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Structural Option

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October 24, 2008

1100 Broadway

Oakland, CA

Technical Report 2: Pro-Con Structural Study of Alternate Floor Systems

http://www.engr.psu.edu/ae/thesis/portfolios/2009/sgh132/

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Where # indicates the order of the table in the report.

Appendices

*Figures and Tables in the Appendix are labeled as follows: Table x.#
Where x indicates the Appendix section and # indicates the order of the table in that section.

Executive Summary

1100 Broadway is a 20-story office building located in the Bay Area of Oakland, California. It contains 310,000 square feet of office space and 10,000 square feet of retail space at the ground level. The project is currently in the design development phase and construction is scheduled to begin in June of 2010. The gravity system is composite metal deck supported by composite steel beams and the lateral system is composed of steel moment and concentric braced frames.

For technical assignment two a structural study and comparison of four floor systems was performed. The current floor system, composite metal deck on composite steel beams, was redesigned for a typical bay of 1100 Broadway along with three alternative systems. The other options include longspan steel joists, a two-way post-tensioned concrete slab, and precast hollow-core concrete planks. The systems were compared based on depth, weight, cost, deflection, and constructability to determine their potential for use in 1100 Broadway.

The steel composite system is easy to construct, somewhat expensive compared to the other systems and is limited by deflections. Total depth of the system is 30.25". The longspan steel joists system depth totals 27". It's the lightest system, average cost but is also limited by deflections. The hollow-core concrete plank system supported by steel beams is 36" deep, very heavy and would be difficult to erect for the 20-story building.

After analyzing and comparing the four alternative floor systems it was determined that the most feasible option for 1100 Broadway is the two-way post-tensioned concrete slab. It allowed the total floor system depth to be reduced from 30.25" to only 9". Although the system is costly and heavy the potential savings due to reduced floor to floor height and possibly overall building height could outweigh the negatives. Further investigation of the post-tension system should follow including considering a one-way slab system.

Introduction

Building Overview

1100 Broadway is a 20-story tower primarily used for offices but also provides shopping and entertainment at the ground level. Its architecture combines a new high-rise tower with the adaptive re-use of the Key System Building facade which houses a smaller portion of the building. The Key System Building is a 37,000 square foot historic office building which was damaged in the 1989 Loma Prieta earthquake and has remained vacant ever since. It is now a National Historic Landmark and its facade is incorporated into the design of the first eight floors of 1100 Broadway. Sustainability was a primary concern in the design of 1100 Broadway. It aims to achieve a LEED Gold rating by incorporating many green features into its design. It takes advantage of the opportunity to utilize Transit Oriented Development (TOD) due to its location directly above the 12th Street/City Center BART public transportation station. It features photovoltaic solar panels on the tower roof, a green roof on the Key System Building portion, and a rainwater collection, filtration and reuse system. The building envelope is comprised of high performance glass from floor to roof with large curtain walls on two of the four elevations. The high performance glass is "tuned" depending on which side of the building it's on: At the south and west facades, which receive more direct sun, the glass is slightly darker, at the north and east facades the glass is slightly clearer.

Existing Structural System

Typical office floors are 3¼″ light weight concrete fill on a 3″ 18 gage Verco W3 Formlock composite steel deck for a total thickness of 6¼″. Composite steel beams support the deck. Columns supporting the composite deck are standard structural steel wide flange sections. Mechanical areas are similar to the typical office floors with the exception of normal weight concrete fill in place of the lightweight fill on composite metal deck. The roof system on the tower portion of the structure consists of the same composite steel deck system as the typical office floors.

Wind and earthquake forces are resisted by a dual system composed of Steel Special Concentric Braced Frames located around and across the building core and Special Moment Resisting Frames (SMRF) at the building perimeter. Braces are wide flange members with welded connections. Diagonal bracing member sizes range from W12x96 to W14x132. Member sizes of the moment resisting frames range from W24x94 to W24x207. Lateral forces are distributed to the SMRF at the perimeter of the building and the loads are distributed to surrounding members based on their relative stiffnesses with a higher percentage of the load being distributed to the stiffer members.

The main tower of the building is supported by 110 ton, 14″-square, driven prestressed precast concrete piles beneath a reinforced concrete mat foundation. The structure utilizes 117 existing 14″ square piles and requires 334 new 70′-0″ long prestressed concrete piles. The concrete mat slab is 5′-9″ thick with #11 bars spaced at 12″ O.C. each way on both faces. The remaining portion of the foundation is a 9″ thick reinforced concrete slab with #5 bars spaced at 12″ O.C. Framing within Key System portion of the structure is supported by 6′-0″ square spread footings.

Floor Systems: Overview

A36'x31' exterior bay typical of levels 10 - roof was analyzed and redesigned as the following types of floor systems:

1. Steel Composite (Current System)
2. Longspan Steel Joists
3. Two-Way Post-Tensioned Concrete Slab
4. Precast Hollow-Core Concrete Plank

The four systems were evaluated on depth, weight, cost, deflection control, constructability and fireproofiing and compared with the current steel composite system to determine if they were feasible for use in 1100 Broadway. See Figure 1 on page 4 for the typical floor plan indicating the bay analyzed for redesign.

A summary of the advantages and disadvantages of each system is provided after each alternative design. For this assignment only gravity loads were required for calculations, including a superimposed dead load and a live load. The self-weight of each system varies. See Table 1 below for design loads used for this assignment.

IBC 2006 was referenced for fire safety and required horizontal assembles for 1100 Broadway to meet a 2-hour fire-resistance rating. A Live load deflection criterion of L/360 and a total load deflection criterion of L/240 were considered in the design of the floor systems.

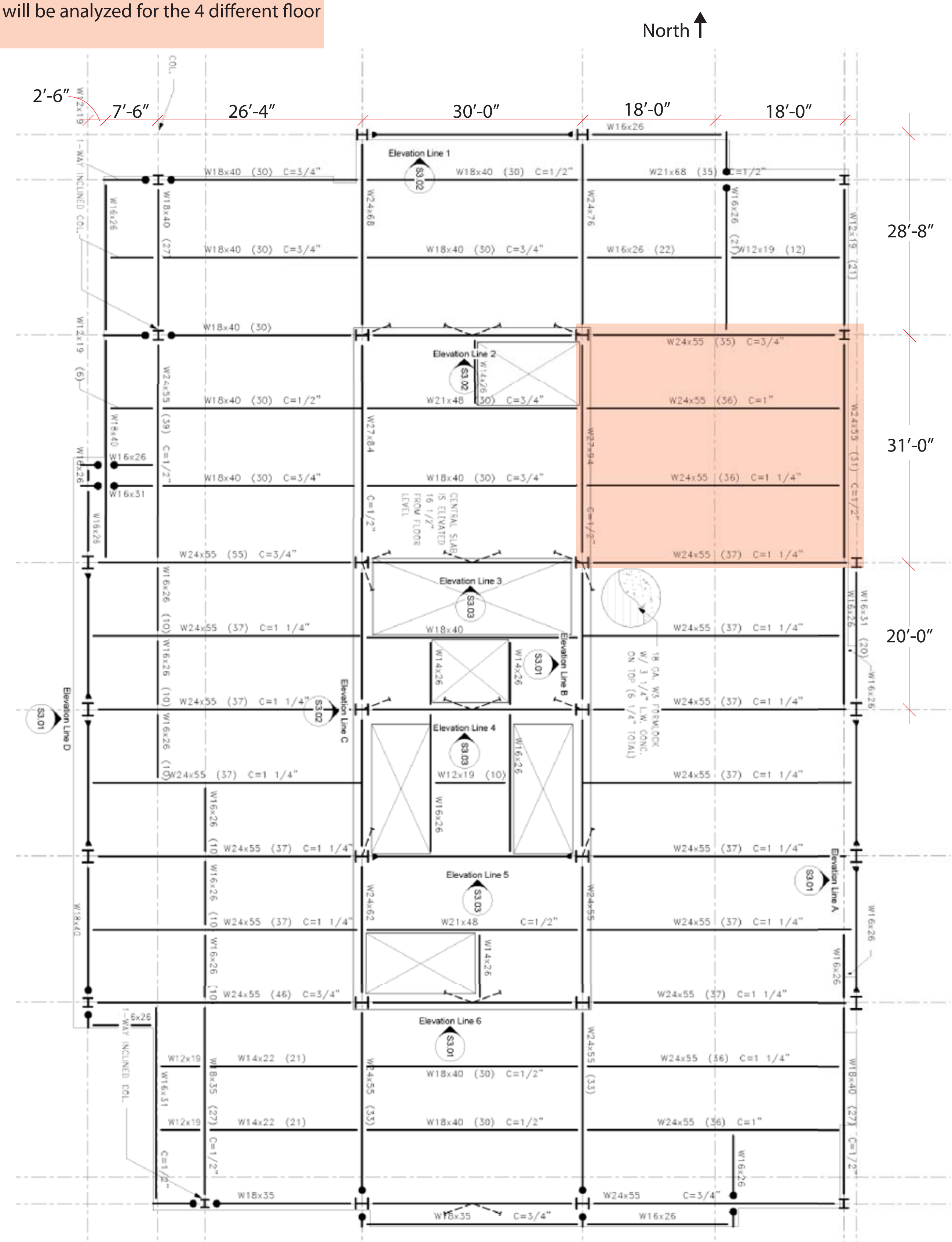
Table 1: Loads used in design of floor systems

Loads used in design	
Superimposed dead load for MEP, finishes, misc.	20 psf
Live load for corridors above first floor (ASCE 7-05)	80 psf*

* ASCE 7-05 requires a minimum live load of 100 psf for lobbies and first floor corridors and a live load of 80 psf for corridors above the first floor. Typical floors are open office plans with no designated corridors and therefore a live load of 80 psf was used in calculations in lieu of the 50 psf office load to be conservative since partition layout in the offices are subject to change.

Typical Framing Plan

Figure 1
Typical Framing Plan for Levels 10 - Roof. The typical bay highlighted in red will be analyzed for the 4 different floor systems.



Floor Systems: Steel Composite

1100 Broadway's current floor system is composite metal deck supported by composite steel beams. The assembly consists of a 3", 18 gage, W3 Verco Formlok deck with 3 ¼" lightweight concrete topping for a total slab depth of 6 ¼". A check was performed on the current design of the W24x55 girder supporting the east end of the beams with 31 evenly spaced shear studs to verify that the available flexural strength of the composite system could be achieved between the points of zero and maximum moment which is between the supports and location of beams framing in. The current design is sufficient and the maximum moment does not exceed the available flexural strength of the composite system.

Using the composite deck properties and the loads from Table 1 the supporting composite members were redesigned. A redesign of the current system was necessary to serve as a reference for the alternative systems to compare with. The systems can now be directly compared because they were designed using all of the same loads and assumptions. Deflection due to the total load was the controlling design parameter for the beams and girders. The redesign consists of W21x55 beams with 24 evenly spaced shear studs. A W24x55 girder supports the east ends of the beams and the west ends of the beams are supported by a W24x84 girder. The shear studs are concentrated in the 10.33 feet on each end of the girders in order to achieve the full strength of the system at the points of maximum moment where the beams frame into the girder. The minimum required shear studs are evenly spaced in the middle 10.33 feet of the girder.
The redesigned members are slightly smaller than those called out in the current design except for the W24x55 girder on the east end which is the same. The placement of shear studs along the redesigned members is more efficient because the current design calls for uniformly placed studs causing more studs to be placed on the center portion of the girder than are required. This is usually typical of members with a uniformly distributed load versus two point loads. A comparison of the current design and the redesign can be seen below in Figures 2 and 3 respectively.

The 6.25" slab/deck depth with 24" deep supporting composite members gives a total system depth of 30.25". For system properties see Table 2 below. The assembly meets the required 2-hour fire rating and does not require additional fireproofing on the deck but the supporting composite steel members will require fireproofing. No changes to the lateral system are necessary.

