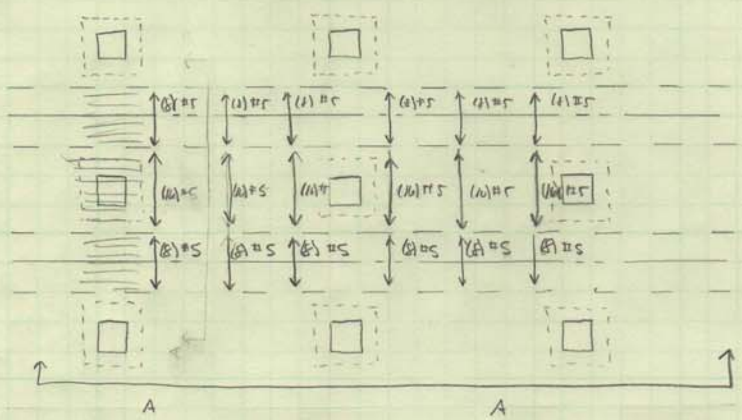
DESIGN OF SLAB REINFORCEMENT IN MIDDLE STRIP [ASSUME #5 BAR]

ITEM NO	DESCRIPTION	INTERIOR SPAN M^-	M^+
1.)	M_u (ft-k)	-62	+54
2.)	b(in)	117	117
3.)	d(in)	6.44	5.81
4.)	$M_n = M_u/0.9$ (ft-k)	-69	+60
5.)	$R = M_u/bd^2$	-171	+182
6.)	ρ	0.00295	0.00312
7.)	$A_s = \rho bd$ (in²)	2.22 ✓	2.12 ✓
8.)	$A_{s,min} = 0.0018bt$ (in²)	1.76	1.76
9.)	N = GREATER 7 or 8 / 0.31	8	6
10.)	N_{min} = width strip / 2t	8 ✓	8 ✓

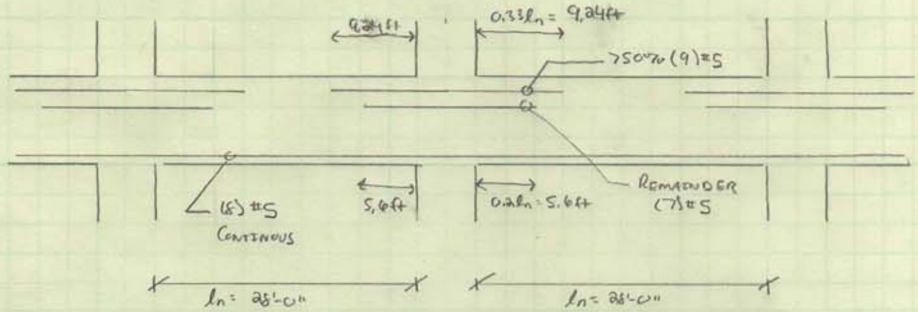


Minimum Extensions for Reinforcement Slabs without Beams

+ ACI 318-08 Figure 13.3.8

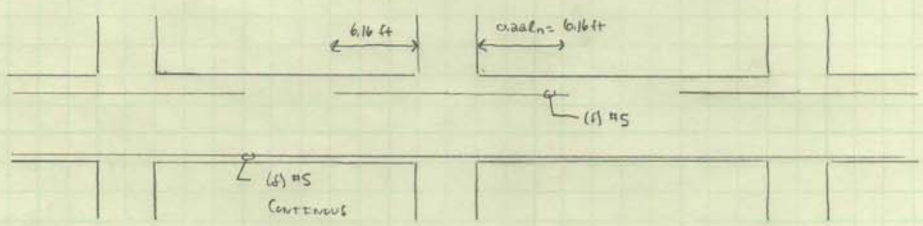
Column Strip Reinforcement

Section A-A



Middle Strip Reinforcement

Section A-A



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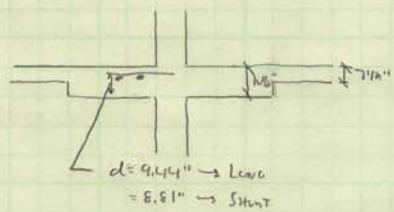
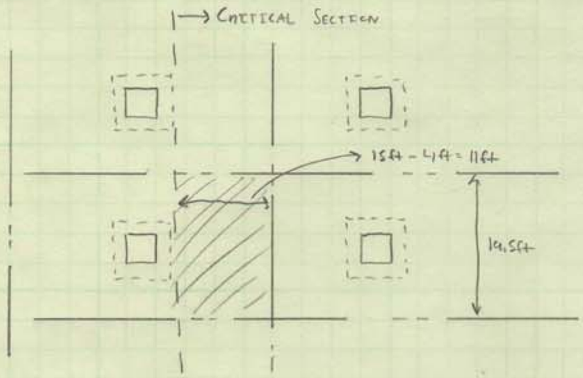
CHECK SHEAR



$d_{avg} = \frac{9.44+8.81}{2} = 9.12\,in$



$V_u = 0.200\,ksf(11\,ft)(14.5\,ft) = 42.9^K$ ⇒ FOR WIDE BEAM ACTION CHECK

$V_n = V_c = 2\sqrt{f'_c}\,b_w d = \frac{2\sqrt{4000}\,(14.5\,ft)(12\,in/ft)(9.12\,in - 3\,in)}{1000} = 181^K$

$\phi V_n = 0.75(181^K) = 136^K > 42.9^K$ ∴ OKAY

PUNCHING SHEAR:

$V_u = 0.200\,ksf(14.5\,ft)(30\,ft) = 117^K$

$b_o = 31.5\,in(4) = 126\,in$

$V_c = 4\sqrt{f'_c}\,b_o d = 4\sqrt{4000}\,(126\,in)(9.12\,in)(1/1000) = 291^K$ ⇒ USING SIMPLIFIED EQUATION

$\phi V_c = 0.75(291^K) = 218^K > V_u$ ∴ okay