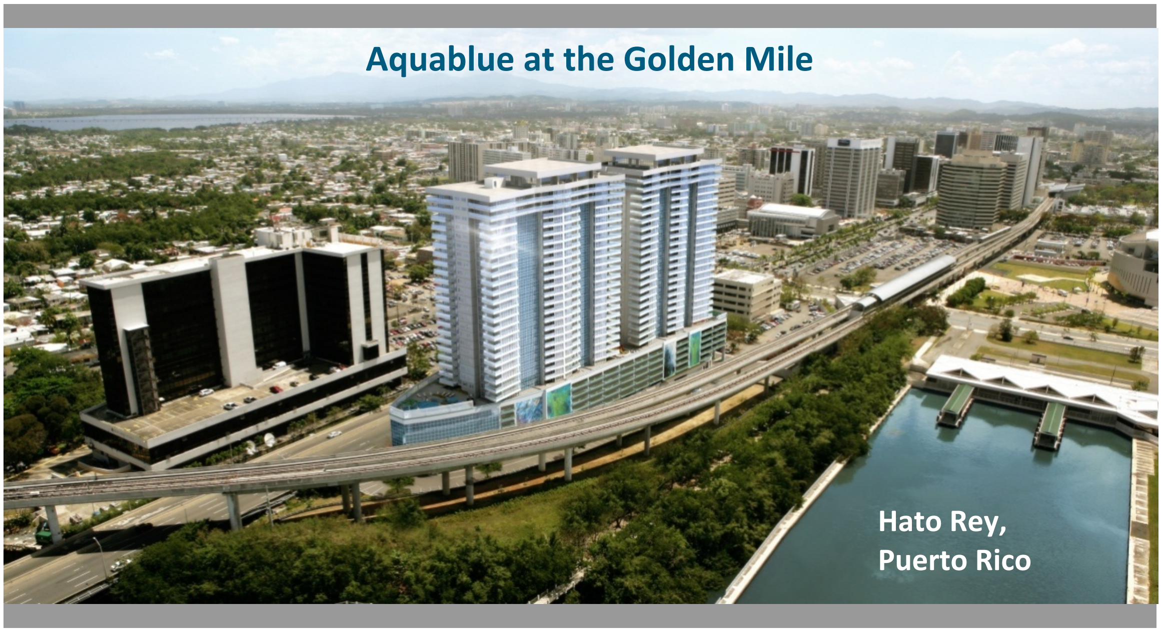
Structural Technical Report 2

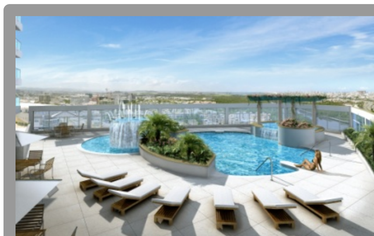


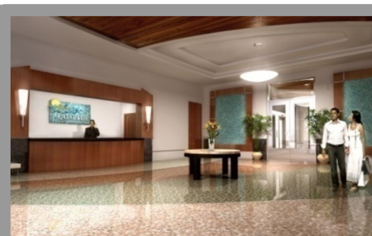
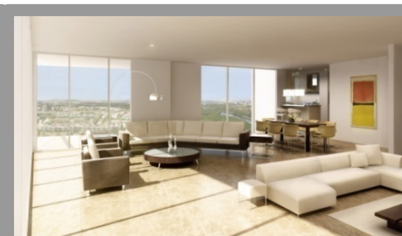
Lindsay Lynch

Structural Option

Dr. Andres Lepage

24 October 2008



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Executive Summary

Aquablue at the Golden Mile is a 31-story apartment building with a parking garage up to level 7. The primary building material for the structure is concrete, and the existing floor design consists of two-way, post-tensioned slabs of varying thicknesses.

The purpose of this report was to research various alternative floor systems and determine preliminary designs. The analyses were based on a regular bay on a typical apartment level. This information (which includes a study of the post-tensioned slab) was used to make a comparison among various systems and to determine which ones are worth investigating further. Other general information was included in the comparison, such as constructability, fireproofing, and the effect of the design on the existing lateral system. The following four systems were studied in this report:

- Post-Tensioned Slab (existing)
- Two-Way Flat Plate Slab
- Steel/Concrete Composite System
- Precast Hollow Core Slab

It was determined that the flat plate slab should not be investigated further. This system is most similar to the existing system, but the comparison suggested that the post-tensioned slab is the better design. The composite system is still a viable option for a re-design of the structure. Based on the system comparison, it did not seem like a potential candidate, but the challenges provide opportunities for breadth studies. For example, the depth of the system would require architectural changes in the floor-to-floor height and façade. The hollow core slab also has the potential for further investigation, because there are many advantages to this precast system. Some of these advantages are the light weight floor, low cost, and ease of construction.

General Building Information

Aquablue at the Golden Mile is a high-rise apartment building in Hato Rey, Puerto Rico. It is located in an urban area, about two miles away from the San Juan Bay (fig. 1). The building size is about 900,000 total square feet, and there are 31 stories above grade. (Up to level 7, the typical floor area is about 51,900 square feet. For the apartment floors, which are above level 7, the typical floor area is about 26,100 square feet.) The ground level will be developed as a commercial area, and the rest of the floors up to level 7 will be used for both parking and office space. Level 7 is an indoor/outdoor public area for the apartment residents, and the floors above are private apartments, which are separated into two towers. There is a sky lobby above the penthouse apartments.

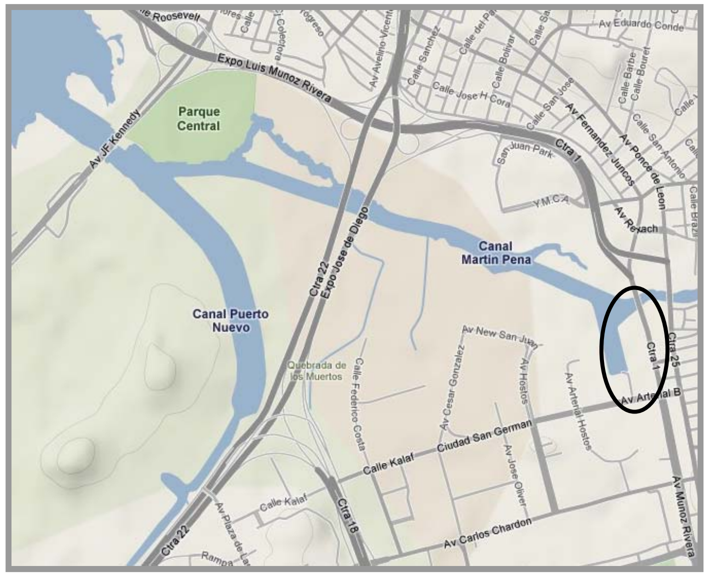


Fig. 1 – Building Site (maps.google.com – Hato Rey Central, PR)

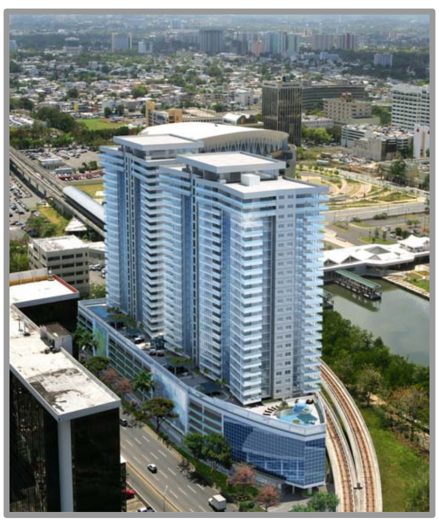


Fig. 2 – Rendering of Aquablue

The parking structure (levels 2-6) is open, with concrete parapets along the exterior. As an architectural feature, there are two sections of an 8" masonry wall that extend from the ground up to level 7. The office areas of these floors are enclosed with a glass curtain wall system, as can be seen at the bottom of figure 2. Above level 7, the primary façade materials are glass and concrete precast panels.

The primary building material is reinforced concrete, and the structure consists of a building frame system with shear walls. Each floor has a post-tensioned slab supported by concrete columns.

Description of Existing Structure

The **foundation** consists of drilled piles that are aligned with the columns. They are the primary foundation system, although there are some grade beams as well. (The grade beams are only used occasionally; they do not span all of the piles.) At the foundation level, there is a 10" reinforced concrete slab.

Each floor consists of a two-way, post-tensioned **structural slab** supported by reinforced columns, which span between 25'-0" and 34'-0". It is a flat plate system, so beams are not a part of the general floor framing. The slabs are 9" thick for the first six stories. At level 7, parts of the slab are 12" thick because the loads are heavier on this partially outdoor level (due to pool and landscaping). For the apartment levels, the post-tensioned slabs are 8" thick.

The primary **lateral force resisting system** is a series of shear walls near the core of the building. They are 18" thick, and most of them require boundary elements. The system of shear walls is grouped into two sections, and each one extends into one of the apartment towers. Figure 3 shows an example of one section of shear walls.

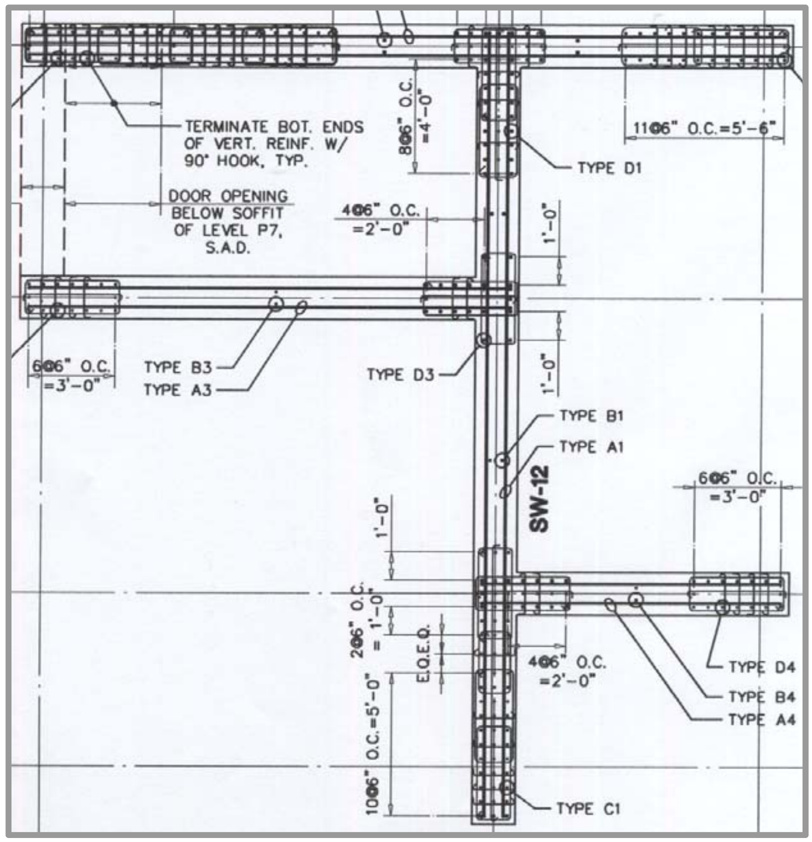


Fig. 3 – Example of Shear Wall System (Levels 7-9)

There is one **expansion joint**, which breaks the building into two sections that are nearly square. It is a 5" seismic joint, and it runs parallel to the short dimension of the building.

