# LETTER OF TRANSMITTAL

**DATE**: October 18, 2013

**TO**: Dr. Linda Hanagan

**FROM**: Alyssa Stangl

**ENCLOSED:** AE 481W – Senior Thesis | Structural Technical Report 3

Dear Dr. Hanagan,

This report was prepared to be submitted for Technical Report 3 for AE 481W – Senior Thesis. The building in question is La Jolla Commons Phase II Office Tower. Items included in this report are as follows:

- Gravity Load Calculations
- Corrected Gravity Loads
- Gravity Spot Checks of the Existing System – Flat Plate Slab
- Alternative Floor Design #1 – Non-Composite Steel
- Alternative Floor Design #2 – Composite Steel
- Alternative Floor Design #3 – One-Way Concrete Slab

System comparisons related to cost, weight, architectural impact, and such have also been included in this report. As well as, possible lateral systems for each floor system discussed.

Thank you for your time reviewing this report. I look forward to discussing it with you in the near future.

Sincerely,

Alyssa Michelle Stangl

# Technical Report 3

October 18, 2013

# La Jolla Commons Phase II Office Tower

**San Diego, California**

Alyssa Stangl | Structural Option | Advisor: Dr. Linda Hanagan

## Table of Contents

## Executive Summary

La Jolla Commons Phase II Office Tower is a 13 story office building in San Diego, California. Each floor is about 40,320 square feet, and the structure reaches 198 feet from ground level to the top of the penthouse. With two levels of underground parking, the building extends about 20 feet below grade. Acting as an office building for LPL Financial, the building has open floor plans and large areas of glass curtain wall. La Jolla Commons Tower II received a LEED-CS Gold Certification and is the nation's largest and most advanced net-zero office building.

The building's gravity system begins with a mat foundation, two stories below grade. The mat foundation was chosen for its constructability, when compared to a system of footers and grade beams. The super structure consists of two way, flat plate, concrete slabs on a rectangular column grid. A typical bay is 30 feet by 40 feet. Each level varies in thickness – 18, 14, or 12 inches, reinforcing was used as required by code. Camber was used for the slab at each level (except Lower Level 2 where the mat foundation serves as the floor). This was done because large construction loads crack the slab, causing considerable deflections after construction and finishes are completed. Camber ranges from ¼ inch at the exterior edge of a bay to 2 ¼ inches at the center of the bay, creating an essentially flat slab after building loads have been applied.

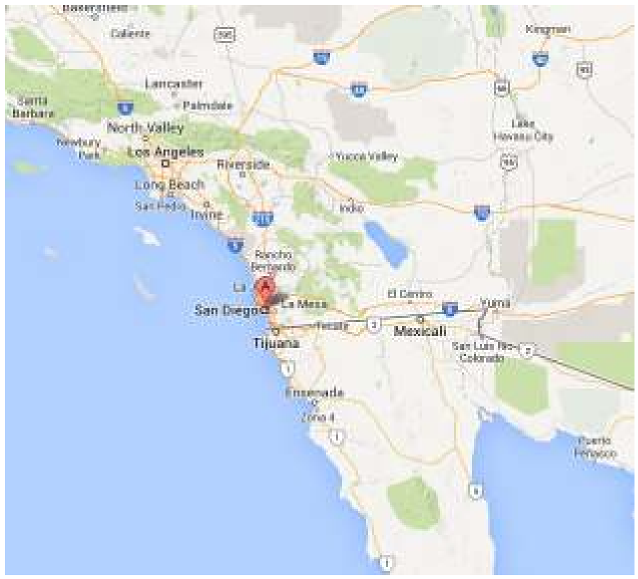
Laid out at the core of the building, the lateral system of La Jolla Commons Tower II consists of reinforced concrete shear walls. Due to the high shear forces associated with earthquake loading in this Seismic Category D structure, the diaphragm is not relied upon to transfer lateral loads to the shear wall system; therefore, collector beams are used to aid in load transfer.

La Jolla Commons Tower II has two unique structural and architectural features. The north and south sides of the building feature 15 foot cantilevers that start at Level 3 and continue up to the roof level. The structure is similar to that of the rest of slab; however, it does have additional reinforcement and a thickened slab edge, creating a back-span for the cantilever. Also, the building has a plaza area on the Ground Level which essentially carves out a portion of the Ground Level and Level 2. Main building columns are exposed here, and additional 18 inch columns are added to support the slab edge above.

