246 West 17th Street

New York, NY

2nd Technical Report

Floor Systems Analysis

October 24, 2008

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Executive Summary

Intent

The purpose of this study was to evaluate alternate floor systems in the design of 246 West 17th Street with the intent to determine which is most feasible. Five systems were analyzed against gravitational loading within a typical bay on a typical floor. A total of six additional considerations of varying significance were evaluated and compared between each of the systems to ultimately pick the most suitable system for the floor design.

Content

Included within this report are descriptions and discussions regarding each floor system. Schematics of the systems as applied to a transverse and longitudinal typical bay are included with each discussion. Ultimately, using by comparing the systems based on the considerations listed above, one of the systems was selected as the most feasible for use in the project.

Results

It was concluded that the two-way concrete flat plate system is the most feasible for use within the 246 West 17th Street typical floor framing plan. This system was the thinnest and lightest overall, and it works best with existing architectural geometry. While other systems might improve in efficiency and depth with an alternate architectural layout, the complexity of the condominiums and the current column layout make this option very difficult. The cost of this system is also comparable to the others, especially when taking depth into account. The deepest system happens to also be the cheapest, but this undesirable depth makes the cost savings less appealing overall. A chart summarizing the comparison criteria and results can be found in the conclusion section.

Please Note
To clearly distinguish between the various structures present in 246 West 17th Street, the terms *existing*, *historic*, and *original* shall refer to the 1925 structure. The terms *current*, *as-designed*, and *new* shall refer to the 2008 renovation design.

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Introduction

Overview

There are two distinct floor types within 246 West 17th Street: existing and new construction. The existing system consists of steel frame with an 8" concrete slab on deck. In many cases, the top flange of the steel (which measures 26" to 28" in depth) is encased in concrete, so the original sizes of most of the beams cannot be determined. Typical beam spacing is 5'-6" on center in the North-South direction; the typical bay size is 20'-8" in the East-West Direction. Girders measure 35'-8" in length, with just two girders spanning the entire length of the building in the North-South direction. These act as long-span transfer girders on the third floor, which is the top of the existing structure and where the first façade set-back occurs. To ensure that the original steel and slab can support this additional concentrated loading, these long-span girders have been reinforced using two parallel support beams beneath. Beams on this level have also been reinforced with diagonal bracing comprised of 4"x4" angles. Alternatively, the new floor system consists of an 8" two-way flat plate concrete slab system. Bay sizes vary slightly in the North-South direction, although they remain regular in the East-West direction, where they measure 20'-8".

Scope

A typical bay was selected on the typical floor and each system was designed according to required live loading by ASCE 7, superimposed dead loads, and system self weight. Five floor systems were analyzed within a typical floor of 246 West 17th Street, including the two-way flat plate system.

Alternative systems that were looked at in this report include a steel non-composite framing system, pre-case concrete hollow core, post-tensioned two-way slab, and open web steel joists with metal deck.

Design Parameters

For design selection, the following system and material were taken into account in each of the systems: strength, fire protection / system rating, overall system thickness, constructability, cost, and serviceability.

Strength

The system must be able to support a required residential live load of 40psf and the superimposed dead load of 20psf, in addition to the weight of the system itself. The factored ASCE 7-05 load combination that governed in all cases was 1.2D + 1.6L.

Fire Protection

A fire rating of 2 hours is required as separation between all residences by ASCE 7-05 for an apartment building. This includes horizontal separations, so all floor systems must meet this 2 hour fire rating.

System Thickness

With a typical floor-to-floor thickness of 10’-8”, and a desired residence ceiling height of 9’-0”, the overall system thickness was limited to 1’-8” to meet this limitation. While not required by code, this is a standard that was set by the design team to ensure that

Constructability

As the project is located in New York City, the construction methods required for the system should be available in the area.

Cost

Overall system cost – including material procurement, installation, and overhead – were calculated for each system per typical bay in the North-South direction. This standardized the comparison so that one may easily see which method is most expensive and which is least expensive.

Serviceability

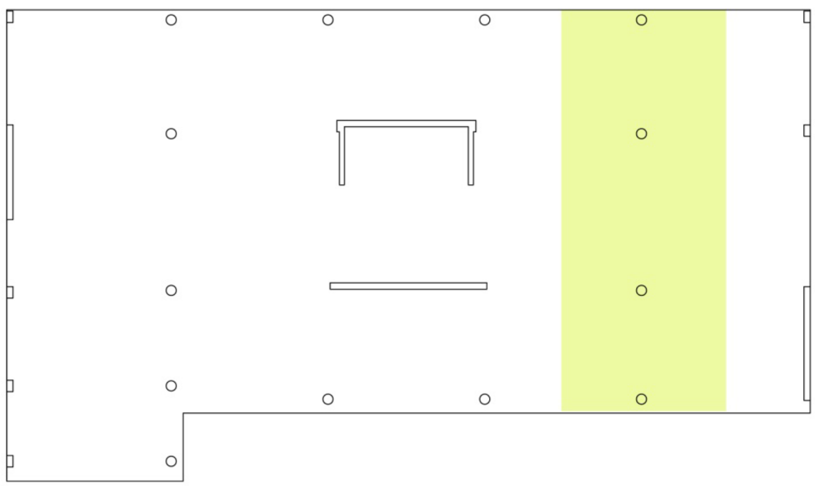
Deflection was limited to L/240 for total service loading and L/360 for unfactored live loading.

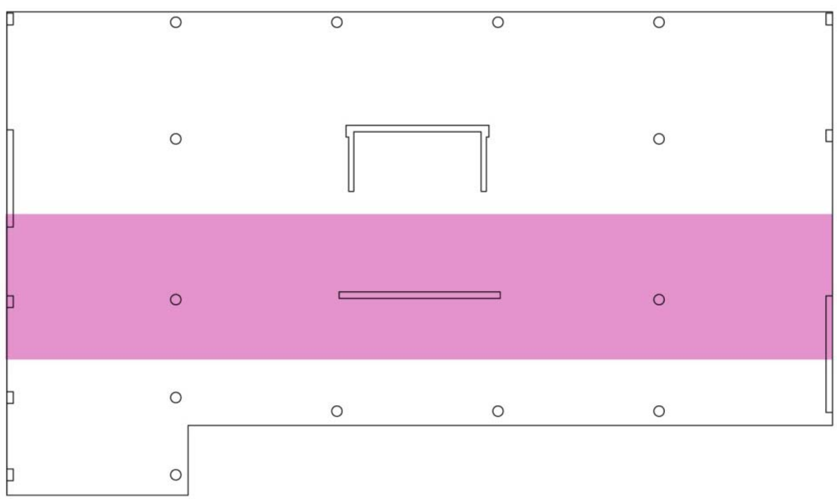
Discussion of Floor Systems

Two-way Flat Plate Slab System

The existing floor system is an 8” two-way flat plate slab system. Transverse and longitudinal reinforcing steel is comprised of #4 bars, and shear heads have been placed above the columns to prevent punching shear failures. Deflection is not an issue here, because the slab thickness meets the minimum required by ACI 318-08 for deflection-controlled design. Detailed calculations of the required steel area can be found in Appendix A.

Below are images of the typical floor plan, with the typical North-South and East-West bays highlighted. Note that the shear wall boundary elements were considered as columns in the calculations for the East-West Typical bay.





Non-Composite Steel Floor System

To preserve the architectural layout of the original design, the column layout was left unchanged. Intermittent beams were added to divide the 20'-8" span in the East-West direction into two equal spans with the intention of eliminating the need for shoring and ultimately cut down on system cost. A 3" Lok-Floor decking system was therefore selected based on a two-span rating that would guarantee that no shoring be required. The overall slab thickness is 5.5", comprised of the 3" deck with 2.5" concrete topping. Normal concrete and a 19-gage deck were selected (in lieu of a 20- or 22-gage deck, although also meeting spanning requirements) so that the system would not be sensitive to floor vibrations.