The **material strengths** of the concrete for the various structural elements are listed in table 1. The concrete strength of the slabs and columns changes around level 12. The highlighted material strengths are relevant to the floor system being analyzed.

Concrete Material Strengths		
Structural Component		Strength, f'_c (ksi)
pile cap		4
retaining wall / basement wall		4
grade beam		4
slab on grade		5
formed slab	foundation - level 12	6
	above level 12	5
beams		5
parapet / vehicle barrier wall		5
columns	foundation - level 13	8
	above level 13	6

Table 1 – Concrete Strengths for Various Structural Elements

Typical Framing Plan of Existing System

There are two typical floor plans in this building: one for the parking garage levels and one for the apartment levels. In this report, the analyses of floor systems was done for the residential section of the building. The existing floor plan is shown below in figure 4. The two-way, flat plate, post-tensioned slab is 8" thick, and it is supported by rectangular, concrete columns. The lateral system of shear walls is also highlighted in the figure below.

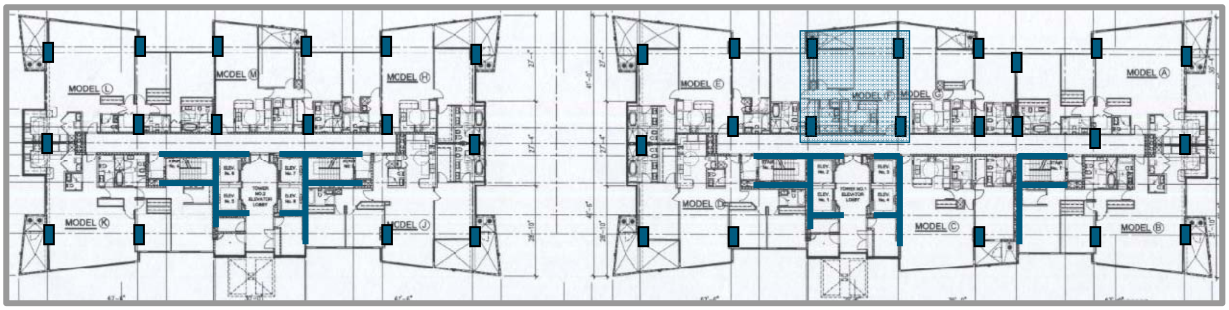


Fig. 4 – Column and Shear Wall Layout for Typical Apartment Level

For the design of the alternative floor systems, a 26'-0" x 29'-0" panel based on the original column layout was used. That panel, shown above in figure 4, was chosen because it was a relatively large bay, and no shear walls interfered with the regular column grid at that location. Below, figure 5 shows an expanded and simplified view of the panel. This same rectangular bay was used throughout the design of the alternative floor systems.

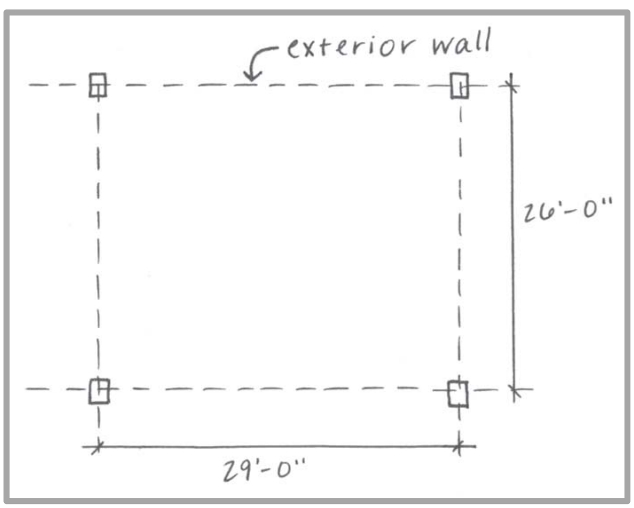


Fig. 5 – Typical Panel Used for Floor System Design

Codes and References

- General References:
    - IBC 2006 (International Building Code)
    - ACI 318-08 (American Concrete Institute)
    - AISC Steel Construction Manual, 13th edition (American Institute of Steel Construction)

- Primary References for Post-Tensioned Concrete Slab Design:
    - PCA 'Time Saving Design Aids' – Two Way Post Tensioned Design (Portland Cement Association)
    - pcaSlab – computer program by the Portland Cement Association

- Primary Reference for Two-Way, Flat Plate Slab Design:
    - pcaSlab – computer program by the Portland Cement Association

- Primary References for Steel/Concrete Composite Floor Design:
    - USD 'Design Manual and Catalog of Products' (United Steel Deck)
    - RAM Structural System – computer program by Bentley

- Primary References for Precast Hollow Core Slab Design:
    - "Precast/Prestressed Concrete Products and Building Systems" (publication by Nitterhouse Concrete Products)
    - PCI Design Handbook, 6th edition (Precast/Prestressed Concrete Institute)

New Floor System Designs

The new designs were all done for the same 26'-0" x 29'-0" panel, and the design loads were kept constant. The superimposed dead load was assumed to be 15 psf, and a live load of 40 psf (reducible) was used. The live load, based on the residential occupancy, was found in ASCE 7-05. When checking the deflections of the systems, the live load deflection was limited to L/360, and the total load deflection was limited to L/240. Vibrations were not considered in this report because they are not a critical issue in residential buildings.

Post-Tensioned Slab (existing system)

A simplified design of the post-tensioned slab was completed using one of the 'Time Saving Design Aids' (Two Way Post Tensioned Design) by the Portland Cement Association. The computer program pcaSlab was also used to determine the moments in the slab under various loading conditions. Toward the end of the analysis, it was found that the interior support is overstressed in tension. It was assumed that additional mild steel reinforcement could be added in that location to make the design feasible.

concrete weight	normal
concrete strength, f'_c	5 ksi
steel strength	60 ksi
banded tendons	26 (1/2"ϕ, 7 wire strands)
slab thickness	8"
fire resistance rating	1 hr

Table 2 – Material Properties

The material properties for the final design can be found above in table 2. The slab thickness was designed to be 8", and 26 draped tendons are required to span along the 26'-0" long column line. The preliminary calculations only involved determining the number of tendons in one direction. In the other direction, the tendons would be evenly spaced across the slab (fig. 7). No mild steel reinforcement is required at midspan, but (4)-#7 bars are required at the top of the exterior support, and (11)-#7 bars are required at the top of the interior support (limited by requirement for ultimate strength). The location of the steel in the slab can be seen in figure 6. The fire resistance rating is obtained by allowing a ¾" cover for the rebar at the top and bottom of the slab.

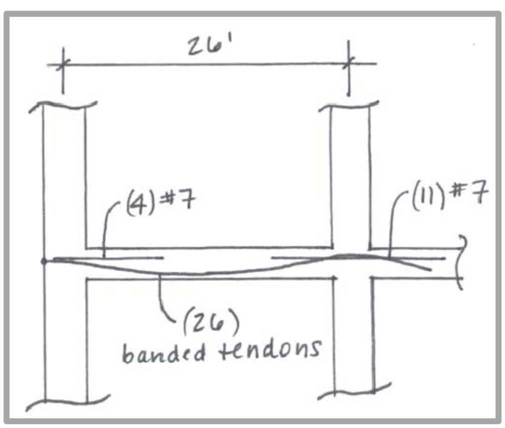


Fig. 6 – Cross Section of Reinforcement

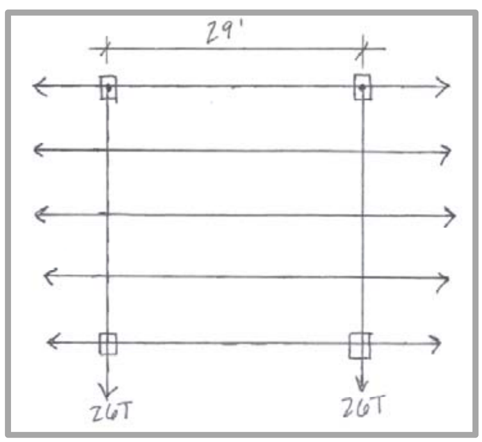


Fig. 7 – Floor Plan with Tendon Locations

Advantages

- Depth: The post-tensioned slab is only 8" deep, which can keep the floor-to-floor height to a minimum.
- Fireproofing: No additional fireproofing is needed as long as the cover requirement of ¾" for the rebar is applied.
- Speed of Construction: The type of concrete used for post-tensioned slabs allows for quicker erection than typical concrete systems.

Disadvantages

- Weight: An 8", normal weight concrete slab weighs about 100 psf, which is heavy compared to some of the other systems.
- Cost: According to RS Means, the post-tensioned slab is one of the more expensive options for the floor system, especially when the labor costs are included.
- Constructability (labor): Post-tensioned slabs are one of the more complicated floor systems, so specialty labor is required.

Two-Way Flat Plate Slab

concrete weight		normal
concrete strength, f'_c	slabs	5 ksi
	columns	6 ksi
steel strength, f_y		60 ksi
slab thickness, t		11"
column dimensions		18"x36"
rebar size		#5

Table 3 – Material Properties and Design Dimensions

The two-way flat plate slab was designed in pcaSlab, a computer program by Bentley for slab design. The program was run for a few different slab thicknesses, but it was determined that the minimum was 11" (based on this exterior panel). The column sizes were kept the same as the original design, because it is likely that there are architectural reasons to limit the dimension in one direction. The summary of design criteria is listed in table 3.

The final design with the slab reinforcement is shown in figures 8 and 9. The designated rebar is placed either in the top or bottom of the slab (for negative or positive moments, respectively) throughout the column and middle strips in each direction.

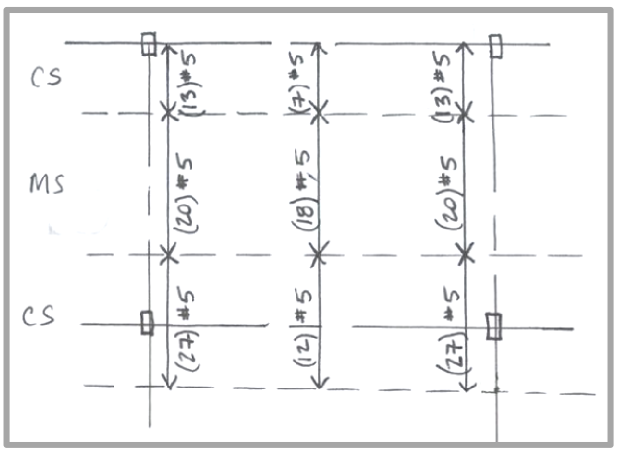


Fig. 8 – Reinforcement for the Horizontal Frames

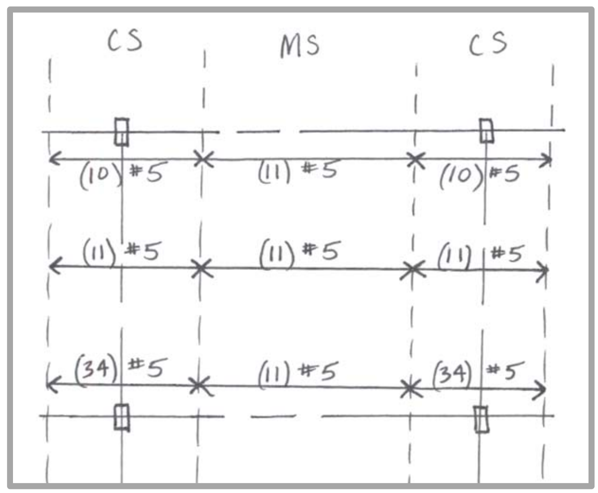


Fig. 9 – Reinforcement for the Vertical Frames

Advantages

- Depth: With the flat plate system (with no drop panels), the entire depth of the structural floor is only 11".
- Fireproofing: No additional fireproofing is needed as long as the cover requirement of ¾" for the rebar is applied.
- Lateral System: The existing lateral system of shear walls could work with the flat plate design (possibly with some minor changes).