Table 2
Steel composite system properties

Deck	3", 18 gage, W3 Verco Formlok deck
Slab depth	6.25"
Total depth	30.25"
Concrete	lightweight, 115 pcf, f'c=3000 psi
Shear Studs	3/4" diameter, 5" length

Figure 2
Existing framing for the typical bay

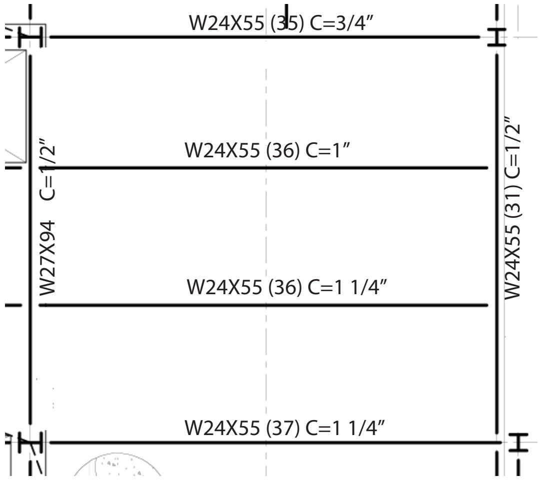
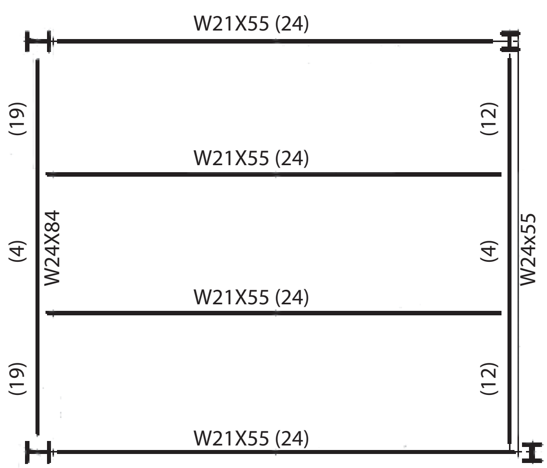


Figure 3
Redesign of composite beams for the typical bay



Floor Systems: Steel Composite

A brief summary of the advantages and disadvantages of the steel composite system as the relate to 1100 Broadway are listed below. A comparison of all four systems can be found at the end of the report.

Advantages :

Depth:	30.25"
Weight:	58.8 psf - deck and slab assembly (48.8 psf) + steel framing (10 psf)
Cost:	$32 per square foot
Constructability:	Simple and easy system to construct
Lateral System:	No changes to current lateral system

Disadvantages:

Fireproofing:	Required fireproofing on supporting beams can be expensive and time consuming
Deflection:	Deflection due to total load controlled design of supporting members

Conclusion for use in 1100 Broadway: High feasibility

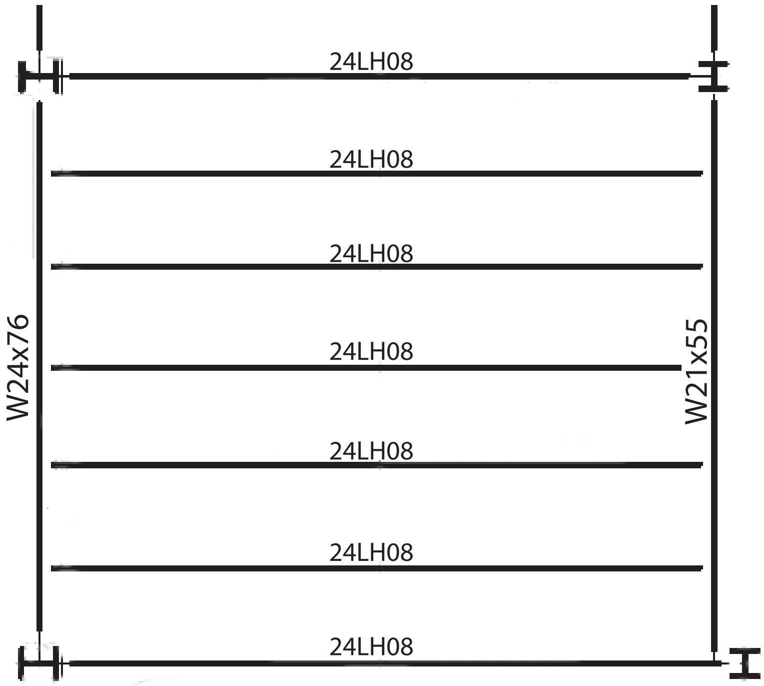
Floor Systems: Longspan Steel Joists

Longspan steel joists were considered as alternative floor system for 1100 Broadway. The deck and slab were designed referencing the United Steel Deck Design Manual and the joists were designed referencing the CMC Joist and Deck Catalog. A 1″ UFX1 form deck with a 3″ lightweight concrete topping was chosen for the deck/slab system. Joists are evenly spaced at 5.17′ over the 31′ length and span 36′ in the East/West direction. See Table 3 below for system properties. 24LH08 joists provide the most economical design but 20LH09 joists may be used if there are restrictions on the floor depth. The 24″ depth of the joists with a 3″ thick deck/slab system amount to a total system depth of 27″. A W24x76 beam will support the west end of the joists and a W21x55 beam will support the East end of the joists. See figure 4 below for the framing layout. The supporting beams were sized to meet industry accepted values for deflection and the joist tables from the CMC Joist and Deck Catalog were designed for the joists to meet deflection limits. The slab/deck assembly meets the required 2-hour fire rating but spray applied fire resistive materials (SAFRM) must be applied to the steel joists in order for the total floor system to meet the 2-hour fire rating. Changing 1100 Broadway's floor system to longspan steel joists would not have any significant impact on the lateral system. The current system of steel moment and braced frames could remain as the lateral system with the longspan steel joist floor system.

Table 3
24LH08 joist system properties

Joist depth	24"
Deck/Slab depth	3"
Total depth	27"
Joist weight	18 plf

Figure 4
24LH08 steel joist design for the 36′x31′ typical bay spaced at 5.17′ O.C.



Floor Systems: Longspan Steel Joists

A brief summary of the advantages and disadvantages of the longspan steel joist system are listed below.

Advantages:

Weight:	31 psf - deck (1.5psf) + slab (24 psf) + joists (0.5psf) + steel framing (5 psf)
Cost:	$26 per square foot
Lateral:	No changes to the lateral system required

Disadvantages:

Depth:	27″
Fireproofing:	Required fireproofing steel joists and supporting beams can be expensive and time consuming
Deflection:	Supporting beam sizes increased due to deflection
Vibrations:	Relatively high

Conclusion for use in 1100 Broadway: Low feasibility

Floor Systems: Two-Way Post-Tensioned Concrete Slab

A Two-way post-tensioned concrete slab was considered as an alternative floor system. For simplicity in the preliminary analysis the post-tensioning tendons were only designed in one direction. The design features a 9″ thick normal weight concrete slab with 1/2″ diameter banded tendons. See Table 4 for system properties. The banded tendons were designed in the East/West direction across the 36′ span and run across the column lines. Twenty-two banded tendons are required to adequately stress the slab. The effective prestress force of the 22 tendons is approximately 585 kips. To avoid overstressing the slab fewer tendons were used than originally designed and additional mild steel reinforcing may be required. Twelve #4 steel reinforcing bars are required at the first interior support where the moment is largest and four #4 bars are required at the exterior support. See Figure 5 for a section of the system. The uniform tendons will span the North/South direction but were not designed in this analysis. Deflections were not calculated due to the complexity of hand calculations and lack of software capable of post-tension design but are normally minimal in post-tensioned systems. The slab meets the cover requirements from IBC 2006 to acquire a 2-hour fire rating and no additional fireproofing is required. See Figure 6 below for the layout of the post-tensioned system. Switching to a post-tensioned slab will require changes to the lateral system. The post-tensioned slab will be able to handle a portion of the lateral forces but shear walls or concrete moment frames will be needed for additional resistance to the lateral loads. Locations of openings in the post-tensioned slab are critical. Unlike openings in a typical mild steel reinforced slab, cutting through a tendon could cause the entire post-tensioned floor slab to fail.

Table 4
Post-tensioned slab system properties

Total depth	9"
Concrete	f'c=5000 psi, f'ci=3000 psi
Banded Tendons	unbonded, 1/2" diameter, 7-wire strand

Figure 5
Post-tensioned concrete slab section

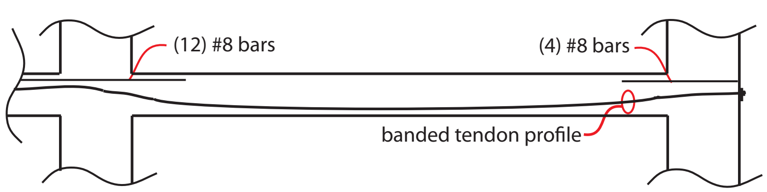
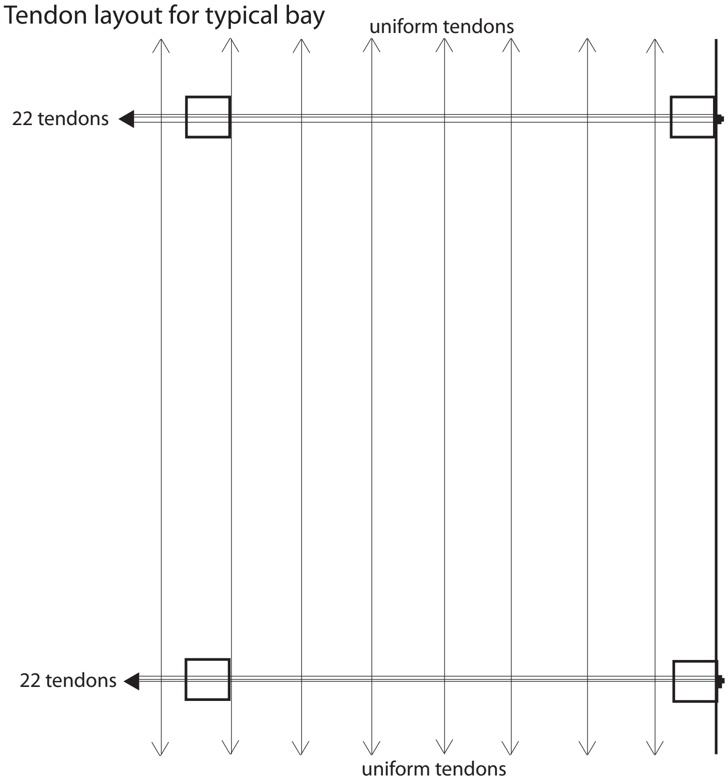


Figure 6
Tendon layout for typical bay



Floor Systems: Two-Way Post-Tensioned Concrete Slab

A brief summary of advantages and disadvantages of the two-way post-tensioned concrete slab system are listed below.

Advantages :

Depth:	9"
Vibrations:	Very limited
Cost:	$24 per square foot
Deflection:	Not calculated but kept to a minimum in post-tensioned design
Fireproofing:	None Required

Disadvantages:

Constructability:	Difficult to properly place and stress tendons. Opening locations are critical.
Weight:	115 psf
Lateral:	Post-tensioned slab will require an alternate lateral system of either concrete moment frames or shear walls.

Conclusion for use in 1100 Broadway: High feasibility

Floor Systems: Precast Hollow-Core Concrete Plank

A precast hollow-core concrete plank system was the final floor system considered for 1100 Broadway. Concrete plank size was determined using the 6th Edition of PCI Handbook. A 4′-0″ x 10″ normal weight concrete plank with 2″ topping was chosen for the design. See Figure 7 for a section view of the plank and Table 5 for system properties. Nine hollow-core planks fit within the 36′ span. Steel beams to support the planks were designed using AISC 13th Edition Manual of Steel Construction. The beam supporting the north end of the planks is a W24x176 while the beam supporting the south end is a W24x146. For the plank layout and beam locations for the typical bay see Figure 8 below. The total system depth including the steel beams is 36″ which exceeds the composite system depth by more than one foot. The precast concrete planks consider deflection in their design and meet fireproofing requirements for a 2-hour fire rating. The supporting beams also meet deflection requirements but will require SAFRM to achieve a 2-hour fire rating. The hollow-core concrete planks will be capable of resisting a significant portion of the lateral load. Steel moment frames or concrete moment frames are options for lateral systems. Locations of openings in the planks are critical and can be costly to perform. Engineering approval should be obtained before cutting any openings. The openings can be core drilled or cut with a concrete saw but should be cut only after the planks have been erected. Erection of the hollow-core plank system may be costly for the 20-story height.