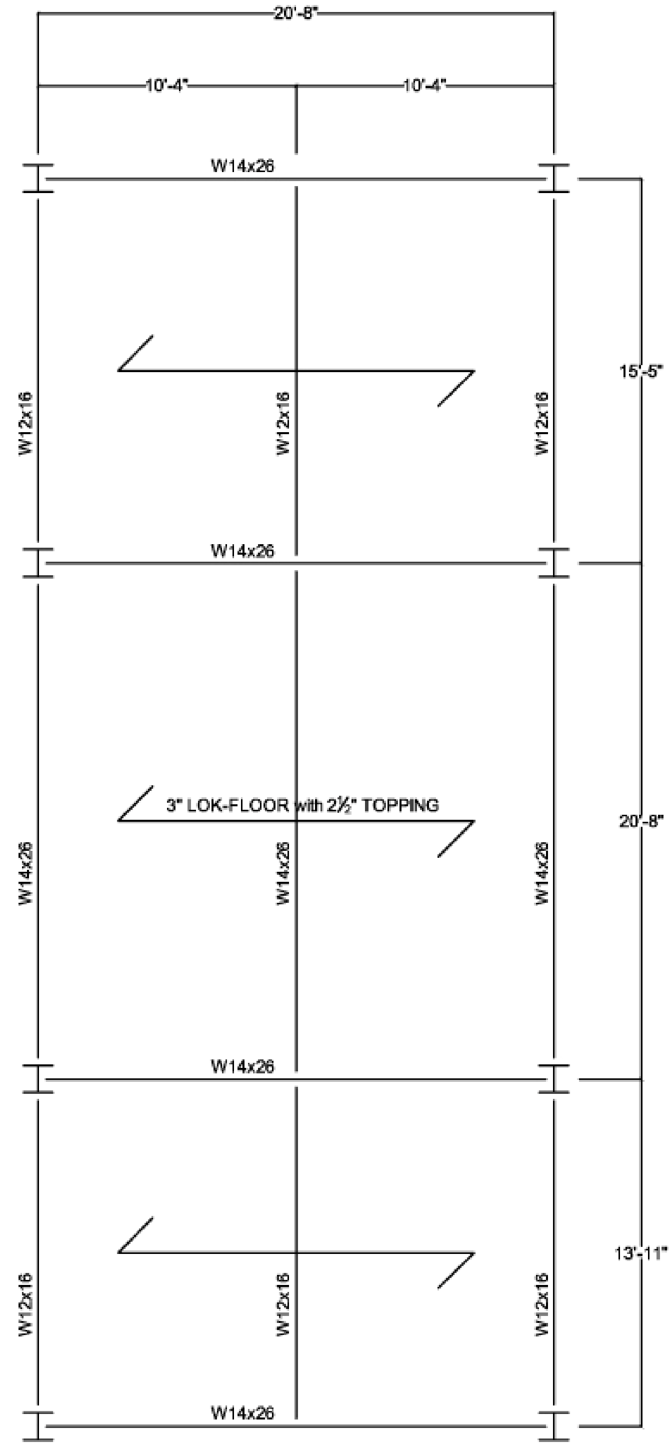
Originally, schematic design included a composite beam system so that the system depth could be reduced to fit within the 18" depth design parameter. With beam spans only ranging between 13'-11" and 20'-8" and with relatively low load requirements, meeting strength requirements was no challenge for an ordinary steel beam system; therefore, composite design was determined to be unnecessary, and shear studs would have simply elevated the overall cost of the system. A summary of material strengths and loadings can be found at below.

The resulting design includes W14x26 girders, W14x26 long beams (designed using the 20'-8" span), and W12x16 shorter beams (designed using the 15'-5" span). While an even shorter beam probably would have been adequate for the shortest span, it made the most sense in terms of constructability to specify one beam size for both of these lengths. These calculations can be found detailed in Appendix B.

Non-Composite Steel Floor System	
Concrete on Metal Deck	normal weight concrete
	f'c = 3,000 psi
	3" Lok-Floor
	19 gage
Steel	Fy = 60,000 psi

Loadings		
Live Load	40	psf
Superimposed Dead Load (SIDL)	20	psf
System Self-weight	50.4	psf

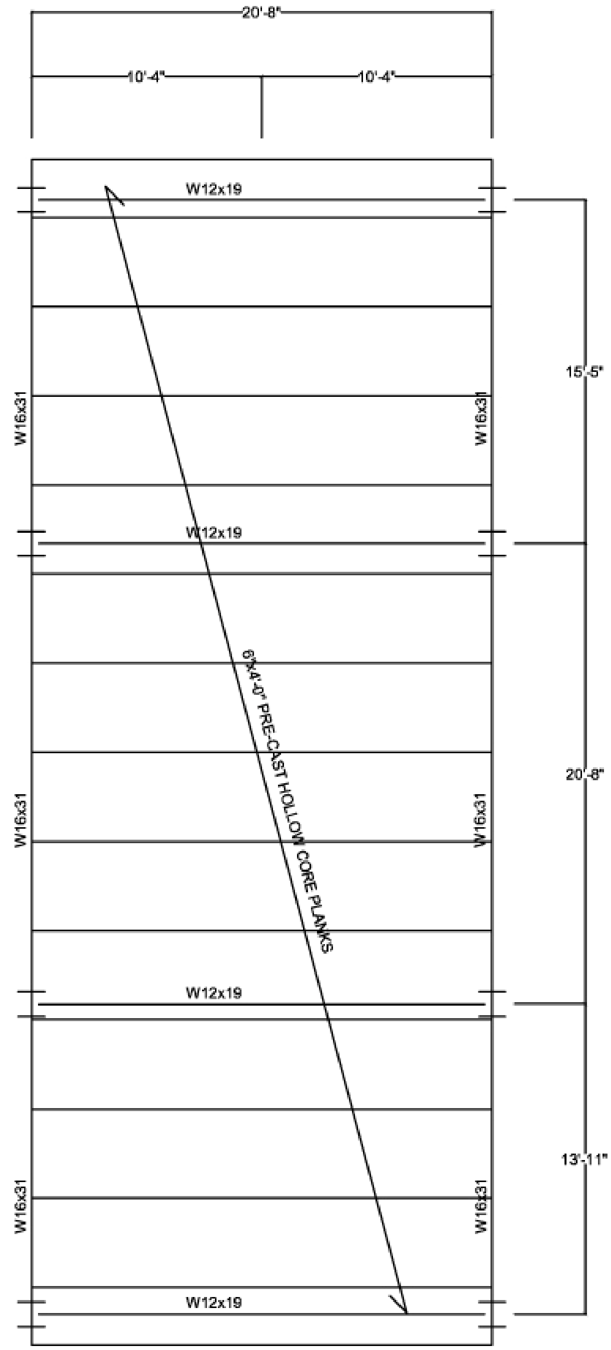
Overall, the steel system is much heavier than the original concrete system, and this would need to be taken into account for foundation loading. Although the soil has relatively high bearing capacity, the new mat slab and spread footings might need to be redesigned to carry a the higher load due to the steel dead weight. In addition, a fireproofing cost estimate could not be obtained, and this would add cost to this already-expensive system. To see the overall cost of the system, see the system comparison chart within the conclusion section. Detailed cost calculations can be found in Appendix E.



STEEL FLOOR FRAMING SYSTEM

Pre-Cast Hollow Core Plank System

The existing column layout was again considered and left in the current location so that the interior architecture would not need to be redesigned. Another important design consideration for the pre-cast hollow core plank (HCP) system was that of the fire rating. The system chosen is a 6"x4'-0 HCP with 2" topping, which has a fire rating of 2 hours, meeting the ASCE 7-05 fire separation requirement. The total concrete depth of this system is 8", leaving 10" for depth of steel. Unfortunately, a beam of this depth could not be utilized without having to seriously increase the weight (and therefore the cost and foundation loads) of the system. After loading and deflection were taken into account, the most efficient girder was determined to be a W16x31, while the most efficient beam was found to be a W12x19. This is still a very heavy, costly system, where foundation loads would need to be re-evaluated and increased in size, adding even more cost to the project. Detailed calculations can be found in Appendix C, and a summary of the loadings and system final design can be found below.