Disadvantages

- Weight: Even though there are no beams or drop panels in the system, the weight would be almost 140 psf for an 11", normal weight concrete slab. Over the height of the building, this increased weight would likely affect the foundation design.

Steel/Concrete Composite Floor System

The composite floor system was modeled in RAM Structural System, and the deck properties were chosen from the United Steel Deck design manual. The live load deflection of the deck was limited in the table to L/360, and the deflection of the beams were determined in RAM and checked by hand. In order for the 1-hr fire rating to be achieved, no fireproofing is required on the deck, but sprayed fiber fireproofing is required on the beams. A summary of the composite deck design is shown in table 4.

concrete weight	light weight
concrete strength, f'_c	3 ksi
decking	3" LOK-Floor, 20 gage
slab depth	6"
composite decking weight	43 psf
stud diameter	3/4"
fire resistance rating	1 hr

Table 4 – Properties of Composite Deck

The final floor plan is shown below in figure 10. The bay that is being studied in this report is highlighted in teal, but the surrounding bays were modeled as well. The number of ¾"ϕ diameter studs is shown in parentheses next to each beam designation in the diagram.

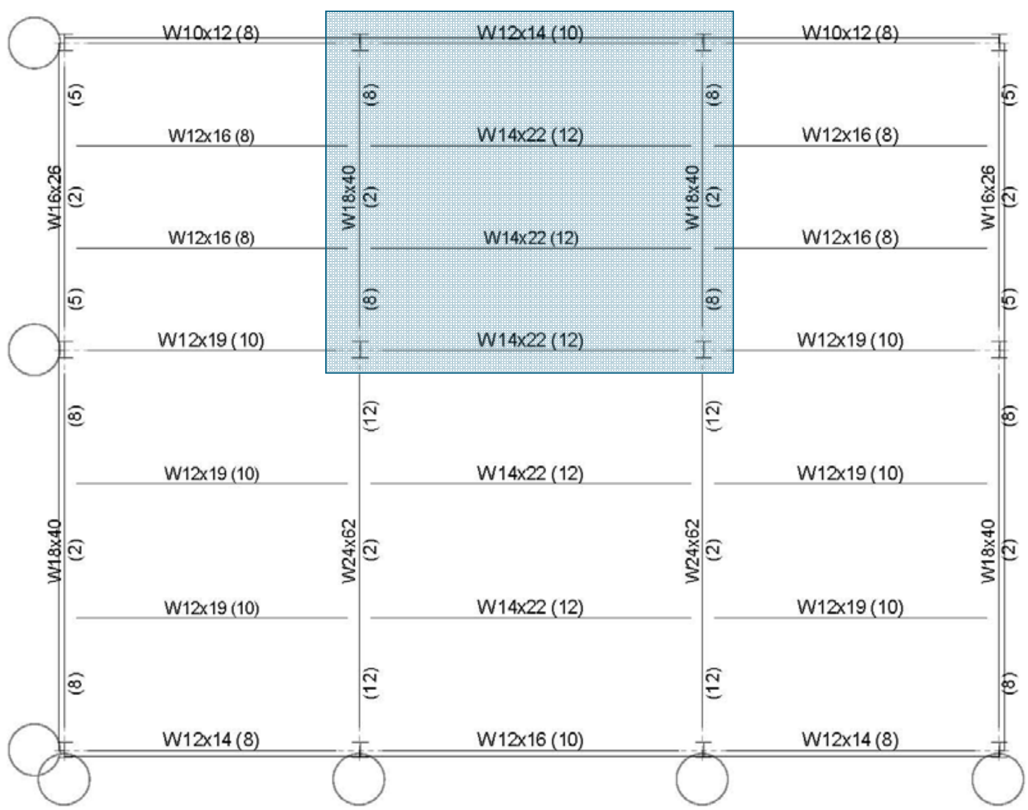


Fig. 10 – Beam Layout of Composite Floor System

Advantages

- Weight: The weight of this floor system is light compared to other potential systems. The composite decking weight is only 43 psf, and the weight of the beams would add less than 5 psf to the overall floor weight. The foundation could be re-designed for the lighter building.

Disadvantages

- Depth: With a 6" composite deck on top of 18" wide flange beams, the total floor depth is 24" (based on the one panel), which is a lot deeper than the current 8" slab. Even with a different beam layout, the depth will not likely be less than 20". This would affect the entire height of the building, because the current floor-to-floor height of the apartment levels is only about 9'.
- Cost: According to RS Means, a composite floor system with these characteristics is one of the more expensive options for floor design.
- Fireproofing: Sprayed fiber fireproofing is needed for the beams.
- Lateral System: A new lateral system would have to be designed to correspond to the steel structure.

Precast Hollow Core Slab

concrete weight	normal
concrete strength, f'_c	6 ksi (at 28 days)
plank size	8" x 4'-0" (untopped)
span	29'-0"
strands	4-1/2"ϕ
slab depth	8"
slab weight	61.25 psf
fire resistance rating	1 hr
supporting beams	W14x74

Table 5 – Properties of Floor System

A publication from Nitterhouse Concrete Products was primarily used to design the hollow core slab system. The plank size was chosen based on the superimposed load and the required fire resistance rating. The planks span the 29'-0" length of the bay, and the beams were sized to span 26'-0" over the column lines. To the left, table 5 summarizes the design and material properties. A sketch of the floor plan is shown below in figure 12, and figure 11 shows a section view of one of the planks.

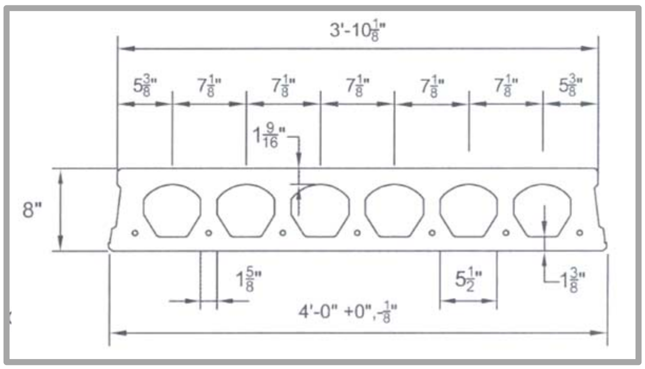


Fig. 11 – Cross Section of Hollow Core Plank from 'Nitterhouse Concrete Products'

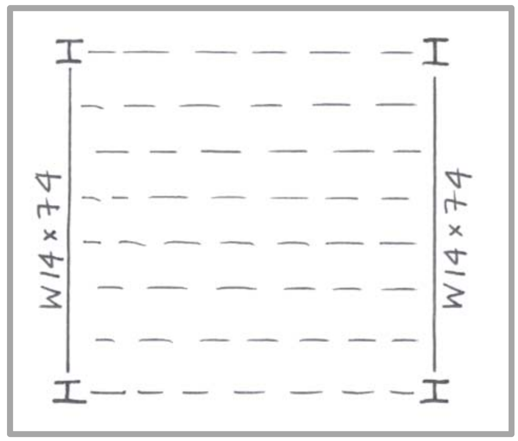


Fig. 12 – Plank and Beam Layout

Advantages

- Weight: The plank is only 8" deep and it is very light. The slab weight is 61.25 psf, and the beams would add less than 3 psf to the overall floor weight. It is possible that the foundation could be re-designed for a lighter building.
- Cost: Precast hollow core planks are relatively inexpensive as compared with other building materials. The total cost (including labor) is also significantly less than most of the other floor systems.
- Constructability: One of the major advantages of hollow core planks is the ease of construction. Once the planks are shipped to the site, they can be installed very quickly and easily.
- Durability: Minimal maintenance is required for this floor design, which is resistant to deterioration.
- Lateral System: The shear wall lateral system could still be used with this design, although there would be some adjustments because the floors are precast concrete and not cast-in-place concrete.

Disadvantages

- Depth: Although the planks are only 8" deep, the supporting wide flange beams increase the floor depth to 22". This would affect the entire height of the building, because the current floor-to-floor height of the apartment levels is only about 9'.
- Fireproofing: Although the planks chosen for this design are already rated for 1 hour of fire resistance, spray-on fireproofing would be required for the steel beams.

Comparison Summary

The following chart (table 6) summarizes the advantages and disadvantages of each system. Seven categories were chosen to evaluate each system, and the scoring is based on a scale of 1-5, with 5 being the best. The total score helps to determine whether or not the system is feasible.

		Post-Tensioned	Flat Plate	Composite	Hollow Core
Depth		5	4	2	2
Weight		4	3	4	5
Cost		3	4	3	5
Constructability	Speed	4	3	3	5
	Labor	2	3	3	5
Fireproofing		5	5	3	3
Impact on Lateral System		5	5	2	4
Impact on Foundation		5	2	4	4
Total		33	29	24	33
Possible alternative?		-	No	Yes	Yes

Table 6 – Comparison Summary

The flat plate slab will not be considered as an alternative floor system because it is similar to the post-tensioned slab, but with a slightly lower score. An investigation of that system would likely prove that the post-tensioned slab was a better design.

Although the composite system had the lowest score, it was chosen to remain as an alternative because of the opportunities it presents. The composite system would cause the most changes to the rest of the building, but that allows for possible breadth studies. For example, the depth of the floor would require architectural changes in the floor-to-floor height and the façade. Also, a new lateral system would have to be designed for the steel structure, and the foundation could be re-designed for a lighter building.

The hollow core slab design is remaining a possible alternative, because it has many advantages and it would make for an interesting comparison between two different systems of the same material.

Appendix A – Post-Tensioned Slab Design

The following calculations were used to determine the preliminary design for the post-tensioned slab. Primarily, the calculations follow the example of a design aid from the Portland Cement Association.