La Jolla Commons Tower II was designed using the 2010 California Building Code which corresponds to ASCE 7-05 and ACI 318-08. CBC 2010 and ASCE 7-05 were used to calculate live, wind, and earthquake loads. ACI 318 – 08, Chapter 21, references the design of concrete Earthquake-Resistant structures, and ASCE 7-05, Chapter 12, details the Seismic Design Requirements for Building Structures. Both of these documents were used heavily in the design of LJC II in order to account for seismic loading and detailing.

La Jolla Commons Phase II Office Tower is full of educational value. It has several structural challenges and unique conditions– punching shear, seismic loading and detailing, concrete shear wall design, and computer modeling.

## Building Site Information




**San Diego California** *(Google Maps)*


**Building Site Plan** *(Courtesy of Hines)*

# La Jolla Commons Phase II Office Tower

**San Diego , California | LPL Financial Office Tower**

## Primary Project Team

Owner | Hines
Tenant | LPL Financial
Architect | AECOM
Structural Engineer | Nabih Youssef Associates
MEP Engineer | WSP Flack + Kurtz
Civil Engineer | Leppert Engineering

## General Building Data

Construction Dates | April 2012 – May 2014
Building Cost | $78,000,000
Delivery Method | Design-Bid-Build
Height| 198’ – 8” | 13 Stories
2 Levels | Underground Parking
Size | 462,301 GSF

## Architecture

- Modern style building with glass curtain wall
- 12 foot floor-to-floor height
- Very open and spacious office area
- Interior features and build out by tenant

## Sustainability Features

- First Class A, NetZero Office Building in the USA
- Building returns more energy to the grid than it uses on an annual basis
- LEED – CS Gold Certification

## Structural

- Two-way, flat plate , reinforced concrete slab
- Concrete columns on a regular column grid
- Special reinforced concrete shear walls
- Mat foundation system