HOLLOW CORE FLOOR FRAMING SYSTEM

Post-Tensioned Two-Way Slab System

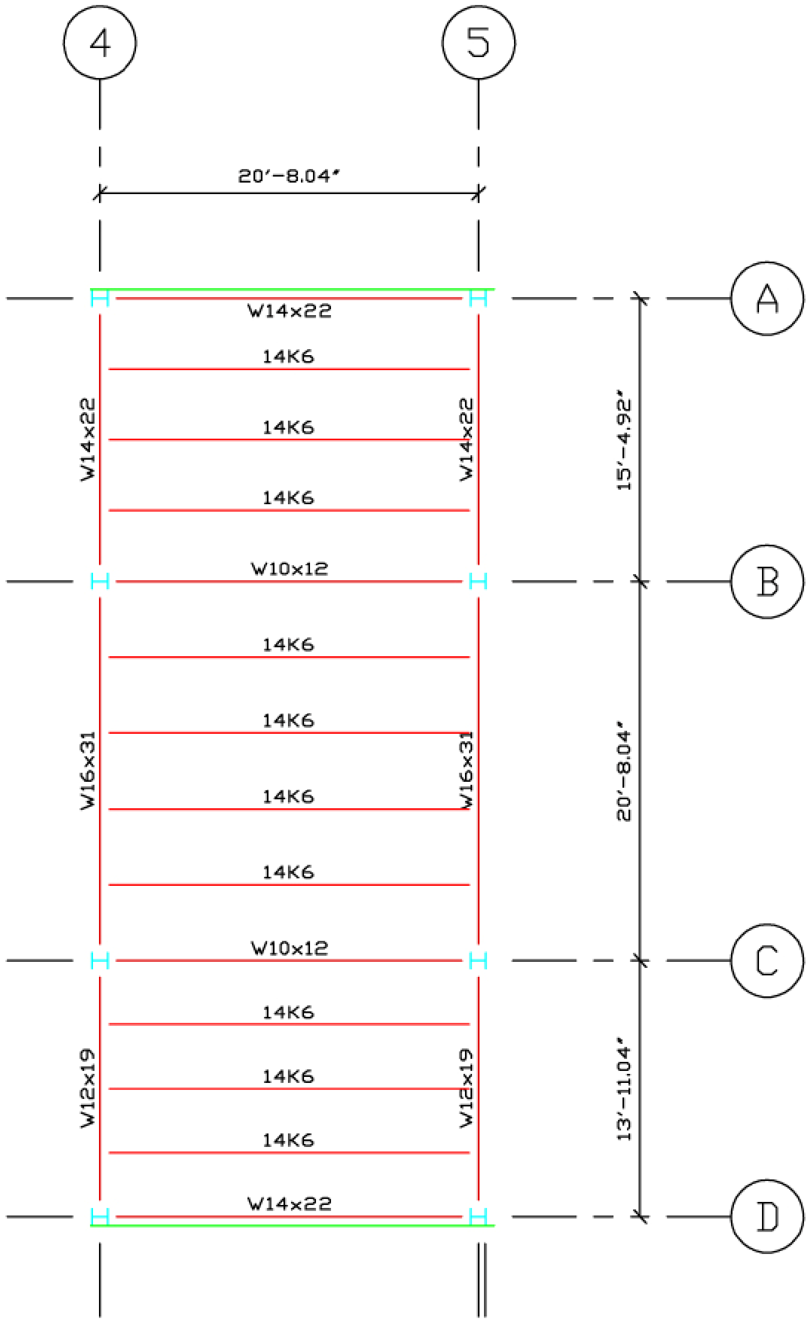
The intention behind the use of post-tensioned (PT) two-way slab design was to decrease the amount of columns required interior of 246 West 17th Street and potentially decrease the slab depth, which would allow for even higher residential ceiling heights, and potentially pull in even more revenue. Calculations began optimistically, with a slab depth requirement of only 6", but unfortunately, even the longest span of 20'-8" in the North-South direction was concluded to be too short for efficient use of this system; the stress did not fall within the limitations of 125 and 300psi. The columns would have needed to be rearranged in order to create a desirable span, and the interior architecture would have to be redesigned. See calculations in Appendix D for Design considerations: span length, thickness

In addition, it has been made clear that the expertise and experience to construct PT two-way slab systems simply does not exist in New York City. This is a system that is rarely used there, and a specialty contractor would have to be brought in for the design. Here, we have a case where the cost implications are high because the constructability is so low.

Open-Web Steel Joist System with Metal Deck

The original column layout was left as-is for the design of this system, and RAM structural system was used in the design. A 6" lightweight concrete slab-on-deck system was chosen in attempts to lessen the joist depth while still lessening the effects of vibration, but the design still resulted in a required 16" joist depth. With a total depth of 22", this system exceeds the 18" set by the design team.

Cost was also the most expensive for this system. It does not include fire proofing, of which a price could not be found, and which would further elevate it above the other systems. The result of the RAM design can be seen on the next page.



TYPICAL BAY within TYPICAL FLOOR

Conclusion

Based on the primary factors of system availability, design strength, thickness, fire rating, and cost, the preferred floor system was found to be that which was originally designed within 246 West 17th Street: the two-way flat plate system. This ranking is based primarily on system thickness, followed by cost and fire rating. All options demonstrated significant strength and constructability (with the exception here being the post-tensioned system, which was disregarded as a feasible option and therefore left out of the comparison). While the hollow core system is significantly less expensive per typical bay, the depth of this system is also the thickest, which is unacceptable as deemed by the owner and original design team.

These comparisons can be found outlined per system in the table below.

SYSTEM COMPARISON	Floor System Type	Construct-ability	Likely to be Used in Residential Construction	Adequate Strength	System Thickness	Fire Rating (Hours)	Cost per Typical Bay	Overall Feasibility Ranking
Original System	**Two-way Flat Plate**	Yes	Yes	Yes	8"	2+	$25,750	**1**
Alternative Systems	**Hollow Core**	Yes	Yes	Yes	8+16 = 24"	2	$15,330	**2**
	Steel with Slab on Metal Deck	Yes	Yes	Yes	14+6 = 20"	2	$24,642	**3**
	Post-tension	No	Yes	n/a	n/a	n/a	n/a	n/a
	Steel Joists with Slab on Metal Deck	Yes	No	Yes	14"	2	$30,180	**4**

Appendix A Two-Way Flat Plate Slab Calculations

CONC.1

FLAT SLAB DESIGN.

1.) check to see if direct design method (DDM) may be used:
by ACI-08 13.6.1: Limitations

13.6.1.1 a) minimum of 3 continuous spans in each direction ✓

13.6.1.2 b) panels shall be rectangular with aspect ratio $\frac{\ell_1}{\ell_2} \leq 2.0$
where $\ell_1 \geq \ell_2$

in our case, most rectangular panel has $\ell_1 = 20.67'$ and $\ell_2 = 13.92'$

$$\frac{\ell_1}{\ell_2} = \frac{20.67}{13.92} \leq 2.0 \quad ✓$$

13.6.1.3 c) successive span lengths (center-to-center) shall not vary by $> \frac{1}{3}$ the longer span.

longest span = 20.67'

shortest adjacent span = 13.92'

$\frac{1}{3}(20.67') = 6.89'$

$20.67 - 13.92 = 6.75' < 6.89'$ ✓

13.6.1.4 d) column offset ≤ 10% span in offset direction

max. offset = 5"
min. span in offset direction = 13.92'

$$\frac{(5''/12)}{13.92} = 0.03 < 0.10 \quad ✓$$

13.6.1.5 e) all loads shall be gravity loads, uniformly distributed, with LL ≤ 2DL

$$40 \leq 2(\underbrace{100}_{\text{assume 8'' slab}} + \overbrace{20}^{\text{superimposed}}) = 240 \quad ✓$$

∴ OK to use DDM.

FLAT SLAB DESIGN BY DDM, cont. CONC. 2

2) Slab thickness

minimum slab thickness unless deflection calculations are carried out, by ACI 318-08

Table 9.5(c) pg 125

knowing $f_y = 60{,}000$ psi with no drop panels } limitation ext. panel w/o edge bms: $\frac{\ell_n}{33}$

int panel: $\frac{\ell_n}{33}$

where ℓ_n = clear span

interior panel will control (much longer length)

$$\frac{20.67'(12'') - 16''}{33} = \frac{248'' - 16''}{33} = \frac{232''}{33} = 7.08''$$

must use 8" slab. $\longrightarrow$ $t_{min} = 8''$

3) loading.

$W_{d,slab} = \left(\frac{8''}{12}\right) 150\ \text{pcf} = 100$ psf — reinf. concr.