SYSTEM #1 – POST-TENSIONED CONCRETE SLAB
(existing)

primary resources:
- pca Slab (computer program)
- PCA 'Time Saving Design Aids' – Two Way Post Tensioned Design (Portland Cement Association)
- ACI 318-08 (American Concrete Institute)

loads
- live load = 40 psf (reducible)

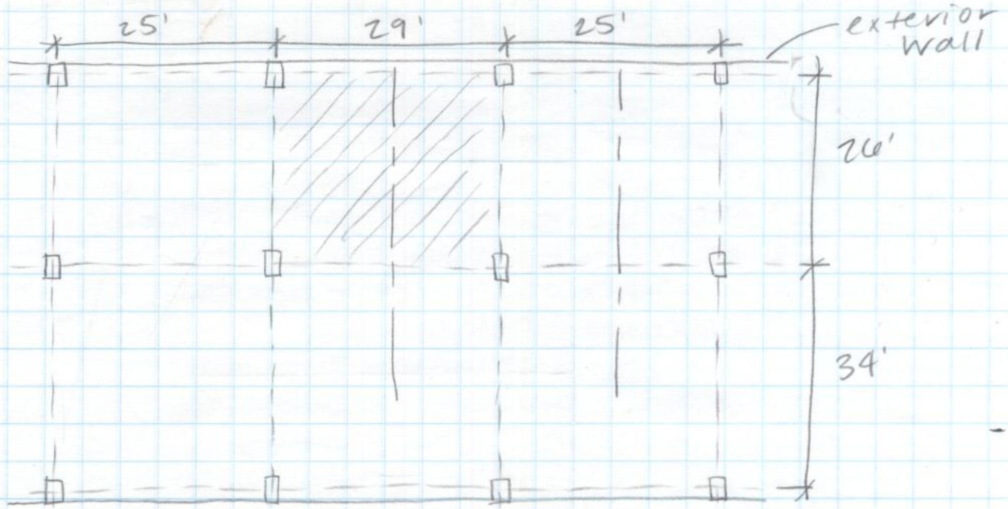
$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

$$= 40\left(0.25 + \frac{15}{\sqrt{1(26\times29)}}\right) = 0.8(40)$$

$$= 32 \text{ psf}$$

- superimposed dead load = 15 psf
- 1-hr fire rating required for floor system (IBC 2006, section 711.3)

materials:
- normal weight concrete
  $f'_c = 5000$ psi
  $f'_{ci} = 3000$ psi
- rebar – $f_y = 60000$ psi
- slab tendons – unbonded
  – ½" φ, 7 wire strands, $A = 0.153 \text{ in}^2$
  – $f_{pu} = 270$ ksi
  – estimated prestress losses = 15 ksi
  – $f_{se} = 0.70 f_{pu} - 15 \text{ ksi} = 174 \text{ ksi}$
  – $P_{eff} = A f_{se} = (0.153)(174) = 26.6 \text{ k/tendon}$



preliminary slab thickness:

$h \approx L/45 = \frac{29 \times 12}{45} = 7.7''$ – use 8" preliminary slab thickness

(29' is longer span for panel being studied)

calculate section properties:

bay width between centerlines $= \frac{29' + 25'}{2} = 27'$

(ignore column stiffnesses)

(no pattern loading required)

$A = bh = (27 \times 12)(8'') = 2590 \text{ in}^2$

$S = \frac{bh^2}{6} = \frac{(27 \times 12)(8)^2}{6} = 3460 \text{ in}^3$

two-way slab designed as Class U

design parameters: (based on class U)

allowable stresses at time of jacking

- $f'_{ci} = 3000$ psi
- compression $= 0.60 f'_{ci} = 0.6(3000) = 1800$ psi
- tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi

allowable stresses at service loads

- $f'_c = 5000$ psi
- compression $= 0.45 f'_c = 0.45(5000) = 2250$ psi
- tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ psi

average precompression limits

- $P/A_{(min)} = 125$ psi (ACI 18.12.4)
- $P/A_{(max)} = 300$ psi

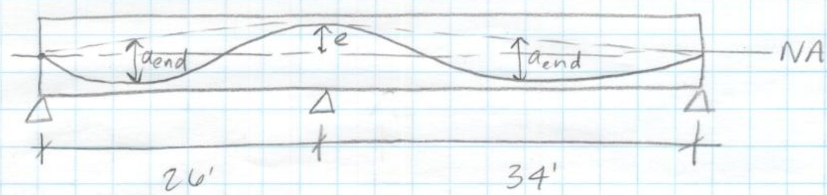
target load balances

- 60% - 80% of DL for slabs
- $DL = 150(8/12) = 100$ psf (self-weight only)
- $0.75(w_{DL}) = 0.75(100) = 75$ psf

cover requirements (1-hr fire rating)

- 3/4" top
- 3/4" bottom (IBC 2006, table 721.2.3)

determine tendon profile:



tendon locations from bottom of slab

- ext. support → 4"
- int. support - top → 7"
- int. span - bottom → 1"
- end span - bottom → 1.75"

$a_{end} = (4+7)/2 - 1.75" = 3.75"$

prestress force required to balance 75% of selfweight DL

- $w_b = 0.75(w_{DL}) = 0.75(100\text{ psf})(27') = 2025$ plf
- force in tendons to counteract load

$$P = w_b L^2 / 8a_{end}$$

$$= \frac{(2.025\text{ klf})(34')^2}{8(3.75/12)}$$

$$= 936\text{ k}$$

check precompression allowance

- determine # of required tendons

$$\# \text{ tendons} = \frac{936k}{(26.6k/\text{tendon})} \cong 35 \text{ tendons}$$

- actual force for banded tendons

$$P_{actual} = (35 \text{ tendons})(26.6) = 931 \text{ k}$$

- balanced load for end span (adjusted)

$$\left(\frac{931}{936}\right)(2.025) = 2.01 \text{ klf}$$

- determine actual precompression stress

$$\frac{P_{actual}}{A} = \frac{(931 \text{ k})(1000)}{2590 \text{ in}^2} = 360 \text{ psi}$$

$360 \text{ psi} > 300 \text{ psi}$ X not good

- check other exterior span

$$P = \frac{(2.025 \text{ klf})(26')^2}{8(3.75/12)} = 548 \text{ k} < 931 \text{ k}$$

(less force required in this bay)

$$w_b = \frac{(931 \text{ k})(8)(3.75/12)}{(26')^2} = 3.44 \text{ klf}$$

(other tendons continue into this bay)

$$w_b/w_{DL} = 3.44/(0.100 \times 27) = 127\%$$ X too high

$w_b = 2.57$ (if 95% of w_{DL})

$P_{tendons} = 695$ k (in order to limit w_b to 2.57 klf)

$$\# \text{ tendons} = \frac{695}{26.6} \cong 26 \text{ tendons}$$

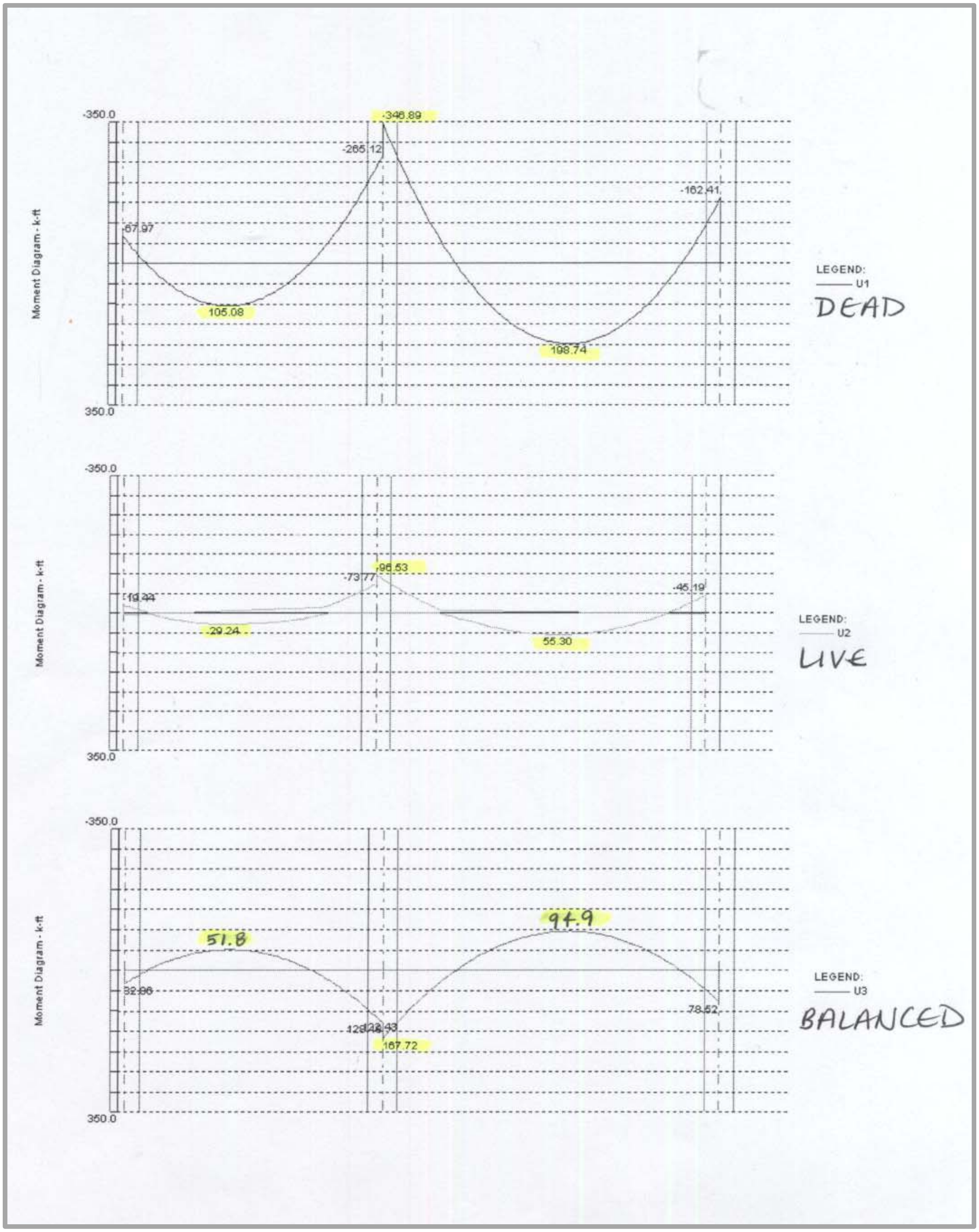
$$P_{actual} = (24 \text{ tendons})(26.6) = \boxed{692 \text{ k} = P_{eff}}$$

$$\text{adjusted balanced load} = \frac{692}{936}(2.025) = 1.50 \text{ klf}$$

actual precompression stress

$$\frac{P_{actual}}{A} = \frac{(692 \text{ k})(1000)}{2590 \text{ in}^2} = 267 \text{ psi}$$

$125 \text{ psi} < 267 \text{ psi} < 300 \text{ psi}$ ✓ OK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-67.97
105.08
198.74
350.0
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DEAD
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29.24
55.30
350.0
Moment Diagram - k-ft
LEGEND:
U2
LIVE
-350.0
51.8
94.9
32.86
78.52
167.72
350.0
Moment Diagram - k-ft
LEGEND:
U3
BALANCED