Table 5
Hollow-core concrete plank system properties

Concrete topping	2" normal weight
Plank width	4'-0"
Plank depth	10"
Plank+topping weight	370 plf
Plank concrete	normal weight, f'c=5000 psi
Beam depth	24"
Total depth	36"

Figure 7
Hollow-core concrete plank section

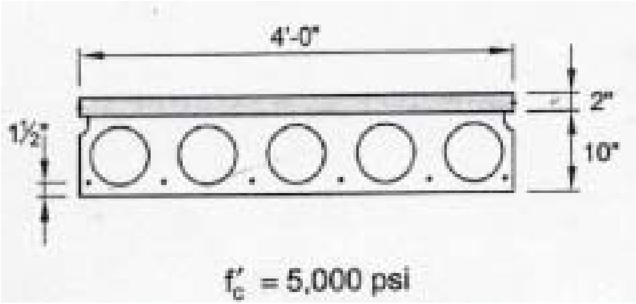
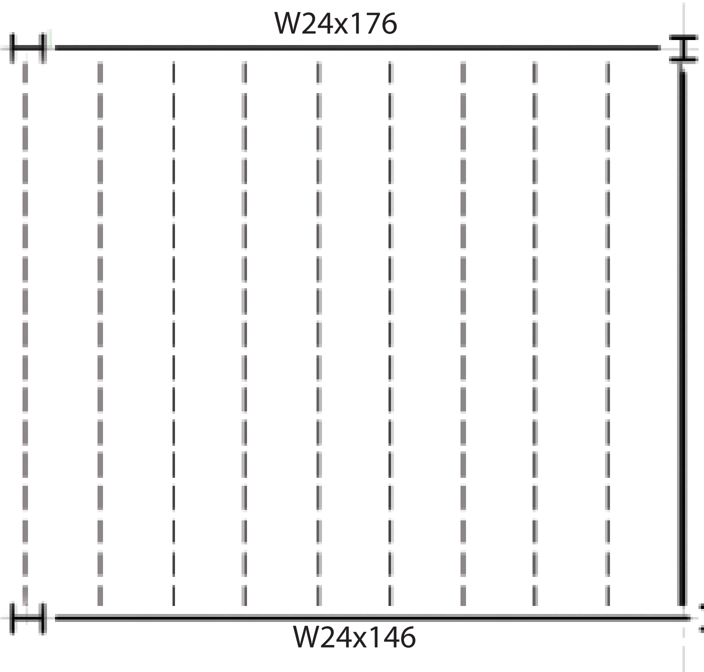


Figure 8
Hollow-core plank layout for typical 36′x31′ bay



Floor Systems: Precast Hollow-Core Concrete Plank

A brief summary of advantages and disadvantages of the precast hollow-core concrete plank system are listed below.

Advantages :

Cost:	$24 per square foot
Deflection:	Precast hollow-core planks were designed for deflections
Fireproofing:	None Required
Lateral:	Concrete planks will handle a portion of the lateral load but steel or concrete moment frames will also be necessary

Disadvantages:

Depth:	36″
Weight:	105 psf
Constructability:	Location of openings are critical and may be costly to perform

Conclusion for use in 1100 Broadway: Low feasibility

Comparison and Conclusion

Comparison:

A comparison of the floor systems as they relate to 1100 Broadway can be seen below in Table 6.

Table 6
System Comparison

	Steel Composite	Steel Joists	Post-Tensioned Slab	Hollow-Core Plank
System Depth (in.)	30.25	27	9	36
Weight (psf)	58.8	31	115	105
Cost (per sq. ft.)	$32.00	$26.00	$24.00	$24.00
Deflections	Medium	High	Low	Low
Constructability	High	Medium	Low	Low
Fireproofing	Supporting Beams	Supporting Beams	Not required	Not required
Feasibility	High	Low	High	Low
Potential for more in depth investigation	Yes	No	Yes	No

Conclusion:

The two-way post-tensioned concrete slab is the best alternative floor system studied for 1100 Broadway. The post-tensioning of the system makes it possible to reduce the total floor depth from 30.25″ with the composite system to 9″. Post-tensioned slab systems are capable of achieving relatively shallow depths for long spans which was exemplified in the redesign of the 36′ span. With the reduction in floor depth comes the potential for reduction in the overall height of the building due to the decreased floor to floor height. This potentially makes the post-tensioned design a very economical alternative for 1100 Broadway. Deflections are also limited with post-tensioned design due to the upward force exerted by the tendons.

The main disadvantages of the system are its heavy weight, low constructability, and location of openings being critical. The increased weight of the system will likely impact the foundations and will require further evaluation. Placing the tendons and concrete so that the tendons follow the appropriate profile can sometimes be difficult and properly stressing the tendons can also be challenging.

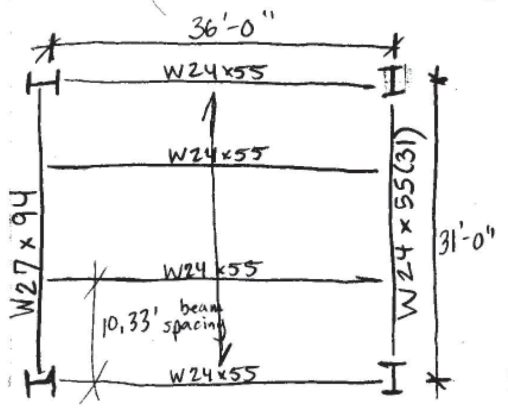
The post-tensioned system will require an alternative lateral system. The post-tensioned slab will be able to resist a portion of the lateral load but either concrete moment frames or shear walls will also be necessary to resist the lateral loads.

Appendix A: Steel Composite

Hand Calculation: Check current design of girder supporting East ends of beams

Check current design: W24 x 55 girder on East Side

Deck/Slab properties: Reference United Steel Deck Design Manual and Catalog of products



For 18 gage, 3" metal deck $w = 2.8$ psf.

3 1/4" lightweight slab $w = 46$ psf

S.I dead load $w = 20$ psf

$w_d = 68.8$ psf.

live load $w_L = 80$ psf.

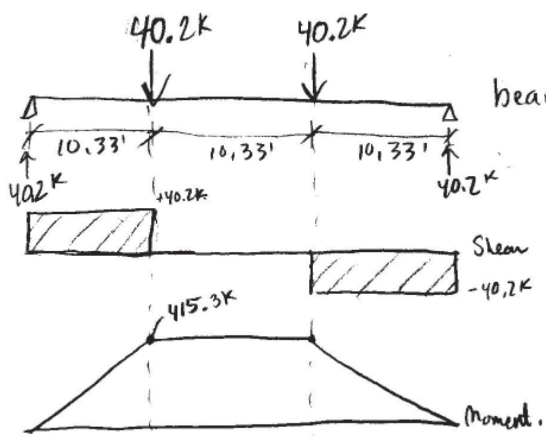
Point loads on the girder from W24x55 beams:

$P_u = (1.2D + 1.6L)(18' \times 10.33')$

$= [1.2(68.8 \text{ psf}) + 1.6(80 \text{ psf})](18' \times 10.33')$

$= 39.15^K$ each.

beam weight = 55 plf * 18 ft ≈ 1K.



∴ $M_u = 415.3^K$

Properties of W24x55 Girder

$A = 16.2$ in^2
$d = 23.6"$
$t_w = 0.395"$
$b_f = 7.01"$
$t_f = 0.505"$

• Reference 13th Edition AISC Steel Manual

31 Shear Studs
↳ Drawing notes specify Nelson S3L or H4L studs.
↳ Assumed 3/4" Φ

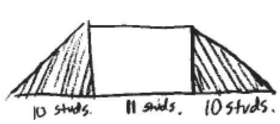
Shear Stud Capacity:

Table 3-21, Deck runs Parallel to girder.

For l.w. conc, $f'_c = 3$ ksi, $\frac{w_r}{h_r} \geq 1.5$ $Q_n = 17.1$ K/stud.

ΣQ_n = sum of nominal strengths of shear connectors between the point of max (+) moment (@ Point load due to beams, 10.33') and point of zero moment (support).

There fore, according to the current design of $\frac{31 \text{ studs}}{31 \text{ ft}} = 1$ stud/ft, in the first 10.33' there will be 10 evenly spaced studs.



10 studs * 17.1 K/stud = 171 K = ΣQ_n.

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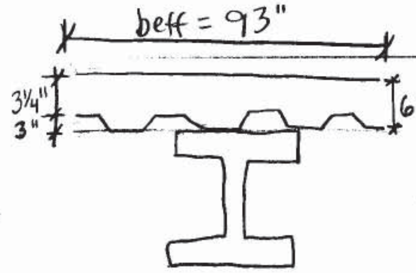
Appendix A: Steel Composite

Hand Calculation: Check current design of girder supporting East end of beams



6¼" (deck runs (∥) to girder)

$b_{eff} = \min \begin{cases} \text{Span}/4 = \frac{31'}{4} = 7.75' = 93" \\ \text{Spacing} = 18' = 216" \end{cases}$

$A_c = b_{eff}(t_c) = 93"(6.25") = 581.25\ in^2$

• $C_c = 0.85 f'_c A_c = 0.85(3\ ksi)(581.25\ in^2) = 1482.2\ K = V_c'$

• $T_s = f_y A_s = 50\ ksi(16.2\ in^2) = V_s' = 810\ K$

since $\Sigma Q_n = 171^K < 1482.2^K = V_c'$, $810^K = V_s'$

$C_q = V_q' = \Sigma Q_n = 171^K$ controls and girder is partially composite.

$a = \dfrac{\Sigma Q_n}{0.85 f'_c \cdot b_{eff}} = \dfrac{171\ K}{0.85(3ksi)(93in)} = 0.721"$

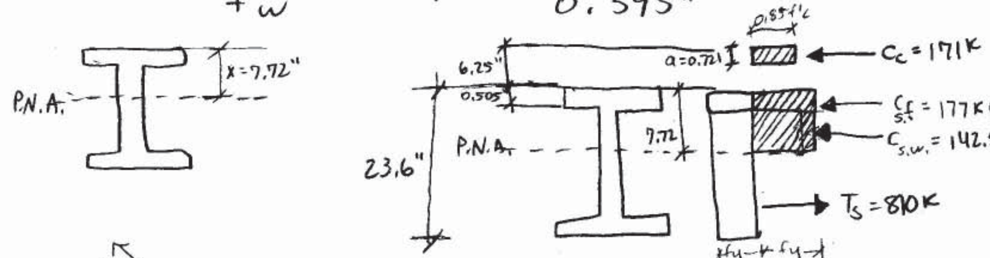
$A_f = t_f b_f = 0.505(7.01)$

Area of Steel in Compression:

$A_{s-c} = \dfrac{T_s - C_c}{2 F_y} = \dfrac{810^K - 171^K}{2(50\ ksi)} = 6.39\ in. > 3.54" = A_f$

∴ PNA is in Web

$x = \dfrac{A_{s-c} - A_f}{t_w} + t_f = \dfrac{6.39" - 3.54"}{0.395"} + 0.505" = 7.72"$



$\Sigma M_{\text{top steel flange}}$

$M_n = 171K(6.25 - \frac{0.721}{2}) - 354(0.505/2) - 285(7.215/2 + 0.505) + 810(23.6/2)$

$M_n = 9304$ in-k

$\phi M_n = 0.9(9304\ \text{in-K}) = 8373.6\ \text{in-k} / 12"/ft = 698$ ft-k.

$\phi M_n = 698$ ft-k > 415.3 ft-k $= M_u$.

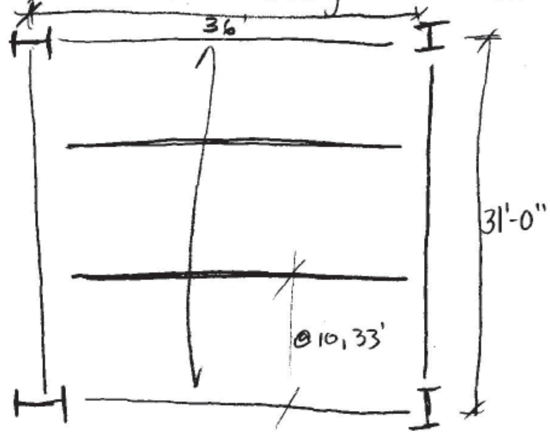
∴ Current design is sufficient.

Appendix A: Steel Composite

Hand Calculation: Design of supporting beams

Redesign of current Composite System.

For Deck/Slab Properties reference United Steel Deck Design Manual
For beam design Reference 13th Edition AISC Steel Manual.



Deck/Slab: 18 gage, 3" metal Deck
3 1/4" lightweight Conc. Slab.

$w = 2.8\text{ psf} + 46\text{ psf} = 48.8\text{ psf}$

S.I. Dead = 20 psf

$w_L = 80\text{ psf}$.

$w_u = 1.2D + 1.6L$

$= 1.2(68.8\text{ psf}) + 1.6(80\text{ psf})$

$w = 210.56\text{ psf}$.

Design of composite beams supporting composite deck:

$w_u = 210.56\text{ psf}\,(10.33\text{ ft})$

$w_u = 2175\text{ plf}$.

Max unshored Span = 13,26'
therefore need beams at 3rd Points of span.

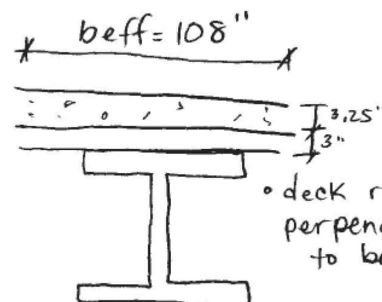
$\frac{31'}{3'} = 10.33'$ spacing.

$M_u = \frac{wl^2}{8} = \frac{2175\text{ plf }(36')^2}{8} / 1000 = 352.4\text{ ft-k}$

$b_{eff} = \min \begin{cases} \text{Span}/4 = 36/4 = 108'' \\ \text{Spacing} = 10.33' = 124'' \end{cases}$

Assume $a = 1''$

$\frac{y}{2} = 6.25'' - \frac{1}{2} = 5.75'' \rightarrow 5.5''$ conserv.