$W_{d,superimp} = 20$ psf — M/E, finishes, partitions.

$W_L = 40$ psf — residential

Factored load

$1.2\,W_d + 1.6\,W_L > 1.4\,W_d$ by observation

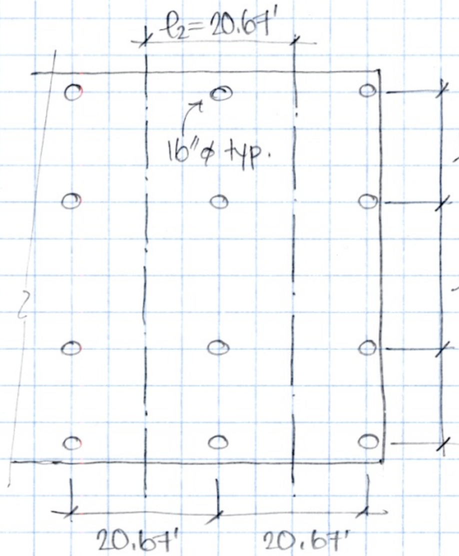
$1.2(100+20) + 1.6(40) = 144 + 64 = 208$ psf

$W_u = 0.208$ ksf

FLAT SLAB DESIGN BY DDM, cont. CONC.3

4A) total factored static moments

N-S Frame



$\ell_n = \ell_1 - 16''$ column diam.

$\ell_{1a} = 15.42'$
$\ell_{na} = 14.09'$

$\ell_{1b} = 20.67'$
$\ell_{nb} = 19.34'$

$\ell_{1c} = 13.92'$
$\ell_{nc} = 12.59'$

$w_u = 0.208$ KSF

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8}$$

$$M_{oa} = \frac{0.208(20.67)(14.09)^2}{8} = 106.7 \text{ k-ft}$$

$$M_{ob} = \frac{0.208(20.67)(19.34)^2}{8} = 201.0 \text{ k-ft}$$

$$M_{oc} = \frac{0.208(20.67)(12.59)^2}{8} = 85.2 \text{ k-ft}$$

5A) column strip / middle strip dimensions

$$a = \tfrac{1}{2}(\text{col. strip width}) \le \frac{\ell_1}{4} \le \frac{\ell_2}{4}$$

use max. values here. for ℓ_1 ∴ use ℓ_{1b}

$$a \le \frac{20.67}{4} \le \frac{20.67}{4}$$

$$a = 5.167'$$

col. strip width = C.S. = 10.335' = b

middle strip width = M.S. = 20.67 − 10.335 = 10.335'

FLAT SLAB DESIGN by DDM cont. CONC 4

6A) Longitudinal Moments.

By ACI 318-08, 13.6.3.2 : For an interior span

$$M^- = 0.65M_o = 0.65M_{ob} = 0.65(201) = 130.7 \text{ 'K}$$
$$M^+ = 0.35M_o = 0.35(201) = 170.3 \text{ 'K}$$

By ACI 318-08, 13.6.3.3 : For an exterior span

$$M^-_{ext} = 0.26M_o = 0.26M_{oa} = 0.26(106.7) = 27.7 \text{ 'K}$$
$$= 0.26M_{oc} = 0.26(85.2) = 22.2 \text{ 'K}$$

$$M^+_{ext} = 0.52M_o = 0.52M_{oa} = 0.52(106.7) = 55.5 \text{ 'K}$$
$$= 0.52M_{oc} = 0.52(85.2) = 44.3 \text{ 'K}$$

$$M^-_{int} = 0.70M_o = 0.70M_{oa} = 0.70(106.7) = 74.7 \text{ 'K}$$
$$= 0.70M_{oc} = 0.70(85.2) = 59.6 \text{ 'K}$$

a			b			c		
-27.7	+55.5	-74.7	-130.7	+170.3	-130.7	-59.6	+44.3	-22.2

7A) Percentage of Pos. Moment going to C.S.

By ACI 318-08, Table 13.6.4.4

Panel a: $\frac{\ell_2}{\ell_1} = \frac{20.67}{15.42} = 1.34$ (b/w 1.0 & 2.0)

$\alpha_{f1} = 0$ b/c no bms. $\therefore \alpha_{f1}\frac{\ell_2}{\ell_1} = 0$

C.S. resists 60% of $M^+_{ext,a}$

Panel b: $\frac{\ell_2}{\ell_1} = \frac{20.67}{20.67} = 1.0$

$\alpha_{f1}\frac{\ell_2}{\ell_1} = 0$

C.S. resists 60% of M^+

Panel c: $\frac{\ell_2}{\ell_1} = \frac{20.67}{13.92} = 1.49$

$\alpha_{f1}\frac{\ell_2}{\ell_1} = 0$

C.S. resists 60% of $M^+_{ext,c}$

FLAT SLAB DESIGN by DDM cont.

CONC 5

8A) Percentage of Neg. Int. Moments going to C.S.

By ACI 318-08, Table 13.6.4.1

Knowing same panel aspect ratios & α_{f1} values as in step (6) } C.S. resists 75% of $M^-_{int,a}$ & c and M^-

9A) Percentage of Neg. Ext. Moment going to C.S.

By ACI 318-08, Table 13.6.4.2

Knowing same panel aspect ratios & α_{f1} values as steps (6) & (7)

Knowing β_t is negligible because there are no edge bms. to resist torsion.

} C.S. resists

100% of $M^-_{ext,a}$

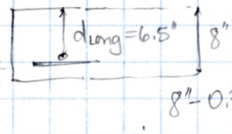
100% of $M^-_{ext,c}$

10A) Summary of Findings:

	a			b			c		
Mo	-27.7	+55.5	-74.7	-130.7	+170.3	-130.7	-59.6	+44.3	-22.2
% to C.S.	100	60	75	75	60	75	75	60	100
C.S. Moment	-27.7	+33.3	-56.0	-98.0	+102.2	-98.0	-44.7	+26.6	-22.2
M.S. Moment	0	+22.2	-18.7	-32.7	+68.1	-32.7	-14.9	+17.7	0

FLAT SLAB DESIGN BY DDM cont CONC. 6

IIA) design of slab Reinf. in C.S.



$d_{\#4}=0.5"$

$8"-0.75"-1.5(0.5")=6.5"$

	a			b			c		
C.S. Moment, M_n	-27.7	+33.3	-56.0	-98.0	+102.2	-98.0	-44.7	+26.6	-22.2
C.S. Slab width, b 10.335' = 124"	124"	124"	124"	124"	124"	124"	124"	124"	124"
Effective depth, d assume #4 bars @12" o/c each way	6.5" (d longitudinal)	6.5"	6.5"	6.5"	6.5"	6.5"	6.5"	6.5"	6.5"
$M_u = \frac{M_n}{\phi}$ $\phi=0.9$	-30.8	37.0	-62.2	-108.9	+113.3	-108.9	-49.7	+29.6	-24.7
$R = \frac{M_u}{bd^2}$	71	85	142	249	260	249	114	68	57
ρ interpol. from Table A-5a in text	0.0012	0.0014	0.0024	0.0043	0.0045	0.0043	0.0019	0.0012	0.0010
$A_{s,req} = \rho bd$	0.97	1.13	1.93	3.47	3.63	3.63	1.53	0.967	0.81
$A_{s,min} = 0.002(bt)$	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98
$A_s \geq A_{s,req} \geq A_{s,min}$	1.98	1.98	1.98	3.47	3.63	3.63	1.98	1.98	1.98
$N_{req} = \frac{A_s}{A_{\#4}}$ $(A_{\#4}=0.20)$	10	10	10	18	19	19	10	10	10
$N_{min} = \frac{b}{2t}$	8	→	→	→	→	→	→	→	→
$N \geq N_{req} \geq N_{min}$	10	10	10	18	19	19	10	10	10
use:	(10) #4			(19) #4			(10) #4		

FLAT SLAB DESIGN BY DDM cont.