<u>stresses immediately after jacking:</u>

- end span → 26' (<u>midspan</u> stresses)

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

$$= \frac{(-105.1 + 51.8)(12000)}{3460\ in^3} - 267\ psi$$

$$= -452\ psi \text{ compression} < 0.6 f'_{ci} = 1800\ psi \checkmark$$

$$f_{bot} = \frac{(M_{DL} - M_{bal})}{S} - \frac{P}{A}$$

$$= \frac{(105.1 - 51.8)(12000)}{3460} - 267$$

$$= -82.1\ psi \text{ compression} < 0.6 f'_{ci} = 1800\ psi \checkmark$$

- end span → 34' (<u>midspan</u> stresses)

$$f_{top} = \frac{(-198.7 + 94.9)(12000)}{3460} - 267$$

$$= -627\ psi \text{ compression} < 0.6 f'_{ci} = 1800\ psi \checkmark$$

$$f_{bot} = \frac{(198.7 - 94.9)(12000)}{3460} - 267$$

$$= 93\ psi \text{ tension} < 3\sqrt{f'_{ci}} = 164\ psi \checkmark$$

- interior <u>support</u> stresses

$$f_{top} = \frac{(346.9 - 167.7)\,12000}{3460} - 267$$

$$= 355\ psi \text{ tension} > 3\sqrt{f'_{ci}} \times$$

(additional reinforcement is needed around interior support)

$$f_{bot} = \frac{(-346.9 + 167.7)\,12000}{3460} - 267$$

$$= -889\ psi \text{ compression} < 0.6 f'_{ci} \checkmark$$

$$\left[f_{top} = \frac{(M_{DL} - M_{bal})}{S} - \frac{P}{A} \qquad f_{bot} = \frac{(-M_{DL} + M_{bal})}{S} - \frac{P}{A} \right]$$

<u>stresses at service load</u>

– end span → 26' (<u>midspan</u> stresses)

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

$$= \frac{(-105.1 - 29.2 + 51.8)12000}{3460} - 267$$

$$= -286 - 267$$

$= -553$ psi compression $< 0.45f'_c = 2250$ ✓

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

$$= 286 - 267$$

$= 19$ psi tension $< 6\sqrt{f'_c} = 424$ psi ✓

– end span → 34' (<u>midspan</u> stresses)

$$f_{top} = \frac{(-198.7 - 55.3 + 94.9)12000}{3460} - 267$$

$$= -552 - 267$$

$= -819$ psi compression $< 0.45f'_c$ ✓

$$f_{bot} = 552 - 267$$

$= 285$ psi tension $< 6\sqrt{f'_c}$ ✓

– interior <u>support</u> stresses

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

$$= \frac{(346.9 + 96.5 - 167.7)12000}{3460} - 267$$

$$= 956 - 267$$

$= 689$ psi tension $> 6\sqrt{f'_c}$ ✗ (additional reinforcement needed at support – already determined)

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

$$= -956 - 267$$

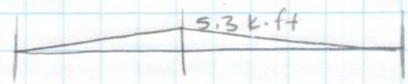
$= -1223$ psi compression $< 0.45f'_c$ ✓

Ultimate strength – determine factored moments

$M_1 = Pe$ (e = 0 at exterior support; e = 3" at interior support)

$M_1 = (692\text{ k})\left(\frac{3''}{12}\right) = 173\text{ k-ft}$ (primary post-tensioning moment)

$M_{sec} = M_{bal} - M_1$
$= 167.7 - 173$
$= -5.3\text{ k-ft}$ (secondary moment at interior support)



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

– midspan of 26' panel

$M_u = 1.2(105.1) + 1.6(29.2) + 1.0\left(\frac{-5.3}{2}\right) = 170\text{ k·ft}$

– interior support

$M_u = 1.2(-346.9) + 1.6(-96.5) + 1.0(-5.3) = -576\text{ k·ft}$

determine minimum bonded reinforcement (for 26' span)

– positive moment region
- end span = 26'

$f_t = 19\text{ psi tension} < 2\sqrt{f'_c} = 141\text{ psi}$ ✓

(no positive reinforcement required)

– negative moment region ($A_{s_{min}} = 0.00075A_{cf}$)
- interior support

$A_{cf} \geq (8")\left(\frac{26' + 34'}{2}\right) \times 12"/' = 2880\text{ in}^2$

$\geq (8")(27')(12"/') = 2592\text{ in}^2$

$A_{s_{min}} = 0.00075(2880) = 2.16\text{ in}^2$

(11) #4 top (2.20 in²)

• exterior support

$A_{cf} \geq (8'')(26'/2)(12''/') = 1248 \text{ in}^2$

$\geq (8'')(27')(12''/') = 2592 \text{ in}^2$

$A_{s\,min} = 0.00075\,(2592) = 1.94 \text{ in}^2$

(10) #4 top (2.0 in^2)

- must span a minimum of 1/6 the clear span on each side of the support
- at least 4 bars in each direction
- top bars within 1.5h (= 1.5×8" = 12") away from the face of the support on each side
- maximum bar spacing is 12"

check minimum reinforcement for ultimate strength

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

$- A_{ps} = (0.153 \text{ in}^2)(\#\text{tendons}) = (0.153)(26) = 3.98 \text{ in}^2$

$- f_{ps} = f_{se} + 10000 + \dfrac{(f'_c\, b d)}{300 A_{ps}}$ $\left(\text{for } \dfrac{L}{h} = \dfrac{26\times 12}{8} > 35\right)$

$= 174000 + 10000 + \dfrac{5000\,(27\times 12)\,d}{300\,(3.98)}$

$= 184000 + 1357d$

$- a = \dfrac{(A_s f_y + A_{ps} f_{ps})}{0.85 f'_c\, b}$

• at supports (interior)

$d = 8'' - 3/4'' - 1/4'' = 7''$

$f_{ps} = 184000 \text{ psi} + 1357(7) = 193{,}499 \text{ psi}$

$a = \dfrac{(2.20 \text{ in}^2)(60 \text{ ksi}) + (3.98 \text{ in}^2)(193.5 \text{ ksi})}{0.85(5)(27\times 12)} = 0.655''$

$\phi M_n = 0.9\left((2.20 \text{ in}^2)(60 \text{ ksi}) + (3.98 \text{ in}^2)(193.5 \text{ ksi})\right)\left(7'' - \dfrac{0.655''}{2}\right)/12$

$= 451.5 \text{ k-ft} < 576 \text{ k-ft} = M_u$

$\therefore$ need more reinforcement

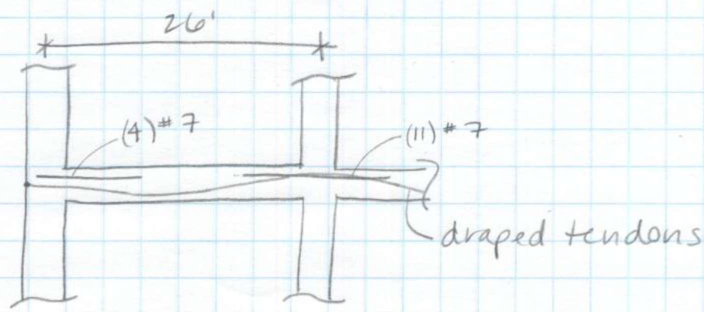
$A_{s,req} = 6.35 \text{ in}^2$

(11) - #7 bars top at interior support ($A = 6.6 \text{ in}^2$)

(4) - #7 bars top at exterior support ($A = 2.4 \text{ in}^2$)

final design

- slab thickness = 8" (normal weight concrete)
- 26 tendons (unbonded)
  - 1/2" Φ, 7 wire strands
- reinforcement
  - (11) #7 bars top at interior support
  - (4) #7 bars top at exterior support
  - no reinforcement needed at midspan



(unable to check deflection due to difficulty of calculation and inability to use appropriate program)

Appendix B – Two-Way Flat Plate Slab Design

The following design criteria and computer output summarize the two-way flat plate design. The computer program 'pcaSlab' was used to determine the reinforcement. A sketch of the final layout is shown in the report.

$\boxed{\text{SYSTEM \#2 – TWO-WAY, FLAT PLATE SLAB}}$

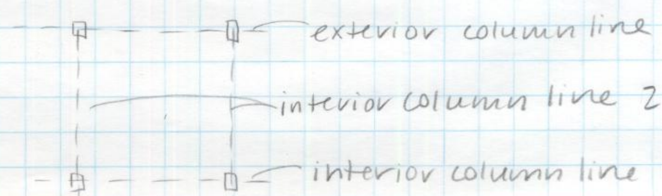
primary resources:
- pca Slab (computer program)
- ACI 318-08 (American Concrete Institute)

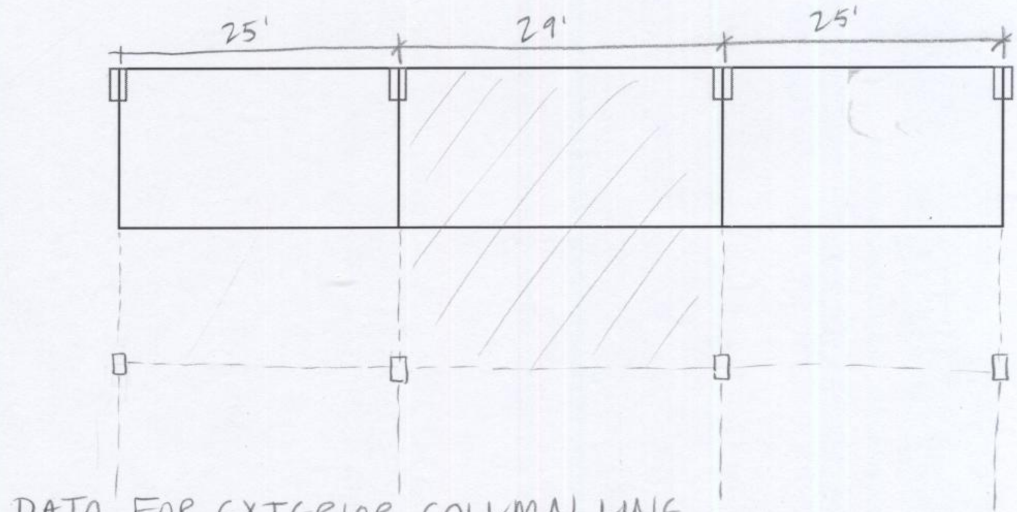
design criteria
- normal weight concrete for slabs and columns
- $f'_c = 5$ ksi (slabs)
  $f'_c = 6$ ksi (columns)
  $f_y = 60$ ksi
- live load = 40 psf (reducible)
  $L = 0.8(40)$
  $= 32$ psf
  } calcs. same as for system 1
- superimposed dead load = 15 psf

preliminary design
- design for 26'-0" x 29'-0" bay
- columns have same dimensions as original system (18" x 36")
- 11" slab thickness

(See sketch in report for summary of final design, including reinforcement)

Note: The following few pages contain selected computer output from pcaSlab. Three 'models' were made for 3 different column lines around the panel being designed





DATA FOR EXTERIOR COLUMN LINE

Span Data:

Slabs: L1, wL, wR (ft); t, Hmin (in)

Span	Loc	L1	t	wL	wR	Hmin
1	ExtL	25.000	11.00	1.500	13.000	9.40
2	ExtL	29.000	11.00	1.500	13.000	11.00
3	ExtL	25.000	11.00	1.500	13.000	9.40

Slab thickness is adequate

Support Data:

Columns: c1a, c2a, c1b, c2b (in); Ha, Hb (ft)

Supp	c1a	c2a	Ha	c1b	c2b	Hb	Red%
1	18.00	36.00	9.000	18.00	36.00	9.000	100 *
2	18.00	36.00	9.000	18.00	36.00	9.000	100
3	18.00	36.00	9.000	18.00	36.00	9.000	100
4	18.00	36.00	9.000	18.00	36.00	9.000	100 *

* Do not check punching shear around this column.