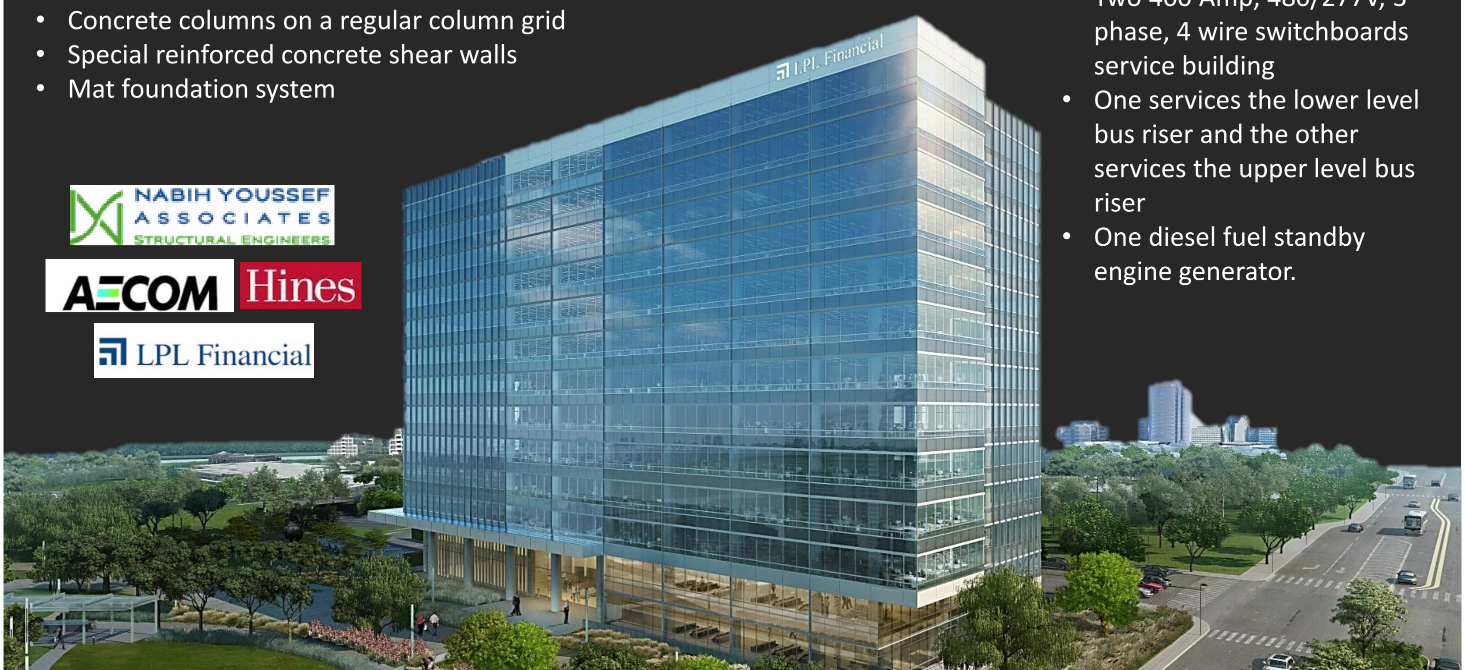
## Mechanical

- Chilled Water, floor-by – floor VAV Dual Path Air Handling Units
- Ventilation and cooling through underfloor air distribution, overhead air to perimeter zones.

## Lighting and Electrical

- High efficiency, low glare lighting fixtures
- High power factor electronic ballasts
- Lighting control system integrated with Building Management System, local override at each floor

- Two 400 Amp, 480/277V, 3-phase, 4 wire switchboards service building
- One services the lower level bus riser and the other services the upper level bus riser
- One diesel fuel standby engine generator.




**Alyssa Stangl [ Structural Option ]**

**http://www.engr.psu.edu/ae/thesis/portfolios/2014/ams6158**

## Documents Used to Create This Report

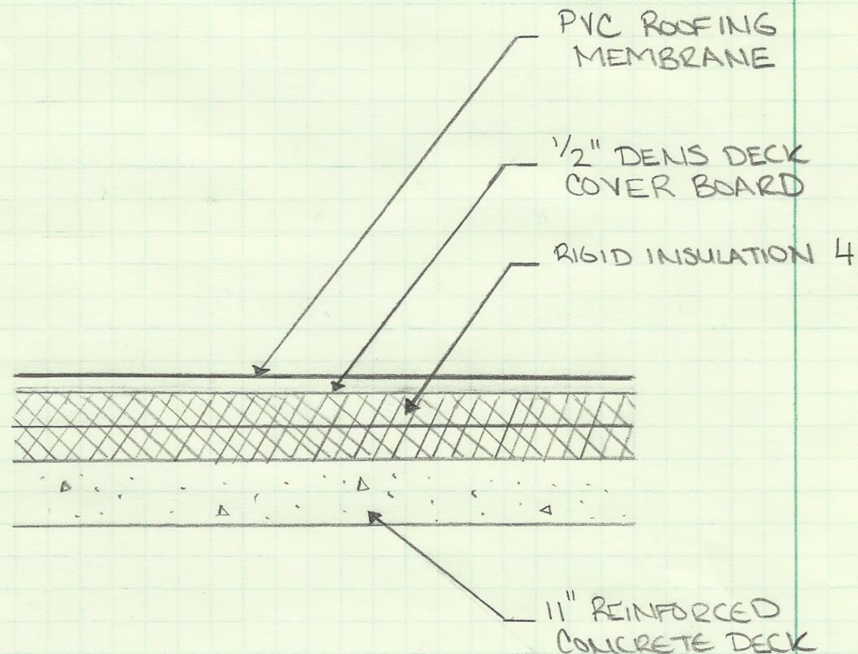
- *American Concrete Institute*
  - ACI 318 – 11
- *American Institute of Steel Construction*
  - AISC – Steel Construction Manual, 14$^{th}$ Edition
- *Concrete Reinforcing Steel Institute*
  - CRSI Handbook 2008
- *Reinforced Concrete Mechanics and Design, 6$^{th}$ Edition*
  - By James K. Wight and James G. MacGregor
- *International Building Code*
  - IBC 2012
- *Reed Construction Data | RS Means*
  - Square Foot Cost 2013
- *American Society of Civil Engineers*
  - ASCE 7 – Minimum Design Loads for Buildings
- *La Jolla Commons Phase II Office Tower*
  - Construction Documents
  - Technical Specifications

# GRAVITY LOADS

# CALCULATIONS

## Typical Roof Bay Loading

Cross Section of Roof Construction (A470 - B2)




ROOF DEAD LOAD:

Adhered PVC Membrane = 2 PSF

1/2" Dens Deck Cover Board = 2 PSF

4" Rigid Insulation = 6 PSF

Concrete Slab, 11"