CONC 7

12A) design of slab reinf. in M.S.

		a			b			c	
M.S. Moment, M_n	0	+22.2	-18.7	-32.7	+68.1	-32.7	-14.9	+17.7	0
M.S. slab width, b	124"	→							
Effective depth, d	6.5"	→							
$M_u = \frac{M_n}{\phi}$, $\phi = 0.9$	0	+24.7	-20.8	-36.3	+75.7	-36.3	-16.6	+19.7	0
$R = \frac{M_u}{bd^2}$	0	54	48	83	173	83	38	45	0
ρ interpolated from Table A-5a in text	0.0005	0.0010	0.0008	0.0014	0.0030	0.0014	0.0006	0.0008	0.0005
$A_{s,req} = \rho bd$	0.40	0.81	0.64	0.13	2.42	1.13	0.48	0.64	0.40
$A_{s,min} = 0.002(bt)$	1.98	→							
$A_s \geq A_{s,req} \geq A_{s,min}$	1.98	1.98	1.98	1.98	2.42	1.98	1.98	1.98	1.98
$N_{req} = \frac{A_s}{A_{\#2}} = \frac{A_s}{0.20}$ (round up)	10	10	10	10	13	10	10	10	10
$N_{min} = \frac{b}{2(t)} = 7.95$	8	→							
$N \geq N_{req} \geq N_{min}$	← (10) #4 →				(13) #4	← (10) #4 →			

FLAT SLAB DESIGN by DDM cont. CONC. 8

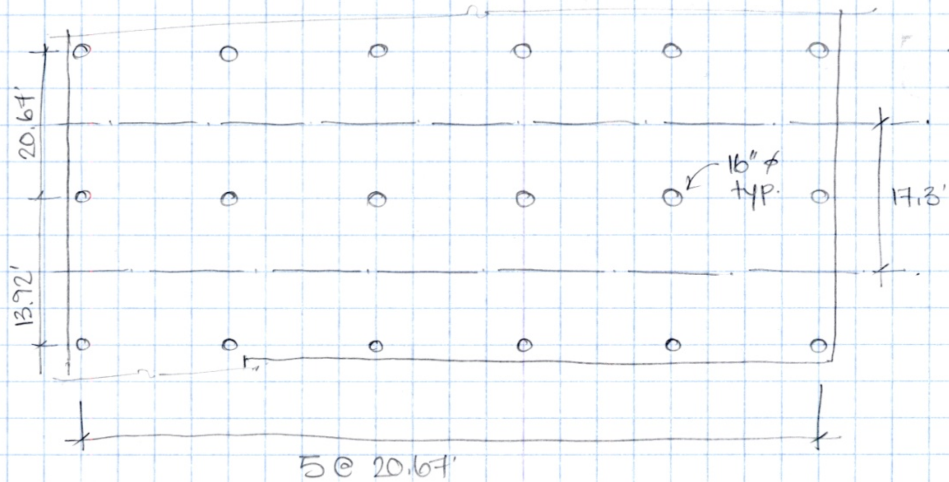
4B) Total Factored Static Moments

$W_u = 0.208$ KSF

E-W frame

$l_1 = 20.67'$ $l_n = 19.34'$

$l_2 = 17.3'$



$$M_o = \frac{W_u l_2 l_n^2}{8} = \frac{0.208(17.3)(17.34)^2}{8} = 168^{1K}$$

5B) Column Strip / Middle Strip Dimensions

$$a \leq \frac{l_1}{4} \leq \frac{l_2}{4}$$

$$\leq \frac{20.67}{4} \leq \frac{17.3}{4}$$

$$a \leq 5.167' \leq 4.325'$$

$$b = \text{C.S. width} = 2(4.325) = 8.65'$$

$$\text{M.S. width} = 17.3 - 8.65' = 8.65'$$

FLAT SLAB DESIGN by DDM cont. CONC 9

6B) Longitudinal Moments $M_o = 168^{1K}$

By ACI 318-08, 13.6.3.2, for an interior span:

$M^- = 0.65 M_o = 0.65(168) = 109.2$
$M^+ = 0.35 M_o = 0.35(168) = 58.8$

By ACI 318-08, 13.6.3.3, for an exterior span:

$M^-_{ext} = 0.26 M_o = 0.26(168) = 43.7$
$M^+_{ext} = 0.52 M_o = 0.52(168) = 87.4$
$M^-_{int} = 0.70 M_o = 0.70(168) = 117.6$

| -43.7 +87.4 -117.6 | -109.2 +58.8 -109.2 | -109.2 +58.8 ... symmetrical spans / loading.

7B) Percentage of Pos. Moment going to C.S.

By ACI 318-08, Table 13.6.4.4

$\frac{l_2}{l_1} = \frac{17.3}{20.67} = 0.84$ (b/w 0.5 & 1.0)

$\alpha_{f1} = 0$ b/c no bms $\therefore \alpha_{f1} \frac{l_2}{l_1} = 0$

C.S. resists 60% of M^+ & M^+_{ext}.

8B) Percentage of Neg. Int. Moments going to C.S.

By ACI 318-08, Table 13.6.4.1

same panel aspect ratios & α_{f1} values as in (6b.) } C.S. resists 75% of M^-_{int} & M^-

9B) Percentage of Neg. Ext. Moments going to C.S.

By ACI 318-08, Table 13.6.4.2

same panel aspect ratios & α_{f1} values as in (6b) & (7b) } C.S. resists 100% of M^-_{ext}.

FLAT SLAB DESIGN by DDM cont

CONC 10

10B) summary of Findings

M_o	-43.7	+87.4	-117.6	-109.2	+58.8	-109.2	-109.2	+58.8
% to C.S.	100	60	75	75	60	75	75	60
C.S. Moment	-43.7	+52.4	-88.2	-81.8	+35.3	-81.8	-81.8	+35.3
M.S. Moment	0	+35.0	-29.4	-27.4	+23.5	-27.4	-27.4	+23.5

11B) design of slab reinf. in C.S.

$d = 8'' - 0.75'' - \frac{1}{2}(0.5'') = 7.0''$

$d_{transverse} = 7''$

C.S. moment, M_n	-43.7	+52.4	-88.2	-81.8	+35.3	-81.8	-81.8	+35.3
C.S. slab width, b $8.65' = 104''$	104"	104"	104"	104"	104"	104"	104"	104"
Effective depth, d assume #4 bars @ 12" o/c ea. way	7"	7"	7"	7"	7"	7"	7"	7"
$M_u = \frac{M_n}{\phi}$ $\phi = 0.9$	-48.6	+58.2	-98.0	-90.9	+39.2	-90.9	-90.9	+39.2
$R = \frac{M_u}{bd^2}$	114	137	231	214	92.3	214	214	92.3
ρ interpolated from Table A-5a in text	0.0019	0.0023	0.0040	0.0037	0.0016	0.0037	0.0037	0.0016
$A_{s,req} = \rho bd$	1.38	1.67	2.91	2.69	1.16	2.69	2.69	1.16
$A_{s,min} = 0.002(bt)$	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66
$A_s \geq A_{s,req} \geq A_{s,min}$	1.66	1.67	2.91	2.69	1.66	2.69	2.69	1.66
$N_{req} = \frac{A_s}{A_{\#4}}$	9	9	15	14	9	14	14	9
$N_{min} = b/2t$	7							

$N \geq N_{req} \geq N_{min} = N_{req}$

DESIGN:	(9)#4	(9)#4	(15)#4	(15)#4	(9)#4	(15)#4	(15)#4	(9)#4

FLAT SLAB DESIGN by DDM cont.

CONC. 11.