Top Reinforcement:

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	Column	Left	7.75	6.62	0.750	1.841	18.157	15.500	0.160	6-#5
		Middle	7.75	0.00	12.500	0.000	18.157	0.000	0.000	---
		Right	7.75	160.71	24.250	1.841	18.157	7.154	4.021	13-#5
	Middle	Left	6.75	-0.00	0.750	1.604	15.814	13.500	0.000	6-#5
		Middle	6.75	0.00	12.500	0.000	15.814	0.000	0.000	---
		Right	6.75	53.57	24.250	1.604	15.814	13.500	1.312	6-#5
2	Column	Left	7.75	157.46	0.750	1.841	18.157	7.154	3.937	13-#5
		Middle	8.00	0.00	14.500	0.000	18.743	0.000	0.000	---
		Right	7.75	157.46	28.250	1.841	18.157	7.154	3.937	13-#5
	Middle	Left	6.75	52.49	0.750	1.604	15.814	13.500	1.285	6-#5
		Middle	6.50	0.00	14.500	0.000	15.228	0.000	0.000	---
		Right	6.75	52.49	28.250	1.604	15.814	13.500	1.285	6-#5
3	Column	Left	7.75	160.71	0.750	1.841	18.157	7.154	4.021	13-#5
		Middle	7.75	0.00	12.500	0.000	18.157	0.000	0.000	---
		Right	7.75	6.62	24.250	1.841	18.157	15.500	0.160	6-#5
	Middle	Left	6.75	53.57	0.750	1.604	15.814	13.500	1.312	6-#5
		Middle	6.75	0.00	12.500	0.000	15.814	0.000	0.000	---
		Right	6.75	-0.00	24.250	1.604	15.814	13.500	0.000	6-#5

Top Bar Details:

Units: Length (ft)

		Left				Continuous		Right			
Span	Strip	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	6-#5	8.51	---		---		7-#5	8.51	6-#5	5.45
	Middle	6-#5	5.92	---		---		6-#5	8.22	---	

2	Column	7-#5	9.83	6-#5	6.25	---	7-#5	9.83	6-#5	6.25
	Middle	6-#5	8.79	---		---	6-#5	8.79	---	
3	Column	7-#5	8.51	6-#5	5.45	---	6-#5	8.51	---	
	Middle	6-#5	8.22	---		---	6-#5	5.92	---	

Bottom Reinforcement:

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	Column	7.75	81.60	10.000	1.841	18.157	13.286	2.007	7-#5
	Middle	6.75	54.40	10.000	1.604	15.814	13.500	1.333	6-#5
2	Column	8.00	68.66	14.500	1.901	18.743	13.714	1.683	7-#5
	Middle	6.50	45.77	14.500	1.544	15.228	15.600	1.119	5-#5
3	Column	7.75	81.60	15.000	1.841	18.157	13.286	2.007	7-#5
	Middle	6.75	54.40	15.000	1.604	15.814	13.500	1.333	6-#5

Bottom Bar Details:

Units: Start (ft), Length (ft)

		Long Bars			Short Bars		
Span	Strip	Bars	Start	Length	Bars	Start	Length
1	Column	7-#5	0.00	25.00	---		
	Middle	6-#5	0.00	25.00	---		
2	Column	7-#5	0.00	29.00	---		
	Middle	5-#5	0.00	29.00	---		
3	Column	7-#5	0.00	25.00	---		
	Middle	6-#5	0.00	25.00	---		

Punching Shear Around Columns:

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

Supp	Vu	vu	Munb	Comb	Pat	GammaV	vu	Phi*vc
1	--- Not checked ---							
2	98.79	101.6	-5.90	U2	All	0.352	103.9 ≤	212.1
3	98.79	101.6	5.90	U2	All	0.352	103.9 ≤	212.1
4	--- Not checked ---							

punching shear is okay

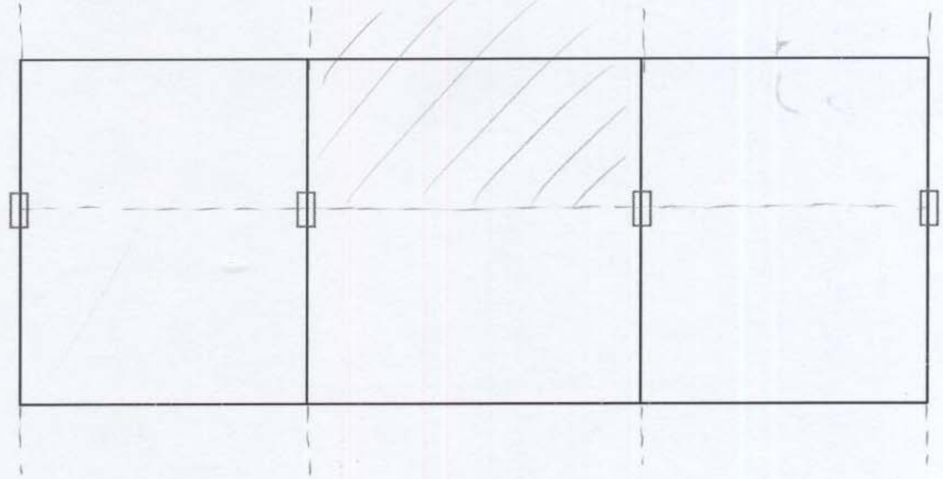
Maximum Deflections:

Units: Dz (in)

	Frame			Column Strip			Middle Strip		
Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.091	-0.027	-0.118	-0.125	-0.037	-0.162	-0.051	-0.015	-0.066
2	-0.077	-0.035	-0.112	-0.094	-0.043	-0.137	-0.056	-0.025	-0.081
3	-0.091	-0.027	-0.118	-0.125	-0.037	-0.162	-0.051	-0.015	-0.066

$$\Delta_{live\ max} = \ell/360 = \frac{29\times 12}{360} = 0.967'' > 0.043'' \checkmark \text{ OK}$$

$$\Delta_{total\ max} = \ell/240 = \frac{25\times 12}{240} = 1.25'' > 0.162'' \checkmark \text{ OK}$$



DATA FOR INTERIOR COLUMN LINE 1

Span Data:

Slabs: L1, wL, wR (ft); t, Hmin (in)

Span	Loc	L1	t	wL	wR	Hmin
1	Int	25.000	11.00	13.000	17.000	9.40
2	Int	29.000	11.00	13.000	17.000	10.00
3	Int	25.000	11.00	13.000	17.000	9.40

Slab thickness is adequate

Support Data:

Columns: c1a, c2a, c1b, c2b (in); Ha, Hb (ft)

Supp	c1a	c2a	Ha	c1b	c2b	Hb	Red%
1	18.00	36.00	9.000	18.00	36.00	9.000	100 *
2	18.00	36.00	9.000	18.00	36.00	9.000	100
3	18.00	36.00	9.000	18.00	36.00	9.000	100
4	18.00	36.00	9.000	18.00	36.00	9.000	100 *

* Do not check punching shear around this column.

Top Reinforcement:

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	Column	Left	12.50	4.29	0.750	2.970	29.285	15.000	0.104	10-#5
		Middle	12.50	0.00	12.500	0.000	29.285	0.000	0.000	---
		Right	12.50	328.75	24.250	2.970	29.285	5.556	8.305	27-#5
	Middle	Left	17.50	-0.00	0.750	4.158	40.999	15.000	0.000	14-#5
		Middle	17.50	0.00	12.500	0.000	40.999	0.000	0.000	---
		Right	17.50	109.59	24.250	4.158	40.999	15.000	2.677	14-#5
2	Column	Left	12.50	323.24	0.750	2.970	29.285	5.556	8.159	27-#5
		Middle	13.75	0.00	14.500	0.000	32.214	0.000	0.000	---
		Right	12.50	323.24	28.250	2.970	29.285	5.556	8.159	27-#5
	Middle	Left	17.50	107.75	0.750	4.158	40.999	15.000	2.631	14-#5
		Middle	16.25	0.00	14.500	0.000	38.071	0.000	0.000	---
		Right	17.50	107.75	28.250	4.158	40.999	15.000	2.631	14-#5
3	Column	Left	12.50	328.75	0.750	2.970	29.285	5.556	8.305	27-#5
		Middle	12.50	0.00	12.500	0.000	29.285	0.000	0.000	---
		Right	12.50	4.29	24.250	2.970	29.285	15.000	0.104	10-#5
	Middle	Left	17.50	109.59	0.750	4.158	40.999	15.000	2.677	14-#5
		Middle	17.50	0.00	12.500	0.000	40.999	0.000	0.000	---
		Right	17.50	-0.00	24.250	4.158	40.999	15.000	0.000	14-#5

Top Bar Details:

Units: Length (ft)

		Left				Continuous		Right			
Span	Strip	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	10-#5	8.51	---		---		14-#5	8.51	13-#5	5.45
	Middle	14-#5	5.92	---		---		14-#5	8.22	---	

2	Column	14-#5	9.83	13-#5	6.25	---	14-#5	9.83	13-#5	6.25
	Middle	14-#5	8.79	---		---	14-#5	8.79	---	
3	Column	14-#5	8.51	13-#5	5.45	---	10-#5	8.51	---	
	Middle	14-#5	8.22	---		---	14-#5	5.92	---	

Bottom Reinforcement:

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	Column	12.50	173.48	9.750	2.970	29.285	10.714	4.290	14-#5
	Middle	17.50	115.65	9.750	4.158	40.999	15.000	2.827	14-#5
2	Column	13.75	145.03	14.500	3.267	32.214	13.750	3.567	12-#5
	Middle	16.25	96.69	14.500	3.861	38.071	15.000	2.361	13-#5
3	Column	12.50	173.48	15.250	2.970	29.285	10.714	4.290	14-#5
	Middle	17.50	115.65	15.250	4.158	40.999	15.000	2.827	14-#5

Bottom Bar Details:

Units: Start (ft), Length (ft)

		Long Bars			Short Bars		
Span	Strip	Bars	Start	Length	Bars	Start	Length
1	Column	14-#5	0.00	25.00	---		
	Middle	14-#5	0.00	25.00	---		
2	Column	12-#5	0.00	29.00	---		
	Middle	13-#5	0.00	29.00	---		
3	Column	14-#5	0.00	25.00	---		
	Middle	14-#5	0.00	25.00	---		