Try W18 x 35:

$\phi M_p = 249$ ft-k.

$\phi M_{pc} = 411$ ft-k $\quad \Sigma Q_n = 194$ k. @ P.N.A 6.

- deck runs perpendicular to beams.
- ribs spaced @ 12"
- ∴ Max # of studs = 36 for this span.

Check Assumption that $a = 1.0''$.

$a = \frac{\Sigma Q_n}{0.85 f'_c\, b_{eff}} = \frac{194\text{ k}}{0.85(3\text{ ksi})(108'')} = 0.71'' < 1''$

∴ Assumption conservative.

$M_u = 352.4\text{ ft-k} < 411\text{ ft-k} = \phi M_{pc}$

∴ OK → still need to check deflections.

Appendix A: Steel Composite

Hand Calculation: Design of supporting beams

Determine # Shear studs required.

- From Table 3-21: Deck perpendicular, strong axis, 1 stud per rib
  $f'c = 3$ ksi, l.w. concrete, 3/4"ϕ studs.

$Q_n = 17.1$ K

studs required $= \frac{\Sigma Q_n}{Q_n} = \frac{194^K}{17.1^K} = 12$ studs

12 studs each side of beam

= 24 total < 36 that will fit. ∴ OK

Check Deflection:

Deflection during construction under wet concrete load:

$w_{Deck/conc.} = 68.8$ psf. Assume Construction Live load = 20 psf

$w_u = [1.2(68.8\text{ psf}) + 1.6(20\text{ psf})] * (10.33') = 1184$ plf

$M_u = \frac{w_u l^2}{8} = \frac{1184(36')^2}{8} = 192$ ft-k < 249 ft-k $= \phi M_p$

Δconstruction Limit = 1" for spans greater than 30 feet.

$I_{18\times35} = 510\text{ in}^4$

$$\Delta_{Const.} = \frac{5 w_{Dead} L^4}{384 EI} = \frac{5(68.8\text{ psf})(10.33\text{ ft})(36\text{ ft})^4(1728\text{ in}^3)}{384(29{,}000{,}000)(510)} = 1.82''$$

$\Delta_{const.} = 1.82'' > \Delta_{allow}$ ∴ Not good.

I required to limit deflection to 1":

$$I_{req'd} = \frac{5 w_{Dead} L^4}{384 E \Delta_{all.}} = \frac{5(68.8\text{ psf})(10.33\text{ ft})(36\text{ ft})^4(1728\text{ in}^3)}{384(29{,}000{,}000)(1'')} = 927\text{ in}^4$$

⇒ choose W21x50 $I_x = 984\text{ in}^4 > I_{req'd} = 927\text{ in}^4$ ∴ OK.

$\phi M_p = 413 > 192$ ft-k ∴ OK.

$\phi M_{pc} = 599$ ft-k > 352.4 ft-k $= M_u$

$\Sigma Q_n = 184$ K ⇒ Shear studs req'd $= \frac{184^k}{17.1^k} = 11$ studs each side.

22 total < 36 ∴ OK.

Appendix A: Steel Composite

Hand Calculation: Design of supporting beams

Check Live Load Deflection:

$$\Delta_L = \frac{5 w_L L^4}{384 EI} = \frac{5(80\ psf \times 10.33\ ft)(36ft)^4\ 1728}{384(29{,}000{,}000)(984\ in^4)} = 1.09''$$

$$\Delta_{max} = \frac{l}{360} = \frac{36' * 12''/ft}{360} = 1.2''$$

$\Delta_L = 1.09'' < 1.2''$ ∴ OK ✓

check deflection due to total load;

$$\Delta_T = \frac{5 w_T l^4}{384 EI} = \frac{5(68.8+80)(10.33)(36ft)^4\ (1728)}{384(29{,}000{,}000)(984\ in^4)} = 2.04''$$

$$\Delta_{max} = \frac{l}{240} = \frac{36' * 12''/ft}{240} = 1.8''$$

$\Delta_T = 2.04 > 1.8'' = \Delta_{max}$ ∴ No good.

Find new I'req'd:

$$I = \frac{5 w_T l^4}{384 E \Delta T} = \frac{5(68.8+80)(10.33)(36ft)^4(1728)}{384(29{,}000{,}000)(1.8'')} = 1113 in^4$$

Choose W21x55 → $I_x = 1140\ in^4$

$\phi M_p = 473$ ft-K > 192 ft-K ∴ ok.

$\phi M_{pc} = 672$ ft-K > 352.4 ft-k = M_u ∴ ok

$\Sigma Q_n = 203$ K. → req'd # Shear Studs $= \frac{203^k}{17.1^k} = 12$ Studs each side.

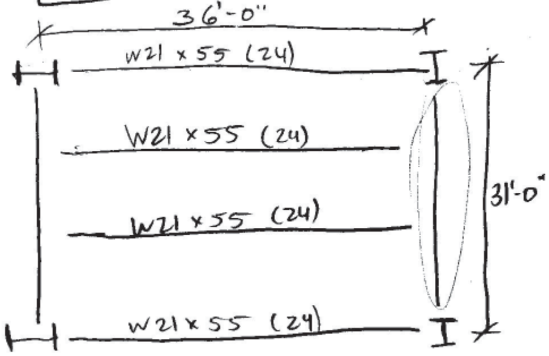
$\boxed{\text{∴ Choose W21x55 beams with 24 shear studs evenly distributed.}}$

Appendix A: Steel Composite

Hand Calculation: Design of girder supporting East ends of beams

Design of girders supporting beams:

East End:



Point loads on girder.

$W_{Deck+Slab+S.I.} = 68.8\ psf \cdot 10.33\ ft. = 710.7\ plf.$

$W_{W21x55\ beam} = 55\ plf$

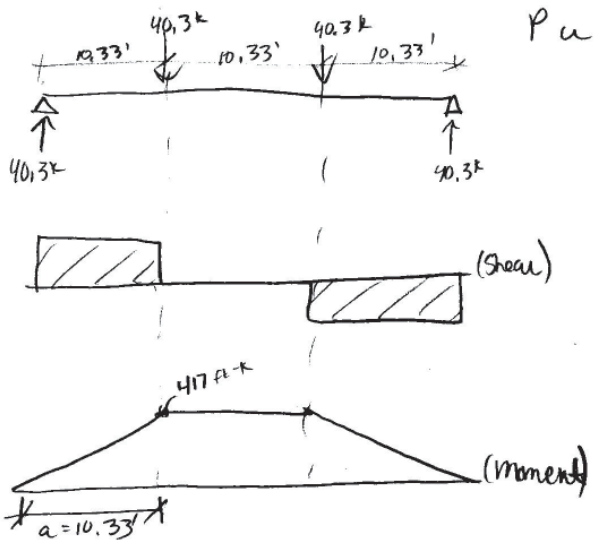
$765.7\ plf$

$w_{Live} = 80\ psf\ (10.33\ ft) = 826.4\ plf.$

$P_u = (1.2D + 1.6L)\ 18'$

$P_u = [1.2(765.7\ plf) + 1.6(826.4\ plf)]\ 18'$

$P_u = 40.3\ k.$



b_{eff} { Span/4 = 31/4 = 7.75 → 93"
= spacing = 18' =

$M_u = 417$ ft-k

Start with W21x55 since beams frame into girder and deflection controlled the design, not M_u.

Assume a = 1" $y_2 = 6.25 - 1/2 = 5.75" \rightarrow$ chose $Y_2 = 5.5$ conserv.

$\phi M_p = 473$ ft-k

Check Assumption A = 1.

$\phi M_c = 672$ ft-k > 417 ft-k = M_u.

$a = \frac{\Sigma Q_n}{0.85 f'c\, b_{eff}} = \frac{203}{0.85(3)(93)} = 0.86 < 1$ ∴ ok

$\Sigma Q_n = 203$ K.

For deflection due to wet concrete during construction:

$P_u = [(765.7\ plf)1.2 + 1.6(20\ psf \times 10.33\ ft)]\ 18' = 22.5^K$

$M_{u_{const.}} = P_u * a = 22.5^K(10.33\ feet) = 232.5$ ft-k.

$\phi M_p = 473$ ft-k > 232.5 ft-k = $M_{u_{const.}}$ ∴ OK.

Appendix A: Steel Composite

Hand Calculation: Design of girder supporting East ends of beams

$\Delta_{const.}$ Limit $= 1"$ b/c Span $= 31\text{ ft} > 30\text{ ft}$,

$I_{W21\times55} = 1140\text{ in}^4$

from dead load $= (765.7\text{ plf})(18\text{ ft})/1000$

$$\Delta_{const.} = \frac{Pl^3}{28EI} = \frac{(13.78^K)(31\text{ ft})^3(1728)}{28(29{,}000)(1140\text{ in}^4)} = 0.77" < 1" \therefore OK.$$

Check live load Deflection:

$$\text{Limit} = \frac{l}{360} = \frac{31' \times 12"/ft}{360} = 1.03" = \Delta_{max}.$$

$$\Delta_L = \frac{Pl^3}{28EI} = \frac{(826.4\text{plf})(18\text{ft}/1{,}000)(31')^3(1728)}{28(29{,}000)(1140\text{ in}^4)} = 0.83"$$

$$\Delta_L = 0.83" < 1.03" = \Delta_{max} \therefore OK$$

Check deflection due to total load:

$$\Delta_{max} = \frac{l}{240} = \frac{31\text{ft} \times 12"/ft}{240} = 1.55"$$

$$\Delta_{Dead} = \frac{(765.7 + 826.4\text{ plf})(18\text{ ft})/1000\ (31\text{ ft})^3(1728)}{28(29{,}000)(1140\text{ in}^4)} = 1.6"$$

$$\Delta_{dead} = 1.6" > 1.55" = \Delta_{max} \text{ No good.}$$

determine $I_{req'd}$:

$$I_{req'd} = \frac{(765.7 + 826.4\text{plf})(18\text{ft})/1000(31\text{ft})^3(1728)}{28(29{,}000)(1.55")} = 1172.2\text{ in}^4$$

Choose W24x55

$\phi M_p = 503$ ft-K > 232.5 ftk $= M_{u\,const.}$

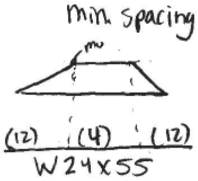
$\phi M_{pc} = 727$ ft-K > 417 ft-K $= M_u$

$\Sigma Q_n = 203K$.

Shear Studs Req'd: Table 3-21, Deck Parallel, $\frac{w_r}{h_r} > 1.5$, $3/4"\phi$, $f'_c = 3$, l.w.c.
$Q_n = 17.1K$.

$$\frac{\Sigma Q_n}{Q_n} = \frac{203^K}{17.1^K} = 12 \text{ on each side up to } M_u.$$

min spacing in mid section $=$ min $\begin{cases} 8 \times \text{slab thickness} = 50" \ (6.25) \\ 36" - \text{controls}: \end{cases}$



Appendix A: Steel Composite

Hand Calculation: Design of girder supporting West end of beams

Design of girder supporting west end of beams: (15 more feet of trib. beam length.)

Point loads on girder from beams. $18+15' = 33'$

$P_u = 1.2D + 1.6L$

$P_u = [1.2(765.7 \text{ plf}) + 1.6(826.4 \text{ plf})] \times 33'$

$P_u = 74K$

$M_u = P_u \times a = 74K(10.33') = 764.5 \text{ ft-K}$

$b_{eff} = \text{Span}/4 = 31' \times 12''/\text{ft}/4 = 93''$

Assume $a = 1''$. $y_2 = 6.25 - 1/2 = 5.75 \rightarrow y_2 = 5.5$ conservative.

Since deflection due to dead load has been controlling design, start with req'd I_x.

$\ell/240 = 1.55''$

$$I_{req'd} = \frac{(765.7 + 826.4 \text{ plf})33 \text{ ft}/1000\ (31 \text{ ft})^3(1728)}{28(29,600)(1.55'')}$$

$I_{req'd} = 2149 \text{ in}^4$. Try W24x84 $I_x = 2370 \text{ in}^4$

$\phi M_p = 840 \text{ ft-K}$

$\phi M_{pc} = 1170 \text{ ft-K} > 764.5 = M_u$

$\Sigma Q_n = 309 \text{ K}$.

Deflection due to wet concrete during construction:

$P_u = [1.2(765.7 \text{ plf}) + 1.6(20 \text{ psf})(10.33 \text{ ft})]33' = 41.3k$

$M_{u\,const.} = P_u * a = 41.3^k(10.33 \text{ feet}) = 427 \text{ ft-K}$.