12 B) design of slab reinf. in M.S.

M.S. Moment, M_n	0	+35.0	-29.4	-27.4	+23.5	-27.4	-27.4	+23.5
M.S. slab width, b	104"							
Effective depth, d	7"							
$M_u = \frac{M_n}{\phi}$, $\phi = 0.9$	0	+38.9	-32.7	-30.4	+26.1	-30.4	-30.4	+26.1
$R = \frac{M_u}{bd^2}$	0	92	77	72	61	72	61	72
ρ interpolated from table A-5a in text	0.0005	0.0016	0.0013	0.0012	0.0010	0.0012	0.0010	0.0012
$A_{s,req} = \rho bd$	0.36	1.16	0.95	0.87	0.73	0.87	0.73	0.87
$A_{s,min} = 0.002(bt)$	1.66							
$A_s \geq A_{s,min} \geq A_{s,req}$	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66
$N_{req} = \frac{A_s}{A_{\#4}}$, $A_{\#4} = 0.20$	9							
$N_{min} = \frac{b}{2t}$	7							
$N \geq N_{req} \geq N_{min}$	9							

Design: (9) #4

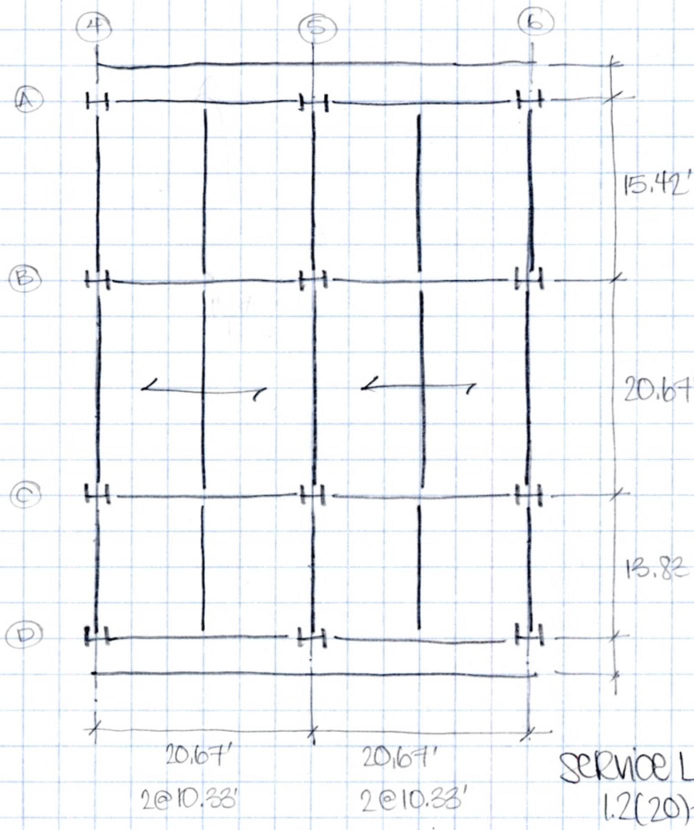
Appendix B Non-Composite Steel Calculations

STL.1

TECH 2

* keeping same layout as to not change architectural layout.

STEEL FRAME DESIGN.



$W_d = 20$ psf $\quad W_L = 40$ psf

$1.2(20) + 1.6(40) = 88$ psf
24 $\quad$ 64

Picking a Deck.

2 span condition
needs to span 10.33'

19 gage LOK-FLOOR
w/ 5.5" slab can
span 11.74' try this

L values, based on:

span	10.0'	10.5'
5.5" slab	260	235

make sure L is less than 235 psf
(L = unif. live service load)

service L_{prov} = ~~40 psf~~ 88 << 235 psf ∴ OK
$1.2(20) + 1.6(40) = 88$ psf

deck dead load: W + W = 48 psf + 2.4 psf = 50.4 psf = $W_{d,deck}$
from catalog

other dead loads: finish + partitions + MEP = 5 + 10 + 5 = 20 psf = $W_{d,FLR}$

total dead load: $W_{d,deck} + W_{d,FLR} = 50.4 + 20 = 70.4$ psf $= W_d$

check allowable $\quad \phi M_n = 72.04$ in-k.
from catalog

$W_u = 1.2W_d + 1.6W_L = 1.2(70.4) + 1.6(40) = 140.5$

$W_u = 141$ psf
$W_u = 141(3') = 0.423$ KLF

$$\phi M_{n,prov} = \frac{W_u \ell^2}{8} = \frac{0.423(10.33)^2}{8}(12"/') = 67.7 \text{ in-k}$$

$\phi M_{n,prov.} = 67.7 < 72.04 = \phi M_{n,allow}$ ∴ OK ✓

Tech 2

STL.2

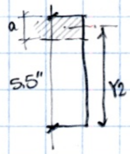
Steel Frame Design, cont.

chose 3" LOK-FLOOR system, normal wt. concrete, w/ SHR. studs

$W_d = 70.4$ psf, $W_L = 40$ psf $\Big\}$ $W_u = 141$ psf   Trib width $= 10.33'$ ∴ $W_u = 141\text{ psf}(10.33') = 1.46$ KLF

BEAM DESIGN: COMPOSITE $M_u = \dfrac{w_u \ell^2}{8} = \dfrac{1.46_{KLF}(20.67)^2}{8} = 78.0$ ft·k

$f'_c = 3$ ksi, $F_y = 50$ ksi $\ell = 20.67'$



assume $a = 1.0"$, then $Y2 = 5.0"$

in this case, only need W10×12, w/ Y1 @ 6 : $\phi M_p = 89.4 > 78.0$

composite is not necessary.

re-check deck w/o studs: 155 psf $\underset{LL}{\text{service}} >$ 88 psf

BEAM DESIGN: NON-COMPOSITE

Found $M_u = 78.0$ ft·k

Find I needed for deflection criteria $\Delta_L \le \dfrac{L}{360}$ $\Delta_T \le \dfrac{L}{240}$

Live: $\dfrac{L}{360} = \dfrac{20.67'(12")}{360} = 0.689"$ 40 psf $(10.33') = 413.2$ PLF $= 0.413$ KLF $= W_L$

$\Delta_L = \dfrac{5W_L\ell^4}{384EI}$ $0.689 \ge \dfrac{5(0.413)(20.67)^4(1728)}{384(29{,}000)I} \longleftrightarrow I_L \ge 84.89$ in^4

Total: $\dfrac{L}{240} = \dfrac{20.67(12")}{240} = 1.03"$ limit to 1"

$\Delta_T = \dfrac{5W_u\ell^4}{384EI}$ $1.0 \ge \dfrac{5(1.46)(20.67)^4(1728)}{384(29{,}000)I} \longrightarrow I_T \ge 206.8$ in^4

I_T controls. I must be ≥ 206.8 in^4 W14×26 $I = 245$ *   W10×39 $I = 209$

STL.3

TECH 2

STEEL FLOOR Frame Design, cont.

Found W14×26 works for deflection.
now check for bending, shear.

CHECK BENDING:

$$M_u = \frac{W_u \ell^2}{8} = \frac{\overset{KLF}{1.46}(20.67')^2}{8} = 78.0 \text{ ft}\cdot\text{k}$$

FOR W14×26, $\phi M_p = 151$ ft·k > 78.0 ft·k ∴ OK.

CHECK SHEAR:

$$V_u = \frac{W_u \ell}{2} = \frac{\overset{KLF}{1.46}(20.67')}{2} = 15.0 \text{ K}$$

FOR W14×26, $\phi V_n = 106 > 15.0$ K ∴ OK ✓

USE W14×26 Bms for 20.67' spans.

DESIGN OF SHORTER BEAMS: $\ell = 15.42'$ $W_L = 0.413$ KLF $W_T = 1.46$ KLF

check deflection requirements:

Live: $\frac{L}{360} = \frac{15.42(12'')}{360} = 0.514''$

$$0.514 \le \frac{5(0.413)(15.42)^4(1728)}{384(29{,}000)\,I_L} \longrightarrow I_L = 35.25 \text{ in}^4$$

Total: $\frac{L}{240} = \frac{15.42(12'')}{240} = 0.771''$

$$0.771 \le \frac{5(1.46)(15.42)^4(1728)}{384(29{,}000)\,I_T} \longrightarrow I_T \ge 83.07 \text{ in}^4$$

I_T controls ∴ I must be ≥ 83.1 in^4 try W12×16 w/ $I = 103$ in^4

STL.4

TECH 2.

Steel Floor Frame Design, cont.

Found W12x16 works for deflection for shorter bms.
now check for bending, shear.

Check Bending: $\ell = 15.42'$ $W_u = 1.46$ KLF

$$M_u = \frac{W_u \ell^2}{8} = \frac{1.46^{KLF}(15.42')^2}{8} = 43.4 \text{ ft·K}$$

For W12x16, $\phi M_p = 75.4$ ft·K $> M_u = 43.4$ ft·K ∴ OK ✓

Check shear:

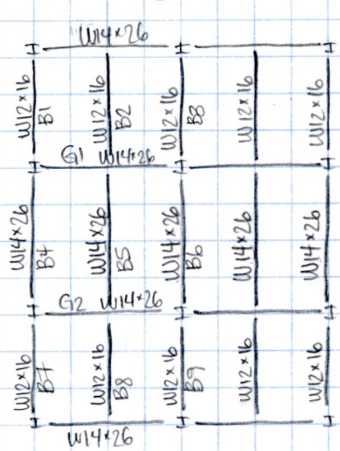
$$V_u = \frac{W_u \ell}{2} = \frac{1.46^{KLF}(15.42')}{2} = 11.8 \text{ K}$$

For W12x16, $\phi V_n = 79.1$ K $> V_u = 11.8$ K ∴ OK ✓

use W12x16 for Bms of 15.42' span.

also use W12x16 for Bms. of 13.92' span. (loading's identical, length is slightly shorter, ∴ slightly conservative ∴ ok.)