Punching Shear Around Columns:

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

Supp	Vu	vu	Munb	Comb	Pat	GammaV	vu		Phi*vc
1	--- Not checked ---								
2	206.28	159.2	-10.73	U2	All	0.341	162.4	<	212.1
3	206.28	159.2	10.73	U2	All	0.341	162.4	<	212.1
4	--- Not checked ---								

punching shear is okay

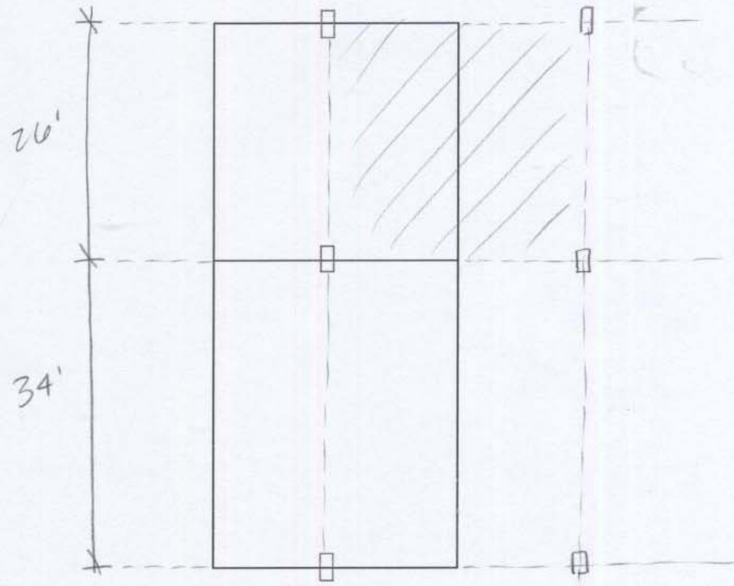
Maximum Deflections:

Units: Dz (in)

	Frame			Column Strip			Middle Strip		
Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.094	-0.028	-0.122	-0.166	-0.050	-0.216	-0.042	-0.013	-0.055
2	-0.078	-0.035	-0.114	-0.115	-0.052	-0.167	-0.047	-0.021	-0.068
3	-0.094	-0.028	-0.122	-0.166	-0.050	-0.216	-0.042	-0.013	-0.055

$\Delta_{live\ max} = 0.967'' > 0.052''$ ✓ OK

$\Delta_{total\ max} = 1.25'' > 0.216''$ ✓ OK



DATA FOR INTERIOR COLUMN LINE 2

Span Data:

Slabs: L1, wL, wR (ft); t, Hmin (in)

Span	Loc	L1	t	wL	wR	Hmin	
1	Int	26.000	11.00	14.500	12.500	9.20	
2	Int	34.000	11.00	14.500	12.500	12.40	*b

NOTES:
*b- Slab thickness is less than minimum.

Support Data:

Columns: c1a, c2a, c1b, c2b (in); Ha, Hb (ft)

Supp	c1a	c2a	Ha	c1b	c2b	Hb	Red%	
1	36.00	18.00	9.000	36.00	18.00	9.000	100	*
2	36.00	18.00	9.000	36.00	18.00	9.000	100	
3	36.00	18.00	9.000	36.00	18.00	9.000	100	*

* Do not check punching shear around this column.

slab thickness adequate for the panel being designed (outside of this panel, the column grid isn't regular - shear walls interrupt the typical layout)

Top Reinforcement:

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	Column	Left	12.75	35.71	1.500	3.029	29.871	15.300	0.868	10-#5
		Middle	12.75	0.00	13.000	0.000	29.871	0.000	0.000	---
		Right	12.75	309.85	24.500	3.029	29.871	4.500	7.800	34-#5
	Middle	Left	14.25	-0.00	1.500	3.386	33.385	15.545	0.000	11-#5
		Middle	14.25	0.00	13.000	0.000	33.385	0.000	0.000	---
		Right	14.25	103.29	24.500	3.386	33.385	15.545	2.527	11-#5
2	Column	Left	12.75	407.39	1.500	3.029	29.871	4.500	10.397	34-#5
		Middle	13.50	0.00	17.000	0.000	31.628	0.000	0.000	---
		Right	13.50	167.23	32.500	3.208	31.628	11.571	4.126	14-#5
	Middle	Left	14.25	135.80	1.500	3.386	33.385	15.545	3.335	11-#5
		Middle	13.50	0.00	17.000	0.000	31.628	0.000	0.000	---
		Right	13.50	-0.00	32.500	3.208	31.628	14.727	0.000	11-#5

Top Bar Details:

Units: Length (ft)

		Left				Continuous		Right			
Span	Strip	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	10-#5	9.09	---		---		17-#5	9.44	17-#5	6.10
	Middle	11-#5	6.56	---		---		11-#5	9.44	---	
2	Column	17-#5	11.73	17-#5	7.70	---		11-#5	11.73	3-#5	7.70
	Middle	11-#5	9.64	---		---		11-#5	8.32	---	

Bottom Reinforcement:

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	Column	12.75	130.47	10.500	3.029	29.871	13.909	3.207	11-#5
	Middle	14.25	86.98	10.500	3.386	33.385	15.545	2.124	11-#5
2	Column	13.50	249.60	18.860	3.208	31.628	7.714	6.221	21-#5
	Middle	13.50	166.40	18.860	3.208	31.628	11.571	4.105	14-#5

Bottom Bar Details:

Units: Start (ft), Length (ft)

		Long Bars			Short Bars		
Span	Strip	Bars	Start	Length	Bars	Start	Length
1	Column	11-#5	0.00	26.00	---		
	Middle	11-#5	0.00	26.00	---		
2	Column	21-#5	0.00	34.00	---		
	Middle	11-#5	0.00	34.00	3-#5	5.10	23.80

Punching Shear Around Columns:

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

Supp	Vu	vu	Munb	Comb	Pat	GammaV	vu	Phi*vc	
1	--- Not checked ---								
2	216.44	167.0	161.31	U2	All	0.463	218.9	212.1	*EXCEEDED
3	--- Not checked ---								

Maximum Deflections:

Units: Dz (in)

	Frame			Column Strip			Middle Strip		
Span	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)	Dz(DEAD)	Dz(LIVE)	Dz(TOTAL)
1	-0.076	-0.019	-0.095	-0.118	-0.030	-0.148	-0.038	-0.010	-0.047
2	-0.301	-0.116	-0.417	-0.444	-0.171	-0.615	-0.158	-0.061	-0.219

$$\Delta_{live\ max} = \frac{34 \times 12}{360} = 1.13'' > 0.171'' \quad \checkmark \text{ OK}$$

$$\Delta_{total\ max} = \frac{34 \times 12}{240} = 1.7'' > 0.615'' \quad \checkmark \text{ OK}$$

The punching shear is slightly too high for the middle support, which can be ignored because the model is simplified. Not all of the load along the 34' span goes to the corner columns because of concrete walls along the column lines not shown.

Appendix C – Steel/Concrete Composite Floor System Design

The following information and computer output were used to determine an appropriate floor system based on a 26'-0" x 29'-0" bay. In the model (RAM Structural System), the surrounding panels were simplified and added to the structure to give a more complete design.

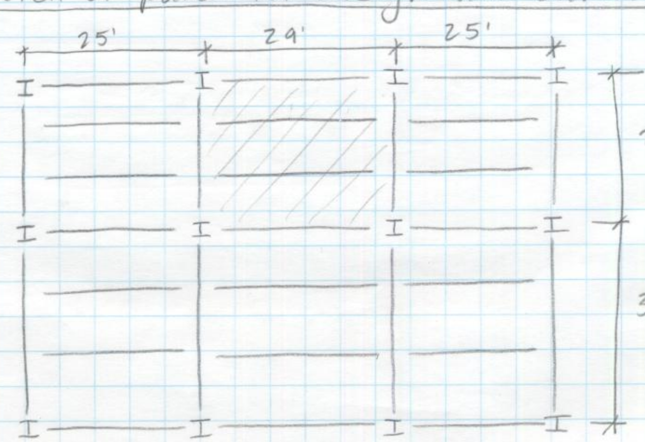
SYSTEM #3 – STEEL/CONCRETE COMPOSITE FLOOR SYSTEM

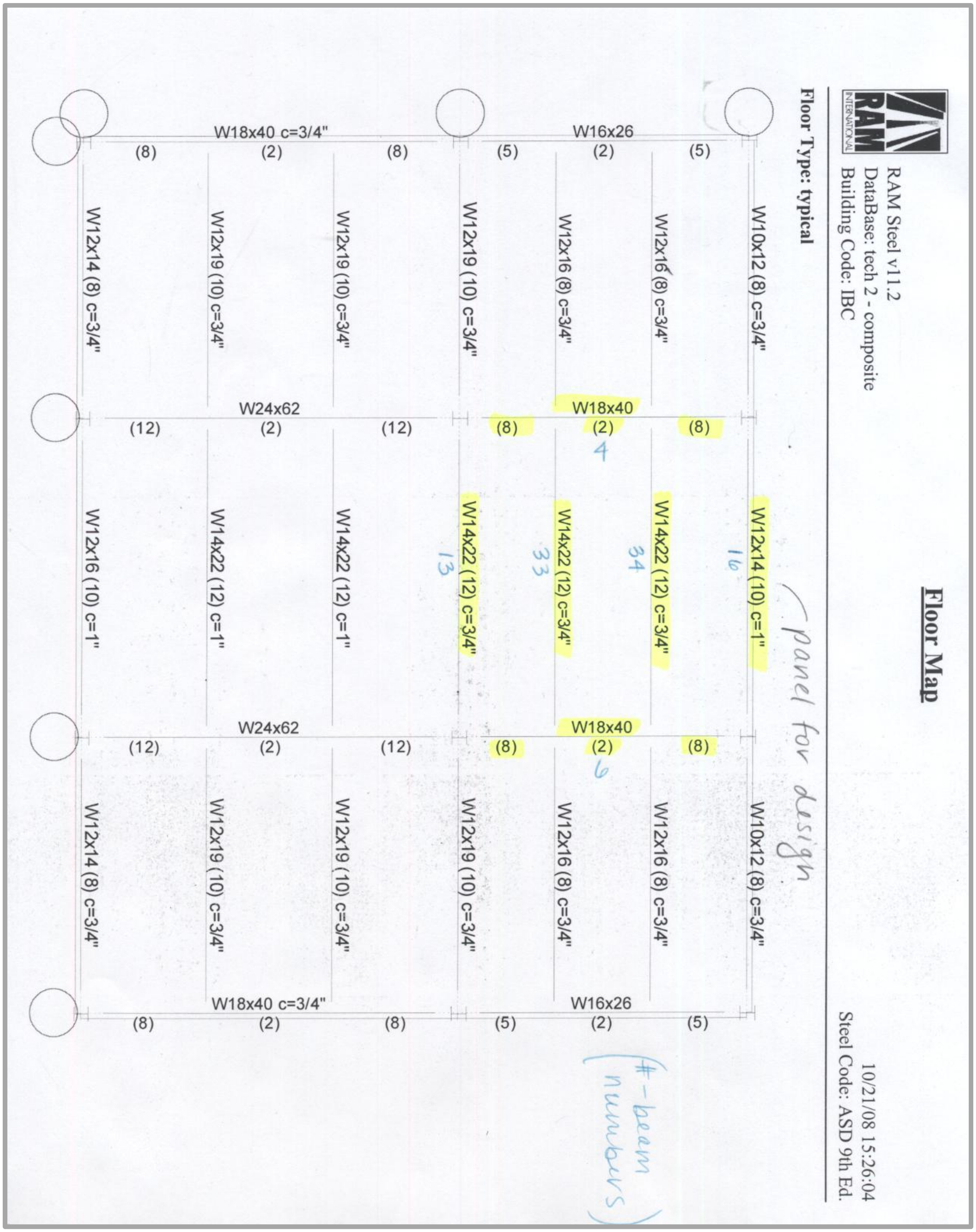
primary resources:
- RAM Structural System (computer program)
- United Steel Deck 'design manual and catalog of products'
- Steel Construction Manual, 13^{th} edition – American Institute of Steel Construction