$M_{u\,const.} = 427 \text{ ft-K} < 840 \text{ ft-K} = \phi M_p \therefore$ OK.

Δ_{const} limit $= 1''$.

$$\Delta_{const.} = \frac{(765.7)(33 \text{ ft})/1000\ (31 \text{ ft})^3(1728)}{28(29,000)(2370 \text{ in}^4)} = 0.68'' < 1'' \therefore \text{OK}.$$

Live load Deflection: $\Delta_{Lmax} = \frac{\ell}{360} = \frac{31 \times 12}{360} = 1.03''$

$$\Delta_{Live} = \frac{(826.4 \text{ plf})(33 \text{ ft})/1000\ (31 \text{ ft})^3(1728)}{28(29,000)(2370 \text{ in}^4)} = 0.73''$$

$\Delta_{Live} = 0.73'' < \Delta_{max} = 1.03'' \therefore$ OK.

Check assumption that $a = 1''$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{309K}{0.85(3)(93'')} = 1.31 \therefore \text{assumption incorrect.}$$

$y_2 = 6.25 - 1.31/2 = 5.60'' \rightarrow 5.5''$ still conserv. therefore OK.

Appendix A: Steel Composite

Hand Calculation: Design of girder supporting West end of beams

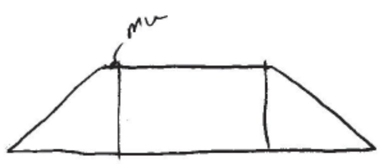
Determine # shear studs required: Table 3-21, Deck parallel, $\frac{w_r}{h_r} > 1.5$, 3/4"ϕ studs, $f'c = 3$ ksi, lwconc. $Q_n = 17.1$ K.

$\frac{\Sigma Q_n}{Q_n} = \frac{369 \text{ K}}{17.1 \text{ K}} = 19$ studs each side

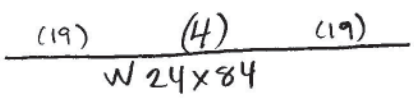
min. center to center spacing of studs along member $= 6 \times d_{\text{diam. stud}}$
$= 6(3/4")$
$= 4.5"$

Check that 19 studs can be placed in 10.33 feet:

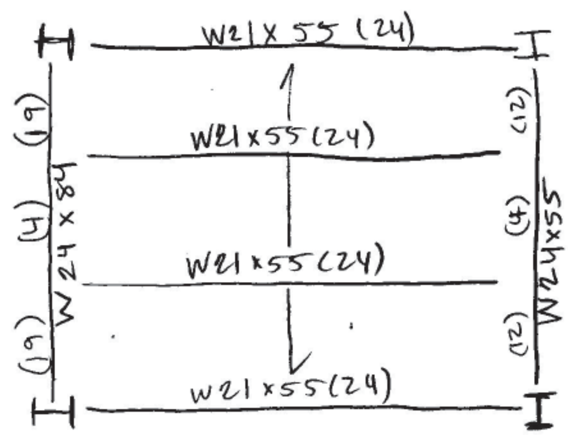
$10.33 \text{ feet} \times 12 \text{ in/ft} = 123.96" / 4.5" \text{ spacing} = 27.5$ ∴ 19 studs will fit in 10.33 feet.



Max spacing @ mid section of beam = 3 feet. → place 4 studs in mid section.



$\boxed{\text{Final Design:}}$



Appendix A: Steel Composite

Tables from the United Steel Deck Design Manual

Table A.1
Maximum uniform live service load

3 x 12" DECK F_y = 33ksi f'_c = 3 ksi 115 pcf concrete

			L, Uniform Live Service Loads, psf *												
	Slab Depth	φMn in.k	9.00	9.50	10.00	10.50	11.00	11.50	12.00	12.50	13.00	13.50	14.00	14.50	15.00
18 gage	5.50	80.96	385	345	305	275	250	225	205	185	170	155	140	130	120
	6.00	92.32	400	390	350	315	285	255	235	210	195	175	160	150	135
	6.25	98.00	400	400	370	335	300	275	245	225	205	190	170	160	145
	6.50	103.68	400	400	395	355	320	290	260	240	220	200	180	165	155
	7.00	115.04	400	400	400	395	355	320	290	265	240	220	205	185	170
	7.25	120.72	400	400	400	400	370	335	305	280	255	235	215	195	180
	7.50	126.40	400	400	400	400	390	355	320	290	265	245	225	205	190
	8.00	137.76	400	400	400	400	400	385	350	320	290	265	245	225	205

Table A.2
Maximum unshored span

	COMPOSITE PROPERTIES												
	Slab Depth	ϕM_{nf} in.k	A_c in^2	Vol. ft^3/ft^2	W psf	S_c in^3	I_{av} in^4	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_w
18 gage	5.50	80.96	37.6	0.333	38	1.94	9.1	54.28	5250	11.48	13.61	14.07	0.0
	6.00	92.32	42.0	0.375	43	2.23	11.6	62.43	5870	10.94	13.07	13.51	0.0
	6.25	98.00	44.3	0.396	46	2.38	13.0	66.67	6180	10.70	12.83	13.26	0.0
	6.50	103.68	46.6	0.417	48	2.53	14.5	70.99	6510	10.48	12.59	13.01	0.0
	7.00	115.04	51.3	0.458	53	2.85	17.9	79.88	7170	10.07	12.16	12.57	0.0
	7.25	120.72	53.8	0.479	55	3.01	19.8	84.42	7510	9.88	11.96	12.36	0.0
	7.50	126.40	56.3	0.500	58	3.17	21.8	89.03	7860	9.71	11.77	12.16	0.0
	8.00	137.76	61.3	0.542	62	3.51	26.2	98.39	8570	9.43	11.42	11.80	0.0
	8.25	143.44	63.9	0.563	65	3.68	28.6	103.15	8930	9.33	11.25	11.62	0.0
	8.50	149.12	66.6	0.583	67	3.85	31.1	107.94	9300	9.23	11.09	11.46	0.0

Table A.3
Fire Rating

2	D913	N	3 ¼ LW	BL,LF15,LF2,LFC2,LF3,LFC3
	D916	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D916	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D916	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D918	N	4 ½ NW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC

-Meets 2-hour fire rating requirement and requires no additional fire proofing on deck

Appendix B: Longspan Steel Joists

Hand Calculation:

Reference: United Steel Deck Design manual and catalog of products

• Choose Deck, Slab thickness and Spacing:

Try 1" (UF1X) Form Deck with a 3" light-weight concrete slab. Total Thickness = 3".

Self-weight of slab = $\frac{(3" - 1\frac{1}{2}")}{12"/ft} * 115\ pcf = 24\ psf$.

Joist Spacing: over 31 ft length → Almost 5' even spacing

$\frac{31\ ft}{6} = 5.17'$ spacing

Determining load:

$w_{LL} = 80\ psf$

$w_{DL} = w_{slab} + w_{S.I.}$

$= 24\ psf + 20\ psf$

$= 44\ psf$

LRFD:

$W_u = (1.2\ W_{DL} + 1.6\ W_{LL})$

$W_u = 1.2(44\ psf) + 1.6(80\ psf)$

$W_u = 181\ psf$

For UF1X deck, $W_u = 181\ psf$ → 22 gage deck is sufficient.

$W_u = 181\ psf < W_{safe}$ = 193 psf for Single Span, 191 psf for double Span

• Design Longspan Steel Joists:
Reference CMC Joist and Deck Catalog.

$W_u = [1.2(44\ psf) + 1.6(80\ psf)] * 5.17\ feet$

$W_u = 935\ plf$

For the 36' Span choose 20LH09 or 24LH08.

20LH09 weighs 21 plf → $W_{safe} = 954\ plf > 935\ plf$ ∴ OK.

24LH08 weighs 18 plf → $W_{safe} = 933\ plf$, only slightly less than 935 plf.

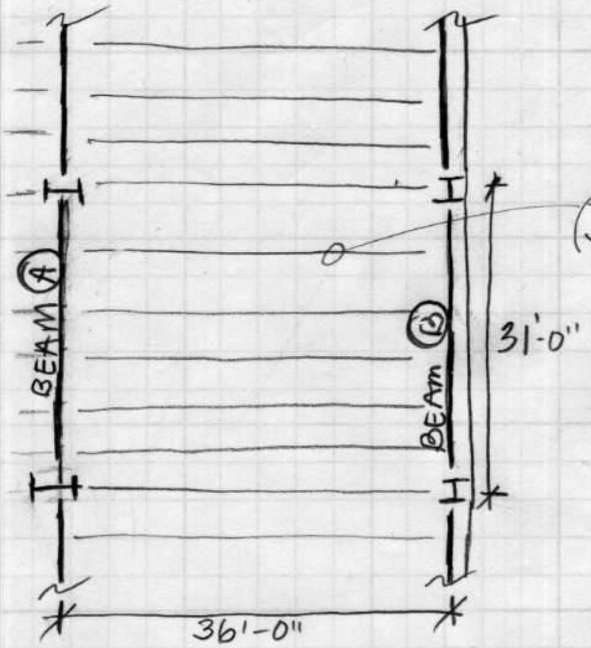
(S.I. D_L = 20 psf is conservative so 24LH08 Joists will be sufficient.)

24LH08 is more economical because it's lighter, but choose 20LH09 if depth is restricted.

Appendix B: Longspan Steel Joists

Hand Calculation:

Design of beams supporting the 24LH08 Steel Joists:



$w_{DL} = 44$ psf (previous page)

$w_{LL} = 80$ psf

$w_u = 1.2(44\text{psf}) + 1.6(80\text{psf}) = 181$ psf.

(Joists spaced @ 5.17' o.c.)

Reference AISC 13th Edition Steel Manual Table 3-2.

Size Beam A:

Trib. width = (15' + 18') = 33 feet. = l_2

$$M_u = \frac{w_u l_2 l^2}{8} = \frac{(181\text{ psf})(33')(31')^2}{8} = 718 \text{ ft-K}$$

=> choose W 24x76

$\phi M_p = 750 > 718 = M_u$

$\boxed{\text{Beam A} \quad W24 \times 76}$

Size beam B

Trib width = 18'

$$M_u = \frac{(181\text{psf})(18')(31')^2}{8} = 392 \text{ ft-K}$$

=> choose W21x48

$\phi M_p = 398 > 392 = M_u$

Beam B W21x48 → After checking deflections on next page, A W21x55 is required.

∴ $\boxed{\text{Beam B} = W21 \times 55}$

Appendix B: Longspan Steel Joists

Hand Calculation:

Deflection Check of beams supporting 24LH08 Joists

$$\Delta_{L\,max} = \frac{L}{360} = \frac{31' \times 12''/ft}{360} = 1.03''$$

$$\Delta_{Total\,max} = \frac{L}{240} = \frac{31' \times 12''/ft}{240} = 1.55''$$

Beam (A) W24 x76: $I_x = 2100\ in^4$

$$\Delta_{Live} = \frac{5 w_L l^4}{384 E I} = \frac{5(80\ psf \times 33\ ft)(31')^4(1728\ in^3/ft^3)}{384(29{,}000{,}000)(2100\ in^4)} = 0.90''$$

$$\boxed{\Delta_L = 0.9'' < 1.03'' = \Delta max \therefore OK✓}$$

$$\Delta_{total} = \frac{5(\overbrace{80+44}^{124})(33\ ft)(31)^4(1728)}{384(29{,}000{,}000)(2100)} = 1.4''$$

$$\boxed{\Delta_t = 1.4'' < 1.55'' = \Delta_{max} \therefore OK✓}$$

Beam (B) ~~W~~ W21x48 : $I_x = 959 in^4$

$$\Delta_L = \frac{5(80\ psf)(18')(31)^4(1728)}{384(29{,}000{,}000)(959)} = 1.08''$$

$\Delta_L = 1.08'' > 1.03''\ \Delta_{L max}$ ∴ Must select larger Beam.

Try W21x55 : $I_x = 1140\ in^4$

$$\Delta_L = \frac{5(80\ psf)(18')(31')^4(1728)}{384(29{,}000{,}000)(1140\ in^4)} = 0.91''$$

$$\boxed{\Delta_L = 0.91\ in < 1.03''\ \Delta_{L max} \therefore OK.✓}$$

$$\Delta_{total} = \frac{5(124\ psf)(18\ ft)(31')^4(1728)}{384(29{,}000{,}000)(1140\ in^4)} = 1.4''$$

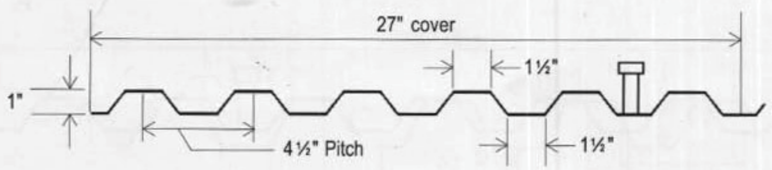
$$\boxed{\Delta_t = 1.4'' < 1.55'' = \Delta_{total\ max} \therefore OK✓}$$

Appendix B: Longspan Steel Joists

Table B.1
Table from the United Steel Deck Design Manual and Catalog of Products: Determination of steel form deck, 22 gage UF1X.