$= (150\text{ PCF})(11''/12'') = 137.5\text{ PSF}$

Superimposed/Misc

Ceilings = 5 PSF
MEP = 15 PSF
Sprinklers = 3 psf
= 23 PSF

+

$\boxed{\text{ROOF DEAD LOAD} = 171\text{ PSF}}$

ROOF LIVE LOAD:

ASCE 7-05 : Ch.4 Table 4-1

$L_r = 20$ PSF

Construction Documents - S001

$\boxed{L_r = 20 \text{ PSF}}$

* Roof live load used for design is equal to the code minimum value

SNOW LOAD:

ASCE 7-05: Ch.7

Below 1500 ft elevation → 0 PSF
Elevation from 2000-1500 ft → 5 PSF

Site elevation is about 330ft (C103)

$P_g = 0$ PSF

$P_f = 0.7 C_e C_t I P_g$

$\boxed{P_f = 0 \text{ PSF}}$

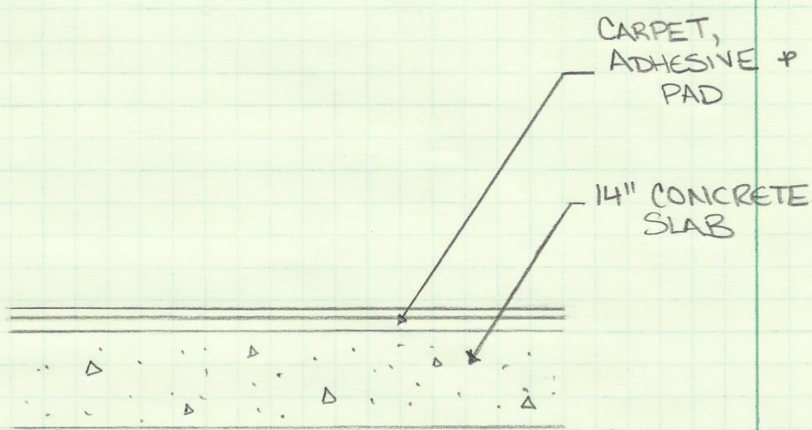
From Figure 7-9 ASCE 7-05:

$h_d = 0$ ft for $P_g = 0$ PSF

$\boxed{\text{Drift Calculation will yield no drift load because } P_g = 0 \text{ PSF}}$

## Typical Floor Bay Loading

Cross Section of floor construction




FLOOR DEAD LOAD:

14" Concrete slab

$= (150\ \text{PCF})(14''/12'') = 175\ \text{PSF}$

Carpet + Adhesive + Pad = 1.5 PSF

Superimposed / Misc

Ceilings = 5 PSF
MEP = 15 PSF
FULLY SPRINKLED = 3 PSF

$\left.\begin{array}{l}\text{Ceilings} = 5\ \text{PSF}\\ \text{MEP} = 15\ \text{PSF}\\ \text{FULLY SPRINKLED} = 3\ \text{PSF}\end{array}\right\} 23\ \text{PSF}$

Raised access floor (by tenant) = 15 PSF (allowance)

\+ ______

214.5

$\boxed{\text{Typical floor bay dead load} = 215\ \text{PSF}}$

Typical Floor Bay Live load:

ASCE 7-05 Chapter 4

Office live load = 50 PSF

Interior partitions = + 20 PSF

70 PSF

Offices, Corridors above 1st floor = 80 PSF

→ Apply 80 psf to entire office area to allow for future layout flexibility.

$\boxed{\text{Typical Bay Floor Live load = 80 PSF}}$

→ This matches the design value from sheet S001 for office spaces = 80 PSF

## Non-Typical Dead Loads:

### Floors and Roofs:

- Ground to Level 13, 18" slab edges
  $= (150 \text{ PCF})(18''/12'') = \underline{225 \text{ PSF}}$