Beam Design Summary:



StL.5

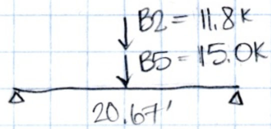
Tech 2.

Steel Floor Frame Design, cont.

Girder design.

Girder G1.

point loads coming from beams B2 & B5



$$B2 = \frac{[(1.2(70.4)+1.6(40))(10.33')]15.42}{2} \quad (W_u)$$

$= 6.7$ k dead
$= 5.1$ k live

$P_u = P_d + P_L$ (factored)

$$B5 = \frac{[(1.2(70.4)+1.6(40))(10.33')]20.67}{2}$$

$= 9.0$ k dead
$= 6.0$ k live.

$P_d = 6.7 + 9.0 = 15.7$ k
$P_L = 5.1 + 6.0 = 11.1$ k

$P_u = 15.7 + 11.1 = 26.8$ k

$$M_u = \frac{PL}{4} = \frac{26.8^{k}(20.67')}{4} = 138 \text{ k}$$

FIRST, CHECK DEFLECTION LIMITS

Live: $\frac{\ell}{360} = \frac{20.67'(12'')}{360} = 0.689''$

divide by factor to get service loads

$$\Delta = \frac{P_L \ell^3}{48EI} = \frac{\left(\frac{11.1}{1.6}\right)^{k}(20.67)^3(1728)^{ft^3}}{48(29{,}000)_{ksi} I} = 0.689 \rightarrow I_L = 110.4 \text{ in}^4$$

Total: $\frac{\ell}{240} = \frac{20.67(12'')}{240} = 1.03$, use 1.0"

$$1.0'' = \left(\frac{15.7}{1.2} + \frac{11.1}{1.6}\right)\frac{(20.67)^3(1728)}{48(29{,}000)I} \rightarrow I_T = 219.5 \text{ in}^4$$

to get service loads

I_T controls $\therefore$ I must be ≥ 219.5 in^4

try W14x26
w/ $I = 245$ in^4

Tech 2

STL.6

Steel Floor Frame Design, continued

Found that W14×26 works for deflection
check shear & bending

Bending:

$M_u = \frac{P_u L}{4} = 138\,k$

ϕM_p for W14×26 = $151^k > 138^k$ ∴ OK

Shear:

$V_u = \frac{P_u}{2} = \frac{26.8}{2} = 13.4\,k$

ϕV_n for W14×26 = $106\,k > 13.4$ ∴ OK

use W14×26 for girder G1

Girder G2: span = 13.92' < 15.42 = span G1.
loading is identical.
use W14×26 for consistency of design
(slightly conservative ∴ OK.)

Appendix C Pre-Cast Hollow Core Calculations

Tech2

HC.1

Hollow Core Floor Design

choose 6" x 4'-0" Plank w/ 2" topping

- meets 2 hr. fire rating required for horiz. separations between residences in R-2 dwelling by IBC

Required Span: 20'-8" → 21'

all options ok for span

Req'd avail. superimposed service load:

superimposed DL = 20 psf
LL = 40 psf
} = 1.2D + 1.6L = 88 psf

@ span = 21', (4) 1/2" diam. strands
SAFE service load (1.2D + 1.6L) = 102 psf > 88 psf

∴ OK

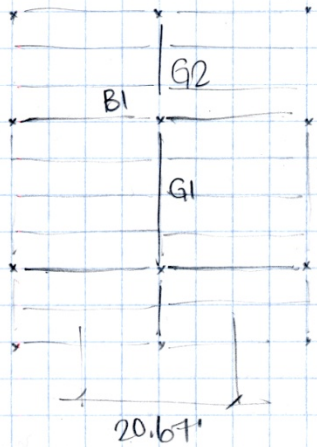
TOTAL Loading

$W_u = 1.2(W_{self} + W_d) + 1.6(W_L)$

$W_u = 1.2(48.75 + 20) + 1.6(40) = 146.5 = 147$ PSF
= 83 + 64

design steel to support.

HC. 2

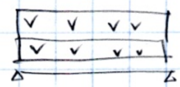


G1: Loading =

W_u(trib. width) = 147(20.67) = 3.04 KLF

3.04 KLF

D: 83
L: 64



1.72 KLF (D)
1.32 KLF (L)

$$\Delta_{max}\ LL = \frac{L}{360} = \frac{20.67'(12'')}{360} = 0.689''$$

$$0.689 \le \frac{5}{384}\,\frac{\left(\frac{1.32}{1.6}\right)(20.67)^4(1728)}{29{,}000\,(I_L)} \qquad I_L \ge 169.6\ in^4$$

~~W14x22~~

$$\Delta_{max}\ TL = \frac{L}{240} = 1.03'' = 1.0''$$

$$1.0 \le \frac{5}{384}\,\frac{\left(\overset{L}{\frac{1.32}{1.6}} + \overset{D}{\frac{1.72}{1.2}}\right)(20.67)^4(1728)}{29{,}000\ I_T} \qquad I_T \ge 319$$

W16x31

Bending $M_u = \frac{W_u L^2}{8} = \frac{3.04(20.67)^2}{8} = 162 < \phi M_p = 203'^k$
∴ OK ✓

Shear $V_u = \frac{W_u L}{2} = \frac{3.04(20.67)}{2} = 31.4 < 131\ \phi V_n$ ∴ OK ✓

HC.3

G2 $\Delta max\ LL = 0.689$

$$0.689 \leq \frac{5}{384}\frac{\left(\frac{1.32}{1.6}\right)(15.42)^4(1728)}{29{,}000\ I_L} \rightarrow I_L \geq 36$$

$$1.0 \leq \frac{5}{384}\frac{\left(\frac{1.32}{1.6}+\frac{1.72}{1.2}\right)(15.42)^4(1728)}{29{,}000\ I_T} \rightarrow I_T \geq 99$$

W12x16

Bending $M_u = \frac{3.04(15.42')^2}{8} = 90.3\ 'K$

$\phi M_p = 75.4\ 'K < M_u$

$\therefore$ D.N.W.

Bending controls.

W12x19 w/ $\phi M_p = 92.6 > 90.3 \therefore$ OK

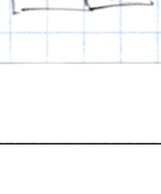
bending OK. b/c $I_x = 130 > 99$

shear:

$$V_u = \frac{w_u L}{2} = \frac{3.04(15.42)}{2} = 23.4$$

$\phi V_n = 85.7 > 23.4 \therefore$ OK.

W12x19. use for other bms also.