SECTION PROPERTIES						ASD			LRFD		
Metal Thickness		Wt.	I_p	S_p	S_n	V	R_1	R_2	ϕV	ϕR_1	ϕR_2
Gage	Inches	(psf)	(in.^{4})	(in.^{3})	(in.3)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
26	0.0179	1.00	0.039	0.066	0.066	2009	309	396	2387	485	715
24	0.0239	1.25	0.056	0.096	0.096	2906	491	629	3310	731	875
22	0.0295	1.50	0.072	0.127	0.127	3625	715	1349	4073	992	1808
20	0.0358	2.00	0.088	0.163	0.163	4338	971	2181	4927	1339	3013

UF1X



The bottom flange can accept a ¾" shear stud.

approx. scale: 1½" = 1'0"

UNIFORM TOTAL LOAD / Load that Produces l/180 Deflection, psf

	Gage	Span Condition	3'0"	3'6"	4'0"	4'6"	Span 5'0"	5'6"	6'0"	6'6"	7'0"
ASD	26	Single	176 / 126	129 / 80	99 / 53	78 / 37	63 / 27	52 / 21	44 / 16	37 / 12	32 / 10
		Double	174 / 304	128 / 192	98 / 128	78 / 90	63 / 66	52 / 49	44 / 38	37 / 30	32 / 24
		Triple	216 / 238	159 / 150	122 / 101	97 / 71	79 / 51	65 / 39	55 / 30	47 / 23	40 / 19
	24	Single	256 / 182	188 / 114	144 / 77	114 / 54	92 / 39	76 / 29	64 / 23	55 / 18	47 / 14
		Double	253 / 437	186 / 275	143 / 184	113 / 130	92 / 94	76 / 71	64 / 55	54 / 43	47 / 34
		Triple	314 / 342	232 / 215	178 / 144	141 / 101	114 / 74	95 / 56	80 / 43	68 / 34	59 / 27
	22	Single	339 / 233	249 / 147	191 / 98	151 / 69	122 / 50	101 / 38	85 / 29	72 / 23	62 / 18
		Double	334 / 562	246 / 354	189 / 237	150 / 167	121 / 121	100 / 91	84 / 70	72 / 55	62 / 44
		Triple	414 / 440	306 / 277	235 / 186	186 / 130	151 / 95	125 / 71	105 / 55	90 / 43	77 / 35
	20	Single	435 / 285	319 / 180	245 / 120	193 / 85	156 / 62	129 / 46	109 / 36	93 / 28	80 / 22
		Double	427 / 687	315 / 433	242 / 290	192 / 204	155 / 148	129 / 111	108 / 86	92 / 68	80 / 54
		Triple	530 / 538	392 / 339	301 / 227	239 / 159	194 / 116	160 / 87	135 / 67	115 / 53	99 / 42
LRFD	26	Single	279 / 126	205 / 80	157 / 53	124 / 37	100 / 27	83 / 21	70 / 16	59 / 12	51 / 10
		Double	191 / 304	163 / 192	143 / 128	123 / 90	99 / 66	82 / 49	69 / 38	59 / 30	51 / 24
		Triple	217 / 238	186 / 150	163 / 101	144 / 71	124 / 51	103 / 39	86 / 30	74 / 23	64 / 19
	24	Single	405 / 182	298 / 114	228 / 77	180 / 54	146 / 39	121 / 29	101 / 23	86 / 18	74 / 14
		Double	233 / 437	200 / 275	175 / 184	156 / 130	140 / 94	120 / 71	101 / 55	86 / 43	74 / 34
		Triple	265 / 342	227 / 215	199 / 144	177 / 101	159 / 74	145 / 56	125 / 43	107 / 34	92 / 27
	22	Single	536 / 233	394 / 147	302 / 98	238 / 69	193 / 50	160 / 38	134 / 29	114 / 23	98 / 18
		Double	482 / 562	385 / 354	297 / 237	235 / 167	191 / 121	158 / 91	133 / 70	113 / 55	98 / 44
		Triple	548 / 440	470 / 277	368 / 186	292 / 130	238 / 95	197 / 71	166 / 55	141 / 43	122 / 35
	20	Single	688 / 285	506 / 180	387 / 120	306 / 85	248 / 62	205 / 46	172 / 36	147 / 28	126 / 22
		Double	666 / 687	493 / 433	380 / 290	301 / 204	245 / 148	203 / 111	171 / 86	146 / 68	126 / 54
		Triple	821 / 538	610 / 339	471 / 227	374 / 159	304 / 116	252 / 87	212 / 67	181 / 53	157 / 42

Appendix B: Longspan Steel Joists

Table B.2
Table from the CMC Joist and Deck Catalog: Determination of longspan steel joist size, 20LH09 or 24LH08.

LRFD

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

			22-24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
20LH02	10	20	11300	663 306	655 303	646 298	615 274	582 250	547 228	516 208	487 190	460 174	436 160	412 147	393 136	373 126	355 117	337 108	322 101
20LH03	11	20	12000	703 337	694 333	687 317	678 302	651 280	621 258	592 238	558 218	528 200	499 184	474 169	448 156	424 143	403 133	382 123	364 114
20LH04	12	20	14700	861 428	849 406	837 386	792 352	744 320	700 291	660 265	624 243	589 223	558 205	529 189	502 174	477 161	454 149	433 139	412 129
20LH05	14	20	15800	924 459	913 437	903 416	892 395	856 366	816 337	769 308	726 281	687 258	651 238	616 219	585 202	556 187	529 173	504 161	481 150
20LH06	15	20	21100	1233 606	1186 561	1144 521	1084 477	1018 427	952 386	894 351	840 320	790 292	745 267	703 246	666 226	631 209	598 192	568 178	541 165
20LH07	17	20	22500	1317 647	1267 599	1221 556	1179 518	1140 484	1066 438	1000 398	940 362	885 331	834 303	789 278	745 256	706 236	670 218	637 202	606 187
20LH08	19	20	23200	1362 669	1309 619	1263 575	1219 536	1177 500	1140 468	1083 428	1030 395	981 365	931 336	882 309	837 285	795 262	754 242	718 225	685 209
20LH09	21	20	25400	1485 729	1429 675	1377 626	1329 581	1284 542	1242 507	1203 475	1167 437	1132 399	1068 366	1009 336	954 309	904 285	858 264	816 244	775 227
20LH10	23	20	27400	1602 786	1542 724	1486 673	1434 626	1386 585	1341 545	1297 510	1258 479	1221 448	1186 411	1122 377	1060 346	1005 320	954 296	906 274	862 254

LRFD

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	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	17250	513 235	508 226	504 218	484 204	460 188	439 175	418 162	400 152	382 141	366 132	351 124	336 116	322 109	310 102	298 96	286 90
24LH04	12	24	21150	628 288	597 265	568 246	540 227	514 210	490 195	468 182	447 169	427 158	409 148	393 138	376 130	361 122	346 114	333 107	321 101
24LH05	13	24	22650	673 308	669 297	660 285	628 264	598 244	570 226	544 210	520 196	496 182	475 171	456 160	436 150	420 141	403 132	387 124	372 117
24LH06	16	24	30450	906 411	868 382	832 356	795 331	756 306	720 284	685 263	655 245	625 228	598 211	571 197	546 184	522 172	501 161	480 152	460 142
24LH07	17	24	33450	997 452	957 421	919 393	882 367	847 343	811 320	774 297	736 276	702 257	669 239	639 223	610 208	583 195	559 182	535 171	514 161
24LH08	18	24	35700	1060 480	1015 447	973 416	933 388	895 362	858 338	817 314	780 292	745 272	712 254	682 238	652 222	625 208	600 196	576 184	553 173
24LH09	21	24	42000	1248 562	1212 530	1177 501	1146 460	1096 424	1044 393	994 363	948 337	903 313	861 292	822 272	786 254	751 238	720 223	690 209	661 196
24LH10	23	24	44400	1323 596	1284 559	1248 528	1213 500	1182 474	1152 439	1105 406	1053 378	1002 351	955 326	912 304	873 285	834 266	799 249	766 234	735 220
24LH11	25	24	46800	1390 624	1350 588	1312 555	1276 525	1243 498	1210 472	1180 449	1152 418	1101 388	1051 361	1006 337	963 315	924 294	885 276	850 259	816 243

Appendix C: Two-Way Post-Tensioned Concrete Slab

Hand Calculations:

Loads:

Dead Load (not including self weight) ≈ 25 psf.

Live load = 80 psf (corridors above the first floor)

Materials:

Concrete: Normal weight = 150 pcf.

$f'_c = 5{,}000$ psi

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

Rebar = 60,000 psi

PT: Unbonded tendons

1/2"Φ, 7-wire strands, $A = 0.153 \text{ in}^2$.

$f_{pu} = 270$ ksi

Estimated prestress losses = 15 ksi

$f_{se} = 0.70 f_{pu} = 0.70(270) - 15\text{ksi} = 174$ ksi

$P_{eff} = A * f_{se} = 0.153(174\text{ksi}) = 26.6$ K/tendon.

Preliminary Slab thickness:

$\frac{L}{h} = 50$ Longest Span = 36'

$\Rightarrow h = \frac{36' \times 12''/\text{ft}}{50} \rightarrow 8.64'' \Rightarrow$ Try 9"

Loading:

Dead Load of Slab = $150 \text{ pcf} \times \frac{9''}{12''/\text{ft}} = 113$ psf

Dead Load discluding Self weight = 25 psf.

Live Load = 80 psf.

Live Load Reduction per IBC 2006:

K_{LL} slab = 1

$A_T = 36' \times 31' = 1116$.

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

$L = 80\text{psf}\left(0.25 + \frac{15}{\sqrt{1 \cdot 1116}}\right)$

$L = 56$ psf

Appendix C: Two-Way Post-Tensioned Concrete Slab

Based on a 9" Slab thickness

Tendon Ordinate	Tendon Location from bottom of Slab
Ext. support - anchor	4.5"
Int. support - top	8"
Int. span - bottom	1"
End span - bottom	1.75"

$a_{INT} = 8'' - 1'' = 7''$

$a_{End} = \frac{(4.5'' + 8'')}{2} - 1.75'' = 4.5''$

Prestress Force Required to balance 75% of self-weight deadload

The endspan will typically govern the max. required post-tensioning force due to the significantly reduced tendon drape, a_{end}.

$$w_b = 0.75 w_{DL} = 0.75(113\,psf)(31\,ft) = 2627\,plf = 2.627\,klf.$$

Force needed in tendons to counteract the load in the end bay:

$$P = w_b L^2 / 8 a_{end} = \frac{2.627(36')^2}{8 \cdot (4.5/12)} = 1135\,k$$

Check compression allowance

Determine number of tendons to achieve $1135\,k = P$

$$\#\,Tendons = \frac{1135\,k}{26.6\,k/tendon} = 42.7 \rightarrow \text{use } 42 \text{ Tendons}$$

Actual force for banded tendons

$$P_{actual} = (42\,tendons)(26.6k) = 1117.2$$

The balanced load for the end span is slightly adjusted

$$w_{b_{end}} = \frac{1117.2k}{1135k}(2.627\,k/ft) = 2.59\,k/ft$$

Appendix C: Two-Way Post-Tensioned Concrete Slab

Design of E/W Frame : bay width = 31'-0" = 372"

Section Properties :

Design two-way slab as Class U (ACI 18.3.3)
Gross cross sectional properties allowed.

$$A = b \cdot h = (372")(9") = 3348 \text{ in}^2$$

$$S = \frac{bh^2}{6} = \frac{(372)(9")^2}{6} = 5022 \text{ in}^3$$

Allowable Stress for class U:

- At time of jacking (ACI 18.4.1)

$$\text{Compression} = 0.6 f'_{ci} = 0.6(3{,}000) = 1800 \text{ psi}$$

$$\text{Tension} = 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164.3 \text{ psi}$$

- At service loads (ACI 18.4.2(a) and 18.3.3)

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

$$\text{Tension} = 6\sqrt{f'_c} = 6\sqrt{5000} = 424.3 \text{ psi}$$

Target load balances :

Assume 0.75% of DL (self-weight) for slabs.

$$= 0.75 w_D = 0.75(113 \text{ psf}) = 85 \text{ psf}$$

Cover Requirements for 2-hour fire rating (IBC 2006)

Restrained Slabs = 3/4" bottom
Unrestrained slabs = 1½" bottom, 3/4" top.