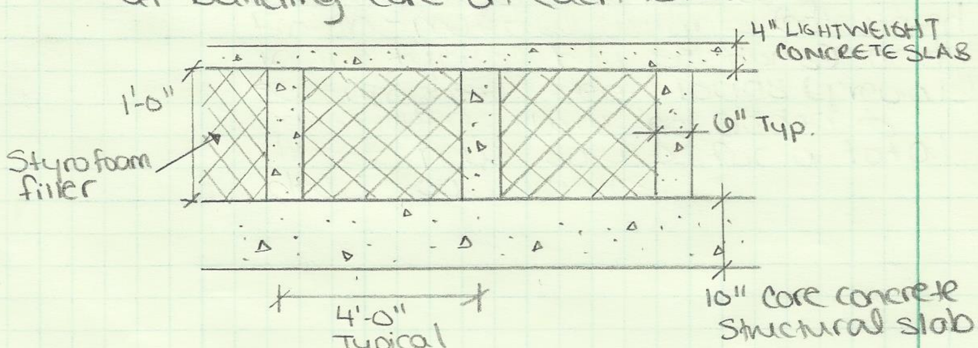
- Ground to Level 13, 10" core slab
  $= (150 \text{ PCF})(10''/12'') = \underline{125 \text{ PSF}}$

- Roof / Penthouse floor, 11" slab
  $= (150 \text{ PCF})(11''/12'') = \underline{137.5 \text{ PSF}}$

- Roof of Penthouse, 8" slab
  $= (150 \text{ PCF})(8''/12'') = \underline{100 \text{ PSF}}$

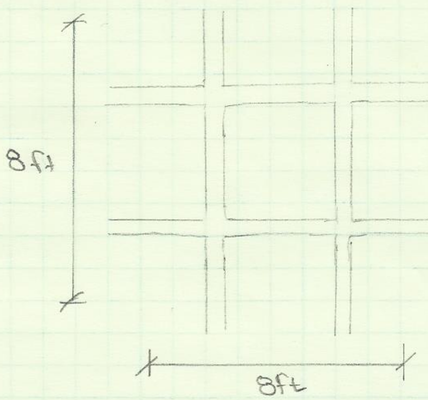
- 1/4" x 2" - Roof metal bar grating = 15 PSF
  * From Grating Pacific Catalogue dercvision was done.

- Built-up slab at several locations at building core on each level




see next page for calculation

Built up slab continued:




• Lets consider an 8ft x 8ft segment

LGTWT Concrete

$= (4/12)(115 \text{ PCF}) = \underline{38.3 \text{ PSF}}$

Structural Slab

$= (10''/12'')(150 \text{ PCF}) = \underline{125 \text{ PSF}}$

Pedistals

$(12''/12'')(6''/12)(32')(115 \text{ PCF}) = 1840 \text{ lb}$

$1840 \text{ lb} / 64 \text{ ft}^2 = \underline{28.7 \text{ PSF}}$

TOTAL at built up slab = $\underline{\underline{192 \text{ PSF}}}$

Special Note For NON-TYP. DEAD LOADS:

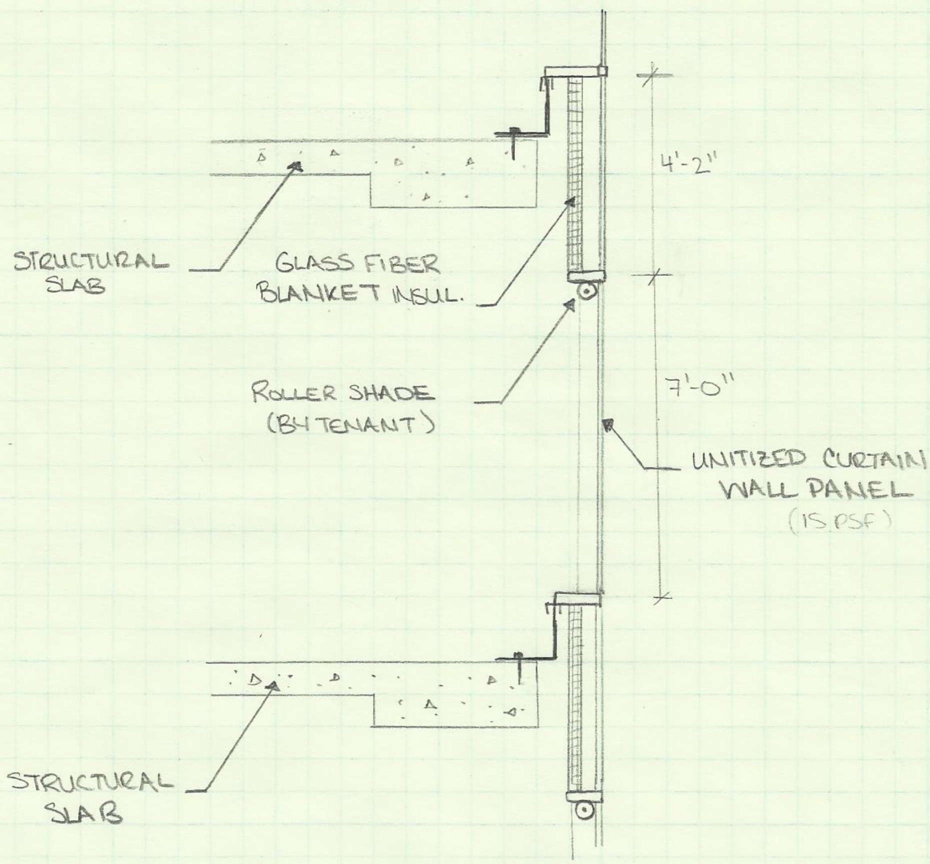
The values provided are modifications to structural components only. The finishes load and superimposed loads (calculated for a typical bay) needs to be added to these values for a total dead load value.