design criteria
- live load = 40 psf (reduced in RAM)
- superimposed dead load = 15 psf
- lightweight concrete – W = 115 pcf
  f'_c = 3000 psi

preliminary design choices (becomes final design)
- 3" LOK-FLOOR (f_y = 33 ksi)
- 6" slab depth (to bottom of decking)
- 20-gage deck
- max span of deck = 34'/3 = 11'-4"
- max uniform live service load = 190 psf (USD)
- live load deflection limited by table to $l/340$
- W = 43 psf (composite – deck + concrete)
- max unshored span = 11.65' (for 3 span ✓)

sketch of panel for design and surrounding panels




RAM Steel v11.2
DataBase: tech 2 - composite
Building Code: IBC
Floor Type: typical
Floor Map
10/21/08 15:26:04
Steel Code: ASD 9th Ed.
W18x40 c=3/4"
W16x26
W24x62
W18x40
W10x12 (8) c=3/4"
W12x16 (8) c=3/4"
W12x19 (10) c=3/4"
W12x14 (8) c=3/4"
W12x14 (10) c=1"
W14x22 (12) c=3/4"
W14x22 (12) c=1"
W12x16 (10) c=1"
(8)
(2)
(5)
(12)
panel for design
(#-beam numbers)
4
16
34
33
13
6

Beam Deflection Summary

RAM Steel v11.2
DataBase: tech 2 - composite
Building Code: IBC

10/21/08 15:26:04
Steel Code: ASD 9th Ed.

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: typical

Composite / Unshored

Bm #	Beam Size	Initial in	PostLive in	PostTotal in	NetTotal in	Camber in
1	W18X40	0.964	0.334	0.480	0.694	3/4
9	W12X14	0.955	0.283	0.389	0.595	3/4
39	W12X19	1.180	0.318	0.453	0.883	3/4
40	W12X19	1.180	0.318	0.453	0.883	3/4
2	W16X26	0.656	0.231	0.322	0.978	
12	W12X19	1.047	0.293	0.413	0.709	3/4
31	W12X16	1.144	0.325	0.450	0.844	3/4
32	W12X16	1.144	0.325	0.450	0.844	3/4
15	W10X12	1.239	0.304	0.418	0.906	3/4
3	W24X62	0.772	0.222	0.344	1.117	
10	W12X16	1.498	0.431	0.593	1.091	1
41	W14X22	1.405	0.387	0.560	0.965	1
42	W14X22	1.405	0.387	0.560	0.965	1
4	W18X40	0.666	0.191	0.288	0.954	
13	W14X22	1.247	0.357	0.510	1.006	3/4
33	W14X22	1.089	0.325	0.458	0.797	3/4
34	W14X22	1.089	0.325	0.458	0.797	3/4
16	W12X14	1.375	0.363	0.499	0.874	1
5	W24X62	0.772	0.222	0.344	1.117	
11	W12X14	0.955	0.283	0.389	0.595	3/4
37	W12X19	1.180	0.318	0.453	0.883	3/4
38	W12X19	1.180	0.318	0.453	0.883	3/4
6	W18X40	0.666	0.191	0.288	0.954	
14	W12X19	1.047	0.293	0.413	0.709	3/4
35	W12X16	1.144	0.325	0.450	0.844	3/4
36	W12X16	1.144	0.325	0.450	0.844	3/4
17	W10X12	1.239	0.304	0.418	0.906	3/4
7	W18X40	0.964	0.334	0.480	0.694	3/4
8	W16X26	0.656	0.231	0.322	0.978	

critical values

$$\Delta_{live\,max} = l/360 = \frac{29\times12}{360} = 0.97'' > 0.363'' \quad \checkmark \text{ OK}$$

$$\Delta_{total\,max} = l/240 = \frac{29\times12}{240} = 1.45'' > 1.006'' \quad \checkmark \text{ OK}$$

check for fire resistance (USD)

- composite decking
  - lightweight concrete
  - no fireproofing on the deck
  - 1 hr required fire rating
  - UL designation number D916 works for $2\frac{5}{8}$" concrete cover (3" concrete cover is acceptable)
- beams → spray on fireproofing

Appendix D – Precast Hollow Core Slab Design

The following information was used to determine an appropriate design for a precast hollow core slab. The slab dimensions and characteristics were determined with the superimposed load and slab span.

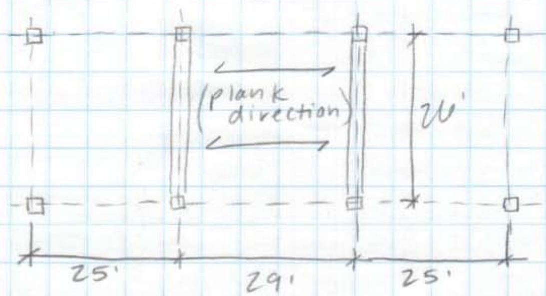
SYSTEM #4 – PRECAST HOLLOW CORE SLAB

primary resources:

- "Precast/Prestressed Concrete Products and Building Systems"
  – publication by Nitterhouse Concrete Products
- PCI Design Handbook, 6th edition
  – Precast/Prestressed Concrete Institute)
- AISC Steel Manual, 13th edition

design criteria

- live load = 40 psf (residential)
- superimposed dead load = 15 psf
- normal weight concrete
- span = 29'-0"



(beams located along column lines, at either end of plank span)

- 1-hr fire rating for floor assembly (IBC 2006 section 711.3)
- 3-hr fire rating for fire walls (table 705.4)

plank design (see next page for design data)

$\boxed{8" \times 4'\text{-}0" \text{ untopped planks}}$ – 1hr fire resistance rating

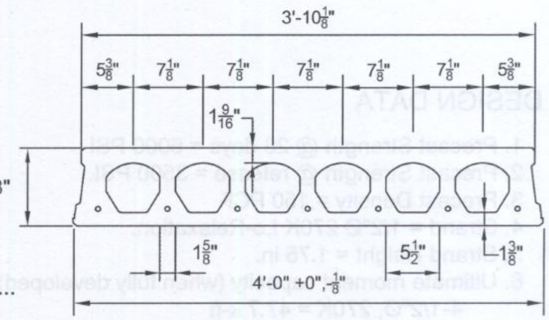
$\boxed{4\text{-}\tfrac{1}{2}" \phi \text{ strands}}$ – max superimposed service load = 62 psf

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

1 Hour Fire Resistance Rating (Untopped)

PHYSICAL PROPERTIES
Precast

A = 235 in.^{2}	S_b = 459 in.^{3}
I = 1838 in.^{4}	S_t = 459 in.^{3}
Y_b = 4.00 in.	Wt.= 245 PLF
Y_t = 4.00 in.	Wt.= 61.25 PSF
e = 2.25 in.	



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 = 72.8 k-ft
   7-1/2"Ø, 270K = 119.8 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. Load values to the left of the solid line are controlled by ultimate shear strength.
12. Load values to the right are controlled by ultimate flexural strength or allowable service stresses.
13. Load values will be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
14. 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) 17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	222	194	176	159	144	132	120	110	99	88	78	70	62	55					
7 - 1/2"ø	LOAD (PSF)	288	269	252	236	222	210	196	179	165	152	144	131	119	107	97	87	78	70	63

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 1 Hour & 0 Minute fire resistance rating.

08/10/07

8SF1.0

beam design

- reduced live load = 32 psf
- superimposed dead load = 15 psf
- self-weight of plank = 61.25 psf

108.3 psf (total service load)

$$w_u = 1.2\, w_{DL} + 1.6\, w_{LL}$$
$$= 1.2(15+61.25) + 1.6(32)$$
$$= 91.5 + 51.2$$
$$= 142.7 \text{ psf}$$

- beam tributary width = $\frac{29' + 25'}{2} = 27'$

beam span = 26'

$$M_u = \frac{w_u (TW) \ell^2}{8} = \frac{(0.1427 \text{ ksf})(27')(26')^2}{8} = 326 \text{ k-ft}$$

– table 3-2 of Steel Manual
W21×44 would work if the unbraced length was ≤ 13.0' (assume not braced along entire span)

– table 3-10 of Steel Manual

$\boxed{W14 \times 74}$ – most economical section with adequate moment capacity ($I = 795 \text{ in}^4$)

- deflection check

$$\Delta_{live} = \frac{5(0.032 \text{ ksf} \times 27')(26')^4 (12''/')^3}{384(29000 \text{ ksi})(795 \text{ in}^4)} = 0.39''$$

$$\Delta_{live\,max} = \ell/360 = \frac{26 \times 12}{360} = 0.87'' > 0.39'' \quad \checkmark \text{ OK}$$

$$\Delta_{total} = \frac{5(0.1083 \text{ ksf} \times 27')(26')^4 (12''/')^3}{384(29000 \text{ ksi})(795 \text{ in}^4)} = 1.30''$$

$$\Delta_{total\,max} = \ell/240 = 26 \times 12/240 = 1.30'' \geq 1.30'' \quad \checkmark \text{ OK}$$

Appendix E – Cost Analysis

The following cost analysis was mostly based on RS Means "Assemblies Cost Data 2009." They are rough estimates based on each floor system, so the actual costs would likely vary. However, for the purpose of this comparison, the costs will just be viewed relative to the other systems.

System	Material Cost (per SF)	Total Cost (per SF)
Post-Tensioned Slab • based on 'Cost Works,' an online resource by RS Means • estimate is for large job (versus small job)	$12.10	$20.94
Two-Way Flat Plate • 25'x25' bay (slight underestimate) • superimposed load = 75 psf, total load = 194 psf	$7.60	$15.90
Steel/Concrete Composite System • composite steel beams with welded shear studs • composite steel deck • light weight concrete • sprayed fiber fireproofing for beams • 25'x30' bay (approximate) • superimposed load = 75 psf, total load = 119 psf	$15.10	$20.35
Precast Hollow Core Slab • normal weight concrete, no topping • 30' span (approximate) • superimposed load = 75 psf, total load = 130 psf • sum of cost of planks and cost of W14x74 beam spread into an area load	$11.14	$13.49