Average Precompression limits :

$$P/A = 125 \text{ psi min.}$$
$$= 300 \text{ psi max.}$$

Appendix C: Two-Way Post-Tensioned Concrete Slab

Determine actual precompression stress

$$\frac{P_{actual}}{A} = \frac{1117.2K \times 1000}{3348 \text{ in}^2} = 334 \text{ psi} > 300 \text{ psi max.}$$

Therefore will need to use less tendons to prevent overstressing. Check int. span before determining how many less tendons to use.

Check interior span force:

$$P = \frac{(2.627 \text{ K/ft})(30')^2}{8 \times (7''/12'')} = 507 \text{ K} < 1117.2\text{K}$$

A significantly less force is required in the center bay.

$$w_{b_i} = \frac{(1117.2 \text{ K})(9'')(7''/12'')}{30^2} = 6.52 \text{ K/ft.}$$

$$\frac{w_{b_i}}{w_{DL}} = \frac{6.52}{(113\text{psf})(31')/1000} = 186\% \gg 100\%$$

Backsolve to determine sufficient # of tendons to avoid overstressing.

$$\frac{w_{b_i}}{w_{DL}} = 95\% \qquad w_{b_i} = 0.95(3.503) = 3.32 \text{ K/ft.}$$

$$3.32 \frac{K}{ft} = \frac{P_{actual}(9'')(7''/12'')}{30^2} \Rightarrow \underline{\underline{P_{actual} = 570.5 \text{ K}}}$$

Determine number of tendons to achieve 570.5 K.

$$\# \text{ tendons} = \frac{570.5^K}{26.6 \text{ K/tendon}} = 21.4 \rightarrow \text{choose } 22 \text{ tendons.}$$

Actual force of banded tendons.

$$P_{actual} = 22(26.6) = 585.2^K$$

balanced load for end span adjusted:

$$w_{b_{end}} = \frac{585.2^K}{570.5^K}(2.627 \text{ K/ft}) = 2.69 \text{ K/ft.}$$

Appendix C: Two-Way Post-Tensioned Concrete Slab

Determine new precompression stress

$\frac{P_{actual}}{A} = \frac{585.2^K}{3348 \, in^2} = 175$ psi > 125 psi min. O.K✓
< 300 psi max. O.K✓

Check interior span:

$W_{bi} = \frac{(585.2^K)(9")(7"/12")}{30'^2} = 3.41$ k/ft.

$\frac{W_{bi}}{W_{DL}} = \frac{3.41 \, k/ft}{3.503 \, k/ft} = 97.5\% < 100\%$ ∴ acceptable.

East-West Interior Frame:

$\boxed{\text{effective prestress force, } P_{eff} = 585.2^K}$

Dead, Live, and Balancing moments were calculated using PCA Slab Software. See Figure C.1 for results.

Appendix C: Two-Way Post-Tensioned Concrete Slab

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

midspan stresses

$$f_{top} = \frac{(-M_{DL} + M_{BAL})}{S} - \frac{P}{A}$$

S from previous = 5022 in^3

$$\frac{P}{A} = 175 \text{ psi}$$

$$f_{bottom} = \frac{(+M_{DL} - M_{BAL})}{S} - \frac{P}{A}$$

Interior Span:

$f_{top} = [(-123.8 + 108.6)(12)(1000)] / 5022 \text{ in}^3 - 175 \text{ psi}$

$f_{top} = -36 - 175 = -211$ psi compression.

$-211 \text{ psi} < 0.60 f'_{ci} = 0.6(3000 \text{ psi}) = 1800 \text{ psi}$ ∴ O.K.

$f_{bot} = 211 - 175 = 36$ psi tension $< 3\sqrt{f'_{ci}} = 164$ psi ∴ OK.

End Span:

$f_{top} = [(-368.11 + 326.06)(12)(1000)] / 5022 \text{ in}^3 - 175 \text{ psi}$

$= -100.5 - 175 = -275.5$ compression < 1800 psi ∴ OK.

$f_{bot} = 100.5 - 175 = -74.5$ (C) < 1800 psi ∴ OK.

Support Stresses:

$f_{top} = (+M_{DL} - M_{Bal})/S - P/A$

$= (553 - 490.81)(12)(1000) / 5022 - 175$ psi

$= 149 - 175 = -26$ psi (c) < 1800 psi ∴ OK.

$f_{bot} = (-M_{DL} + M_{Bal})/S - P/A$

$= -149 - 175 = -324$ psi < 1800 psi ∴ OK

Appendix C: Two-Way Post-Tensioned Concrete Slab

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

Midspan stresses:

$f_{top} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

Interior Span:

$f_{top} = (-123.8 - 50.42 + 108.6)(12)(1000)/5022 - 175$
$= -157 - 175 = -332\ psi\ (C) < 2250\ psi \therefore OK.$

$f_{bottom} = +157 - 175 = -18\ psi < 2250\ psi \therefore OK.$

End. Span:

$f_{top} = (-368.11 - 149.92 + 326.06)(12)(1000)/5022 - 175$
$= -191.97 - 175 = -367\ psi\ (C) < 2250\ psi \therefore OK.$

$f_{bot} = +191.97 - 175 = 17\ psi\ (T) < 424\ psi \therefore OK$

Support Stresses:

$f_{top} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

$f_{bot} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{top} = (+553.16 + 225.29 - 490.81)(12)(1000)/5022 - 175$
$= +687.31 - 175 = 512\ psi\ (T) > 424\ psi \therefore$ Not good.

Stress at interior support exceeds code allowed limit for tension of $6\sqrt{f'c}$ therefore additional reinforcing around column is needed.

$f_{bottom} = -687.31 - 175 = -862\ (C)\ psi < 1800\ psi \therefore OK.$

Ultimate Strength

Determine factored moments

The primary post-tensioning moments, M_1, vary along the length of the span

$M_1 = P \cdot e$ $\quad e = 0''$ @ ext. support

$e = 3.5''$ @ int. support (N.A. to center of tendon)

$M_1 = (585.2k)(3.5'')/12''$
$= 171\ ft\text{-}k$

Appendix C: Two-Way Post-Tensioned Concrete Slab

The secondary post-tensioning moments, M_{sec}

$$M_{sec} = M_{bal} - M_1$$

$$M_{sec} = 490.81 - 171 = 320 \text{ ft-K} \text{ @ int. Support}$$

Typical load combination for ultimate Strength design:

$$M_u = 1.2\,M_{DL} + 1.6\,M_{LL} + 1.0\,M_{sec}$$

@ Midspan: $M_u = 1.2(368.11) + 1.6(149.92) + 1.0\left(\frac{320}{2}\right)$
$M_u = 842$ ft-K.

@ Support: $M_u = -1.2(553.16) - 1.6(225.3) + 1.0(320)$
$= -705$ ft-K

<u>Determine minimum bonded reinforcement:</u> to see if acceptable for ultimate strength design. Designing only reinforcement for the 36'×31' typical panel.

<u>Positive Moment Region:</u>

End Span: $f_t = 17\text{ psi} < 2\sqrt{f'_c} = 2\sqrt{5000} = 141\text{ psi}$

∴ No positive reinforcement is required. (ACI 18.9.3.1)

<u>Negative moment Region:</u>

$A_{s\,min} = 0.00075\,A_{cf}$ (ACI 18.9.3.3)

Int. Support:

A_{cf} = max of. 9" [(30+36)/2, 31'] * 12

$A_{cf} = 9(33)(12) = 3564\text{ in}^2$

$A_{s\,min} = 0.00075(3564\text{ in}^2)$

$= 2.67\text{ in}^2$ ⇒ try #4 bars because they have the same diameter as the tendons.

$\#\text{bars} = \dfrac{2.67\text{ in}^2}{0.2\text{ in}^2}$ ← x-sec. area of #4 bar.

#bars = 14 bars

(14) #4 top (2.80 in²)

Appendix C: Two-Way Post-Tensioned Concrete Slab

exterior support:

A_{cf} = max of 9" [36/2, 31'] ×12 = 3348
A_{smin} = 0.00075 (3348)
A_{smin} = 2.51 in^2

bars = $\frac{2.51 in^2}{0.2 in^2}$ = 13 bars

(13) #4 Top (2.60 in^2)

- Must span a minimum of 1/6 the clear span on each side of support per (ACI 18.9.4.2)
- Place top bars within 1.5h away from the face of the support on each side (18.9.3.3)
  ⇒ 1.5(9")
  ⇒ 13.5"

Check minimum reinforcement for Ultimate Strength

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

$A_{ps} = 0.153 in^2 \times (22 \text{ tendons})$
$= 3.366 in^2$

$f_{ps} = f_{se} + 10{,}000 + (f'c \cdot b \cdot d)/300A_{ps}$ for slabs with L/h > 35

$f_{ps} = 174{,}000 + 10{,}000 + \frac{5000(31')(12)d}{300(3.366)}$

$\frac{36 \times 12}{9} = 48 > 35$

(ACI 18.7.2)

$f_{ps} = 184{,}000 + 1842\,d$

$a = \frac{A_s f_y + A_{ps} f_{ps}}{(0.85 f'c \times b)}$

@ Supports:

$d = 9" - 3/4" - 1/4" = 8"$
$f_{ps} = 184{,}000 + 1842(8)$
$f_{ps} = 198{,}736$ psi

Appendix C: Two-Way Post-Tensioned Concrete Slab

$$a = \frac{(2.80\ in^2)(60\ ksi) + (3.366\ in^2)(198.736\ ksi)}{0.85\ (5\ ksi)\ (31' \times 12''/ft)}$$

$a = 0.53$

$$\phi M_n = 0.9\left\{\left[(2.80)(60\ ksi) + (3.366)(198.736)\right] \times \left[\frac{8 - \frac{0.53}{2}}{12}\right]\right\}$$

$\phi M_n = 485.5$ ft-K $<$ 705 ft-K

∴ Reinforcement for ultimate strength requirements governs.

Determine $A_{s\,req'd}$:

$$705 = 0.9\left[A_{s\,req'd}\,(60\ ksi) + 3.366\,(198.736)\right]\left(\frac{7.735}{12}\right)$$

$A_{s\,req'd} = 9.1\ in^2$. Try a larger bar size.

Using #8 bars $\Rightarrow A_s = 0.79\ in^2$

$$\#\ bars = \frac{9.1\ in^2}{0.79\ in^2} = 12\ bars = 9.48\ in^2$$

$\boxed{(12)\ \#8\ \text{Top @ Interior Support}}$

To use same size bar for the exterior support.

$$(13)\ \#4 \longrightarrow \frac{2.67\ in^2\ req'd}{0.79\ in^2} = 4\ bars.$$

$\boxed{(4)\ \#8\ \text{Top @ Exterior Support}}$

NOTE:
The typical 36'x31' bay was only designed in the East/West direction for the preliminary analysis to give a rough idea of the feasibility of using post-tensioned design for the building.