HC.4

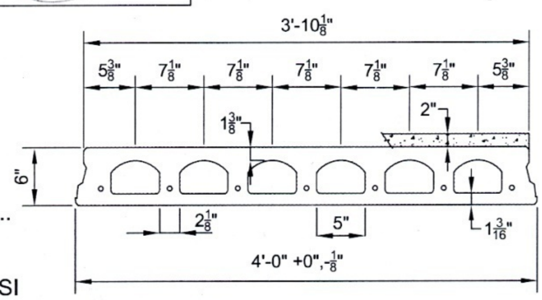
Prestressed Concrete
6"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 253 in.^{2}	Precast S_{bc} = 370 in.^{3}
I_c = 1519 in.^{4}	Topping S_{tc} = 551 in.^{3}
Y_{bc} = 4.10 in.	Precast S_{tc} = 799 in.^{3}
Y_{tc} = 1.90 in.	Wt.= 195 PLF
	Wt.= 48.75 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 67.5 k-ft
   7-1/2"Ø, 270K = 104.2 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2003 & ACI 318-02 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
4 - 1/2"ø	LOAD (PSF)	227	187	360	306	268	229	194	165	141	120	102	86	73	61	50				
7 - 1/2"ø	LOAD (PSF)	367	305	495	455	418	387	340	312	275	243	215	189	167	147	130	114	97	83	70

NITTERHOUSE
CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

05/14/07

6F2.0T

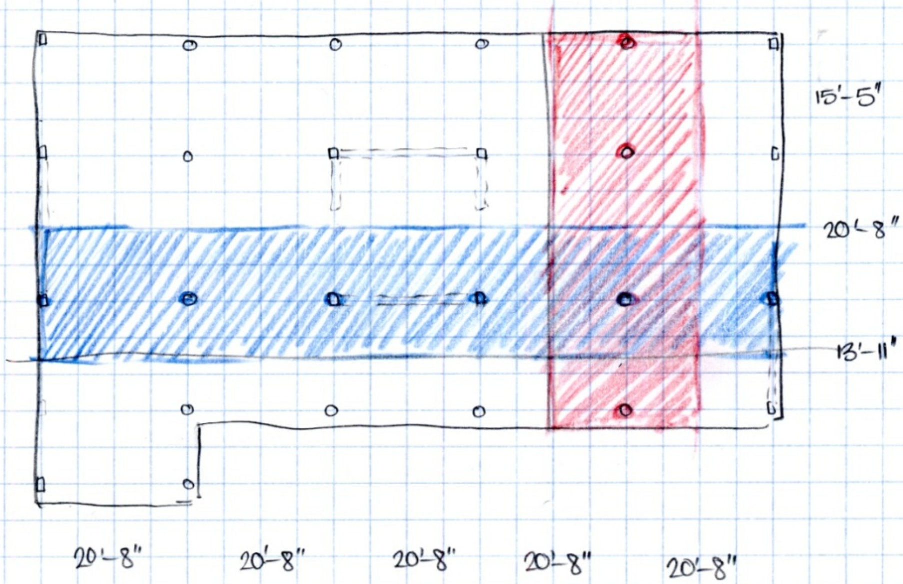
Appendix D Post-Tensioned Two-Way Slab Calculations

PT 1

TECH2

246 W. 17th Street
New York, NY
Alissa Popovich

Two-Way Post-Tension



Loads:

DL Framing $w_{d,self}$

SIDL Superimp. DL: $w_{d,superimp} = 20$ psf partitions, MEP, finishes

LL Live Load: $w_L = 40$ psf residential

Materials:

concrete: NW (150 pcf)

$f'_c = 4000$ psi

$f'_{ci} = 3000$ psi

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

PT: unbonded tendons.

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

$f_{pu} = 270$ ksi

Estimated prestress losses = 15 ksi

$f_{se} = 0.7 f_{pu} - (\text{losses}) = 0.7(270) - 15 = 174$ ksi

$P_{eff} = A f_{se} = \underset{\text{in}^2}{0.153}\underset{\text{ksi}}{(174)} = 26.6 \dfrac{\text{k}}{\text{tendon}}$

1.

PT 2

Prelim. slab thickness:

$\frac{L}{h} = 45 \rightarrow h = \frac{L}{45}$ L = longest span = 20'-8" = 248"

$h = \frac{248''}{45} = 5.51 \rightarrow$ try 6"

$w_{DL} = 75 + 20 = 95$

slab self wt = $150\,pcf\left(\frac{6'}{12^{in/ft}}\right) = 75\,psf$

* NOT TAKING LL REDUCTION INTO ACCT. B/C MOST TRIB. AREAS < 400 SF REQ'D MIN. BY CODE (TO LOOK @ A LL REDUCTION.)

DESIGN OF N-S INTERIOR BAY

- Moments found using EFM.
- Total Bay Width b/w centerlines = 20'-8" = 248" = 20.67'
- No patterned loading req'd b/c $\frac{LL}{DL} = \frac{40}{95} < \frac{3}{4}$ allowed (ACI 18.3.4)

design as class U by ACI 18.3.3; gross cross-sectional prop's

SECTION PROP'S $A = bh = 248(6) = 1488\ in^2$

DESIGN PARAM'S allow stresses by class U.

@ time of jacking:
$f'_c = 3000\,psi$
$C = 0.6f'_{ci} = 0.6(3000) = 1{,}800\,psi$
$T = 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164\,psi$

@ service loading
$f'_c = 4{,}000\,psi$
$C = 0.45f'_c = 0.45(4000) = 1800\,psi$
$T = 6\sqrt{f'_c} = 6\sqrt{4000} = 379\,psi$

avg. precompr. limits
$P/A = 125\,psi$ min $= 300\,psi$ max.

target load balance
60–80% of DL for slabs.
let's use 75% $0.75w_{dself} = 0.75(75) = 56.25\,psf$

cover req'ments: 3/4" top & bottom

PT3

Tendon Profile

	Tendon ordinate	(WRT bottom of slab). Tendon CG location	
(1)	Ext. Support: anchor	3.0	½h
(2)	Int. support: top	5.0	h−1.0"
(3)	Int. span: bottom	1.0	1.0"
(4)	End span: bottom	1.75'	1.75"

$a_{int} = (2)-(3) = 5.0" - 1.0" = 4.0"$

$a_{end} = \frac{(1)+(2)}{2} - (4) = \frac{3.0+5.0}{2} - 1.75 = 2.25"$

e = distance from the center to tendon to the neutral axis; varies along span

Pre-Stress Force Req'd to Balance 75% of the self Wt. DL

$W_b = 0.75 W_{d,self} = 0.75(W_{d,self} \cdot \text{width}) = 0.75(75_{psf})(20.67')$

$W_b - 1.163$ KLF

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

using longer end span

$P = \frac{W_b L_{end}^2}{8 a_{end}} = \frac{1.163^{KLF}(15.42')^2(12)}{8(2.25)} - 184^k.$

check pre-compression allowance.

no. tendons to meet P: $\frac{P_{req}}{p} = \frac{184}{26.6} = 6.92$ use ~~7~~ 6 tendons −n

$P_{actual} = np = 6(26.6) = 159.6$

(adjusted) balanced load for the end span

$W_b = P_{actual}/A \quad \frac{159.6k(1000\frac{lb}{k})}{1488 in^2} = 107.3 < 125$ psi req'd (span very short)

Appendix E Cost Comparison Calculations

System	Unit	Cost/Unit	Units/Bay	Cost/Bay
FLAT PLATE SYSTEM				**$25,751.18**
Normal Weight Concrete	CY	$143	172.25	$24,631.75
Rebar	Ton	$1,800	0.6219	$1,119.43
HOLLOW CORE				**$15,331.89**
Pre-Cast Hollow Core	SF	$9.69	1033.5	$10,014.62
Steel (W16x31)	LF	$44.74	91.35	$4,087.00
Steel (W12x19)	LF	$29.76	41.34	$1,230.28
METAL JOISTS				**$30,181.10**
Joists	LF	$9.83	206.7	$2,031.86
Metal Deck	SF	$3.56	206.7	$735.85
Lightweight Concrete	CY	$147	155.025	$22,788.68
Steel (W16x31)	LF	$44.74	20.67	$924.78
Steel (W14x22)	LF	$38.17	70.68	$2,697.86
Steel (W10x12)	LF	$24.24	41.34	$1,002.08
STEEL				**$24,642.05**
Steel (W14x26)	LF	$38.17	103.35	$3,944.87
Steel (W12x16)	LF	$25.79	58.68	$1,513.36
Metal Deck	SF	$3.56	206.7	$735.85
Normalweight Concrete	CY	$119	155.025	$18,447.98

Steel Framing	Cost/LF	LF/Bay	Cost/SF	SF/Bay	Cost/Bay
Steel Floor System					
W14x26	$38.17	103.35			$3,944.87
W12x16*	$25.79	58.68			$1,513.36
Supporting Metal Joist Floor System					
W16x31	$44.74	20.67			$924.78
W14x22**	$38.17	70.68			$2,697.86
W10x12	$24.24	41.34			$1,002.08
Supporting Hollow Core Floor System					
W16x31	$44.74	91.35			$4,087.00
W12x19*	$29.76	41.34			$1,230.28

*Interpolated between base costs of W12x14 and W12x22 because this was not available, then added labor, equipment, and overhead percentage.
**Used cost of W14x26 because cost for W14x22 was not available