FINAL DESIGN:
Slab:
9" Two-way post-tensioned slab

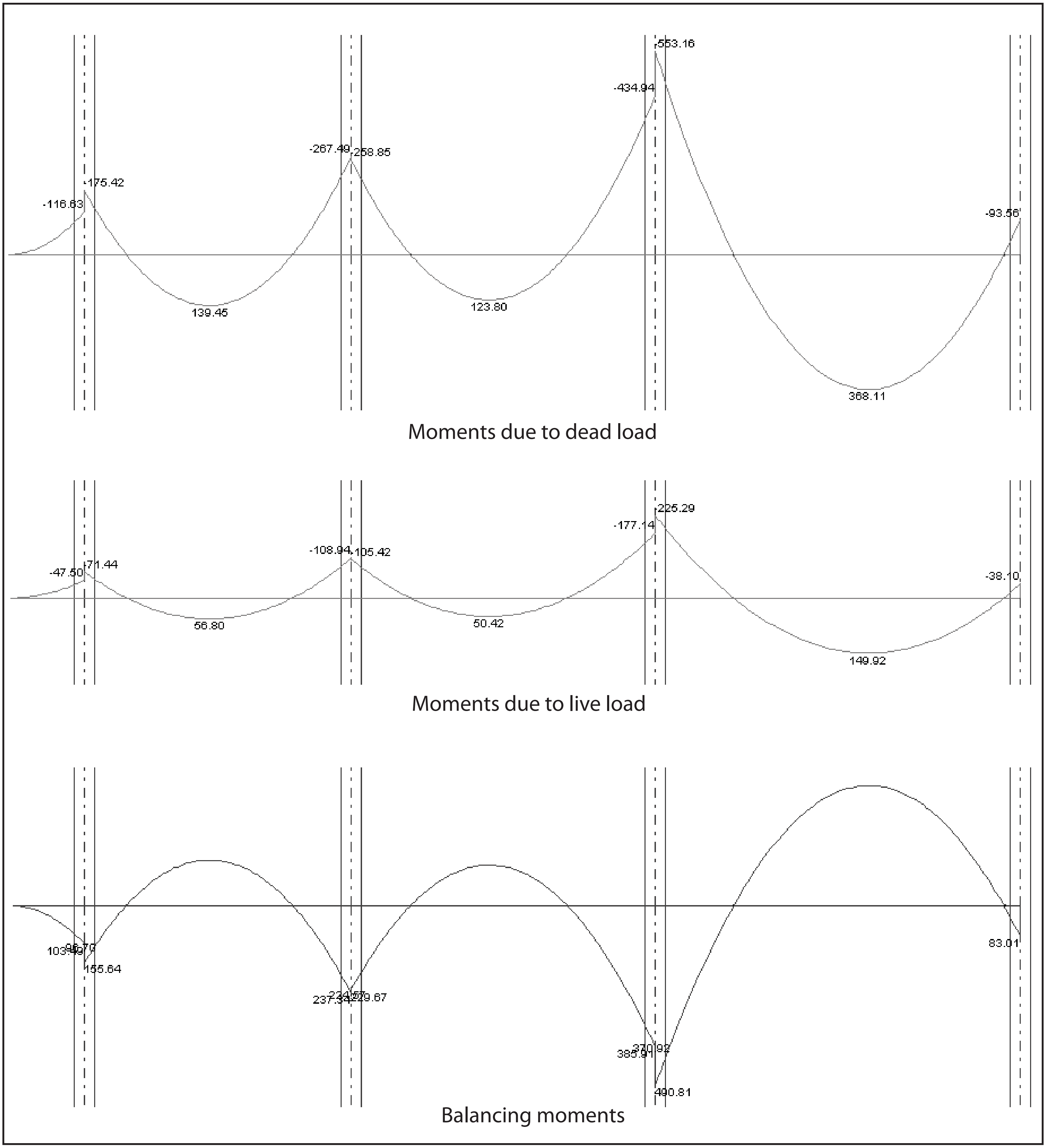
Tendons:
(22) 1/2" diameter, 7-wire strands, banded tendons spanning the E/W frame
Effective prestress force = 585.2 kips

Mild Steel Reinforcing:
(12)#8 top at interior support of typical bay
(4) #8 top at exterior support of typical bay

Sonja Hinish
Structural Option
1100 Broadway, Oakland, CA

Appendix C: Two-Way Post-Tensioned Concrete Slab

Figure C.1
Moment Diagrams computed using PCA Slab software for use in post-tensioned slab design



Appendix D: Precast Hollow-Core Concrete Plank

Hand Notes: Hollow-core plank design

From the 6th Edition PCI Handbook:

<u>Hollow-core concrete plank design:</u>

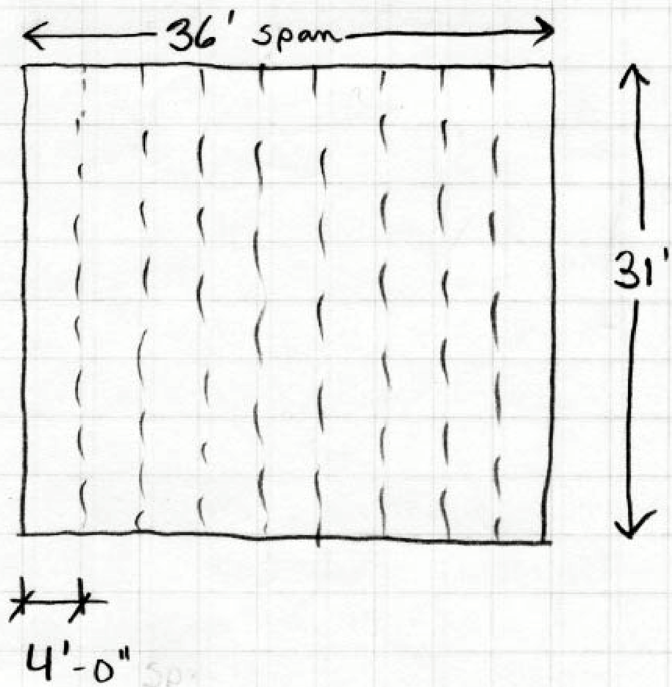
For topped members a superimposed dead load of 15 psf is assumed in the safe load table.

Live Load = 80 psf
S.I. Dead Load = 15 psf
} For a total service load = 95 psf.

Analyzing the 36' span with a service load of 95 psf

⇒ Choose 4'-0" x 10" normal weight hollow-core planks

⇒ With a 2" normal weight topping, Strand designation code = 68-S



Total depth of system = 12"

Appendix D: Precast Hollow-Core Concrete Plank

From PCI Handbook 6th Edition:

Figure D.1
Hollow-Core Properties

HOLLOW-CORE
4'-0" x 10"
Normal Weight Concrete

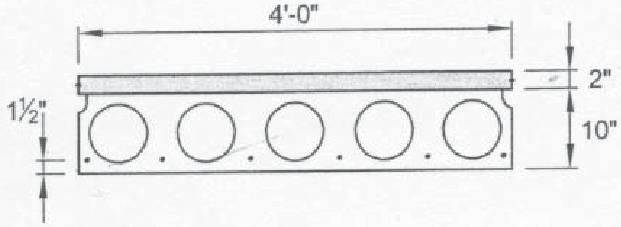
Strand Pattern Designation
48-S

- S = straight
- Diameter of strand in 16ths
- No. of Strand (4)

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

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

Key
258 – Safe superimposed service load, psf
0.3 – Estimated camber at erection, in.
0.4 – Estimated long-time camber, in.



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

Section Properties

	Untopped	Topped
A =	259 in.^{2}	355 in.^{2}
I =	3,223 in.^{4}	5,328 in.^{4}
y_b =	5.00 in.	6.34 in.
y_t =	5.00 in.	5.66 in.
S_b =	645 in.^{3}	840 in.^{3}
S_t =	645 in.^{3}	941 in.^{3}
wt =	270 plf	370 plf
DL =	68 psf	93 psf
V/S =	2.23 in.	

4HC10 + 2

Table D.1

Table of safe superimposed service load (psf) and camber (in.)

2 in. Normal Weight Topping

Strand Designation Code	Span, ft 20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46
48-S	308	287	256	228	204	183	165	148	133	119	107	96	86	74	63	52	43	34	26								
	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4								
	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	[illegible]	[illegible]	[illegible]	–1.0	–1.2	–1.4	–1.7								
58-S	317	298	282	267	252	237	219	198	180	163	148	134	[illegible]	[illegible]	[illegible]	80	69	59	50	41	33	26					
	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	[illegible]	[illegible]	[illegible]	0.2	0.2	0.1	0.0	–0.1	–0.3	–0.4					
	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	[illegible]	[illegible]	[illegible]	–0.5	–0.7	–0.9	–1.2	–1.5	–1.8	–2.1					
68-S	326	307	291	273	258	246	234	222	212	202	188	171	[illegible]	[illegible]	[illegible]	108	96	84	74	64	55	46	38	31			
	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.2			
	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.3	0.2	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.8	–2.2			
78-S	335	313	297	279	267	252	240	228	218	208	196	189	181	165	150	135	122	109	97	86	76	67	58	50	42	35	28
	0.6	0.7	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	0.9	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.8	0.7	0.6	0.5	0.4	0.3	0.1	0.0	–0.2
	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–1.9	–2.3	–2.8
88-S	344	322	306	288	273	258	246	234	221	211	202	195	184	178	172	158	144	130	118	107	96	87	77	68	60	52	44
	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.1	1.1	1.0	0.9	0.8	0.7	0.5	0.3
	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.6	–2.0

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

Appendix D: Precast Hollow-Core Concrete Plank

Hand Calculation: Design of supporting beams

Design of beams supporting hollow-core concrete planks

Reference Figure

Weight of 4'-0" x 10" N.W. concrete plank w/2" topping = 370 plf.

$W_{self} = \frac{370 \text{ lb}}{ft} * \frac{1}{4 ft} = 92.5 \text{ psf}$

$W_{S.I.D.} = 25 \text{ psf}$

$W_{Live} = 80 \text{ psf}$

⇒ Reduced = 56 psf (from post-tension calcs)

Load comb.

$w_u = 1.2D + 1.6L$

$W_u = 1.2(92.5 + 25) + 1.6(56)$

$W_u = 230.6 \text{ psf}$

Size Beam (A)

Trib. width = (28'-8" + 31'-0")/2 = 29.8 ft.

$M_u = \frac{(230.6 \text{ psf})(29.8 \text{ ft})(36')^2}{8} = \underline{1115 \text{ ft-K}}$

AISC 13th Edition Steel Manual → Table 3-2

choose ⇒ W33x99 $\phi M_p = 1170$

$L_b = 36' > 21.4'$ ∴ must use Table 3-10:

Lightest = W24x176

Least Depth = W18x192

$\boxed{\text{Beam (A)} \quad W24 \times 176 \text{ or } W18 \times 192}$

Size Beam (B)

Trib. width = (31'-0" + 20'-0")/2 = 25.5 ft

$M_u = \frac{(230.6 \text{ psf})(25.5')(36')^2}{8} = \underline{953 \text{ ft-K}}$

AISC; Table 3-2

⇒ Choose W30x90 $\phi M_p = 1060$

$L_b = 36' > 20.9'$ ∴ must use Table 3-10;

Most economical = W27x146

$\boxed{\text{Beam (B)} \quad W27 \times 146}$

Appendix D: Precast Hollow-Core Concrete Plank

Hand Calculation: Supporting beam deflection check:

Deflection of Beam (A) Supporting the north end of the hollow-core planks:

$$\Delta_{L\,max} = \frac{L}{360} = \frac{36' \times 12''/ft}{360} = 1.2''$$

$I_{24\times176} = 5680\ in^4$

$I_{W18\times192} = 3870\ in^4$

$I_{W27\times146} = 5660\ in^4$

$$\Delta_{Total\,max} = \frac{L}{240} = \frac{36' \times 12''/ft}{240} = 1.8''$$

$$\Delta_{W24\times176\,(Live)} = \frac{5\,w_L\,l^4}{384\,E\,I} = \frac{5(56psf * 29.8ft)(36')^4(1728\,in^3/ft^3)}{384(29{,}000{,}000\,psi)(5680)}$$

$$\boxed{\Delta_{W24\times176\,Live} = 0.38 < 1.2''\ \Delta_{L\,max}\ \therefore OK\ ✓}$$

$$\Delta_{W24\times176\,Total} = \frac{5(81\,psf * 29.8\,ft)(36')^4(1728)}{384(29{,}000{,}000)(5680)}$$

$$\boxed{\Delta_{W24\times176\,Total} = 0.554'' < 1.8''\ \Delta_{Total\,max}\ \therefore OK\ ✓}$$

If W18×192:

$$\Delta_{18\times192\,(Live)} = \frac{5(56psf \times 29.8)(36)^4(1728)}{384(29{,}000{,}000)(3870)}$$

$$\boxed{\Delta_{18\times192\,Live} = 0.59'' < 1.2\ \Delta_{L\,max}\ \therefore OK\ ✓}$$

$$\Delta_{18\times192\,Total} = \frac{5(81\,psf \times 29.8)(36)^4(1728)}{384(29{,}000{,}000)(3870)}$$

$$\boxed{\Delta_{18\times192\,Total} = 0.82'' < 1.8''\ \Delta_{Total\,max}}$$

Deflection of Beam (B) Supporting the South end of the hollow-core concrete Planks.

$$\Delta_{W27\times146\,(Live)} = \frac{5(56 * 25.5ft)(36')^4(1728)}{384(29{,}000{,}000)(5660)}$$

$$\boxed{\Delta_{W24\times146\,Live} = 0.33'' < 1.2''\ \Delta_{L\,max}\ \therefore\ o.k.\ ✓}$$

$$\Delta_{W24\times146\,Total} = \frac{5(81 \times 25.5)(36)^4(1728)}{384(29{,}000{,}000)(5660)}$$

$$\boxed{\Delta_{W24\times146\,Total} = 0.48'' < 1.8''\ \Delta_{T\,max}\ \therefore\ o.k.\ ✓}$$

Appendix E: Cost Summary

A rough cost per square foot of material estimate for the typical bay was determined using a combination of RSMeans Assemblies Cost Data 2009 and RSMeans Cost Works Online.

System cost summary:

	Material ($/s.f.)	Total ($/s.f)
Steel Composite:		
Composite beams, deck, and slab	24.00	31.35
Longspan Steel Joists:		
Steel joists, beams, and slab	19.00	25.45
Two-Way Post-Tensioned Concrete Slab:		
Cast-in-place (large job) $490.00/c.y. $848.00/c.y.	13.61	23.56
Precast Hollow-Core Concrete Plank:		
Precast plank $10.08/sf $11.54/sf	12.70	23.95
Supporting W16x31 beams $37.50/lf $41.84/lf		