## Non-Typical Live Loads:

| Use | Location in Building | Design Value | ASCE 7-5 Value | Explaination (if necessary) |
|---|---|---|---|---|
| Lobby + Corridor | Ground level at core | 100 PSF | 100 PSF (first floor corridor) | |
| Lobby + Corridor | Above ground level around core | 100 PSF | 80 PSF | 20 PSF was added for partitions in design to allow flexibility in layout for tenant |
| Core/ Egress | Above ground level at core | 250 PSF | N/A | Value based on egress requirements for possible future multi-tenant conditions |
| Exit Stairs | At building core | 100 PSF | 100 PSF | |
| Cafeteria | In lease space * | 100 PSF | 100 PSF (Dining and resturants) | |
| Fitness Center | In lease space * | 100 PSF | 100 PSF (gymnasiums) | |
| Conference Center | In lease space * | 100 PSF | 100 PSF | ASCE 7-10 has an office for heavier anticipated occupancy |
| Data Center | In lease space * | 250 PSF | 100 PSF (computer rooms) | PSF load determined from known equipment weights |
| Mech. Areas | Mechanical Rooms at Core on each level | 200 PSF | N/A | Value based on industry standard and actual equipment loads if known. |

## Typical Exterior Wall Load:

### Typical Curtain Wall Section




### Wall Load Path - Gravity

The curtain wall is essentially hung by the top mullion of each unit from the slab edge. The unit then ties into the unit at the level below. The wall load goes into the slab which transfers the load into the edge columns. The columns will transfer this load down to the mat foundation, which will spread out the load to meet the bearing capacity of the soil.

Typical Curtain Wall Dead Load:

Line load at slab edge

Fiber blanket Insulation

$1.0 \text{ PCF}, 1'' \rightarrow 1 \text{ PCF}\left(\frac{1''}{12''}\right)\left(4' + \frac{2}{12}\right) = 0.35 \text{ PLF}$

Roller Shade

Allowance for tenant selection = 5 PLF

Curtain Wall Units

$10 \text{ PSF}\left(4' + \frac{2''}{12''} + 7'\right) = 112 \text{ PLF}$

+ ______

117.35 PLF

$\boxed{\text{Curtain Wall Assembly Dead Load} = 118 \text{ PLF}}$

# CORRECTED GRAVITY LOADS

# CALCULATIONS

Adjusted Roof Dead load:

Superimposed / Misc

Ceilings = 5 PSF
MEP = 3 PSF
Sprinklers = 3 psf

} 11 PSF

$\boxed{\text{Total New Roof DEAD LOAD} = 156 \text{ PSF}}$

Adjusted Floor Dead Load:

Superimposed / Misc

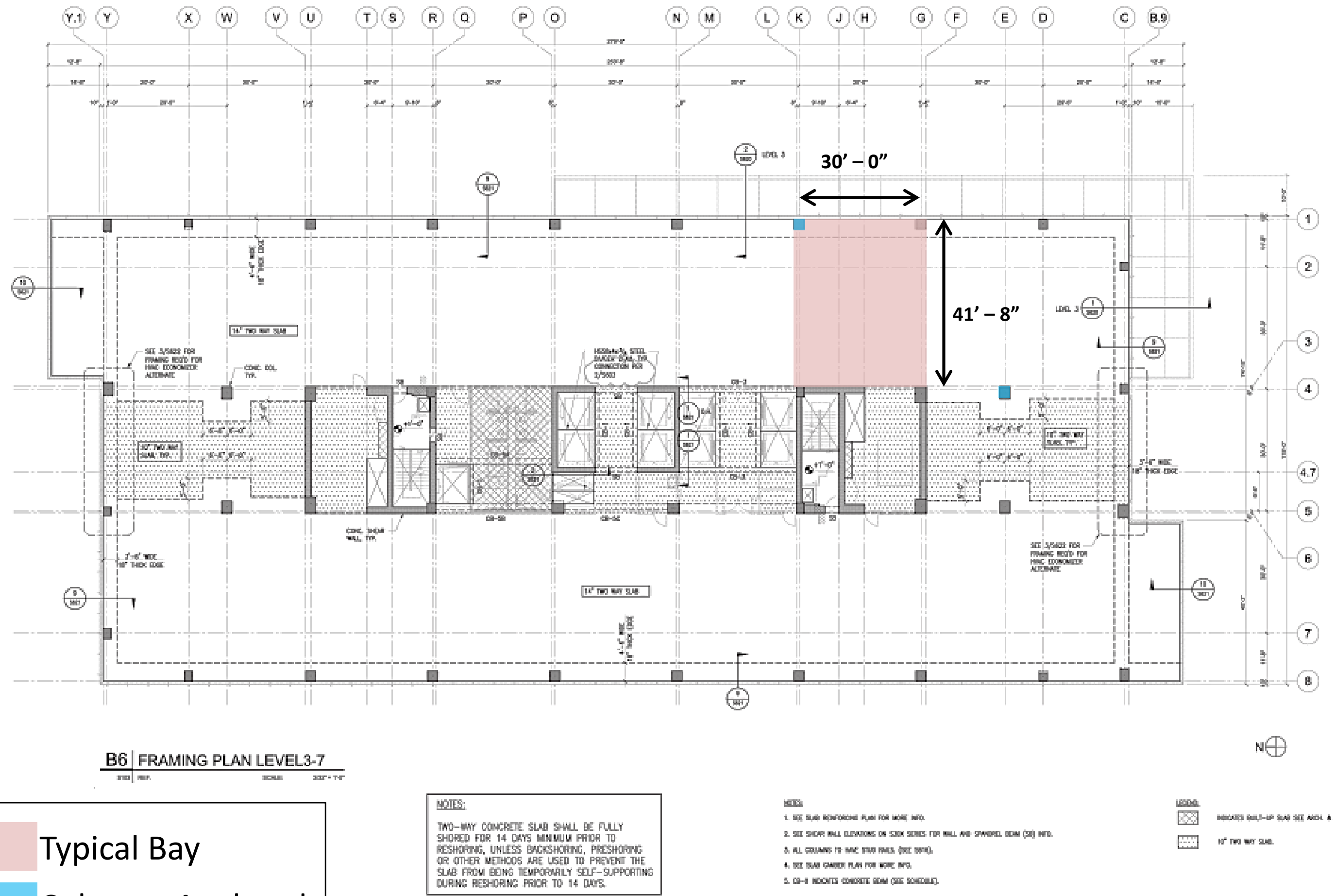
Ceilings = 5 PSF
MEP = 15 PSF
(includes raised floor system)
Sprinklers = 3 PSF

} 23 PSF

$\boxed{\text{Typical Floor Bay New Dead Load} = 200 \text{ PSF}}$

# GRAVITY SPOT CHECKS OF EXISTING SYSTEM

TYPICAL BAY AND COLUMNS ANALYZED FOR GRAVITY LOADS




Typical Bay

Columns Analyzed

Typical Bay Used For Tech 3:

LEVEL 3-7 Between Column lines K and G and column lines 1 and 4, see Appendix A for floor plan.

Check Deflection Control: Columns: 24" x 24"

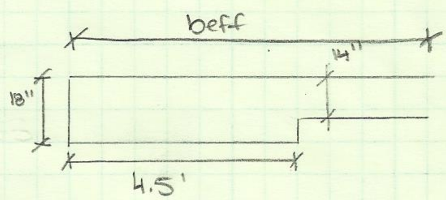
$\ell n_1 = 30' + 11.5' - 12''/12 = 40.5$ ft (long direction)

$\ell n_2 = 30' - 1' = 29$ ft (short direction)

$\beta = \frac{40.5}{29} = 1.397 < 2$ ✓

Table 9.5 (c)
- without drop panels
- exterior panels
- with edge beams
- $f_y = 60{,}000$ PSI

Find $\alpha_{fm}$:




$\alpha_f = \frac{E_{cb} I_b}{E_{cs} I_s}$

$I_b = \frac{1}{12}(4.5 \times 12)(18)^3$
$= 26244 \text{ in}^4$

$b + (a-h) \leq b + 4h$
$4.5(12) + (18-4) \leq 4.5(12) + 4(4)$
$68'' \leq 70''$
$b_{eff} = 68''$

$I_s = \frac{1}{12}(68)(14)^3$
$= 15549 \text{ in}^4$

$\alpha_f = \frac{26244}{15549} = \underline{1.69} > 0.8$ ✓

$h_{min} = \ell n/33 = 40.5 \text{ ft}/33 = 1.2'$

$= 14.4''$

$h_{provided} = 14''$

$h_{min} = 14.4'' < h_{provided} = 14''$ ✗

* Reason cambering of the slab was done. Most likely, calculations were don't to determine actual slab deflections

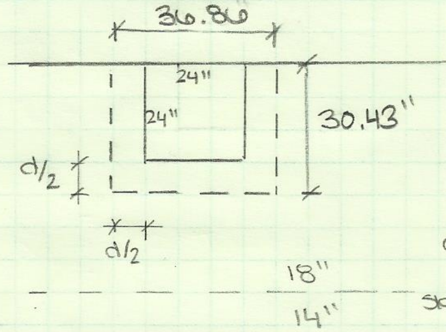
Check Punching Shear (two-way shear)

- Punching shear will control over one-way shear
- Punching shear at the exterior column will control over the interior.

Edge Column: Dead = 215 PSF + (4/12)(150) = 265 PSF
Live = 80 PSF

$q_u = 1.2D + 1.6L$
$= 1.2(265) + 1.6(80)$

$q_u = 446$ PSF (@ thickened edges)




$d = 18" - 3/4" - \frac{1}{2}(0.79) = 12.86"$

$f'c = 5000$ psi

$d/2 = 8.625"$

$b_o = 36.86" + 2(30.43")$ $\lambda = 1.0$ (NW concrete)

$b_o = 97.72$

$\beta = 24"/24" = 1.0$

$\alpha_s = 30$ (edge columns)

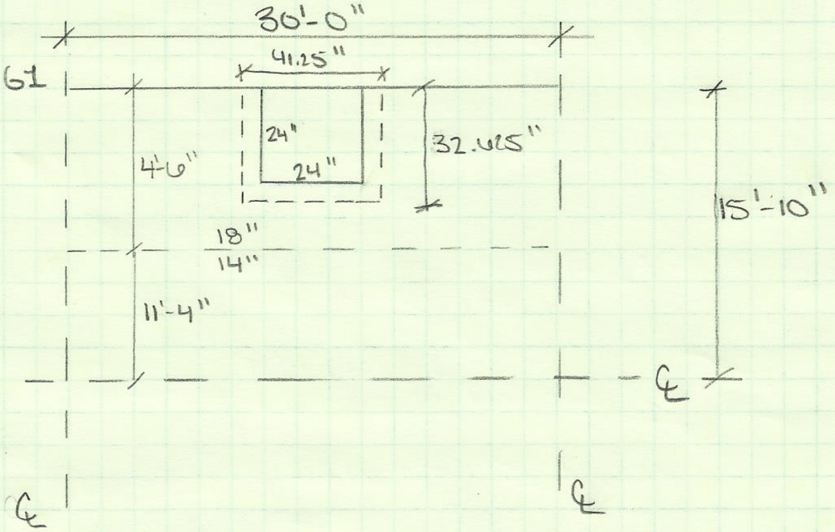
$V_c = $
$(2 + 4/1.0) = 6$ $(2 + 4/\beta)$
$\frac{30 \cdot 17.25}{106.5} + 2 = 6.86$ $(\frac{\alpha_s \cdot d}{b_o} + 2)$
min 4

$V_c = 4\lambda\sqrt{f'c}\, b_o \cdot d$
$V_c = 4(1.0)\sqrt{5000}(97.72)(17.25)/1000 = 477^k$

$\phi V_c = 0.75(477^k)$

$\phi V_c = 358^k$

@ Column G1




$q_u = 446$ PSF @ thick slab edge

$q_u = 1.2(215) + 1.6(80) = 386$ PSF @ 14" slab

$$V_u = (0.446)\left(4.5' \times 30' - \left(\tfrac{41.25}{12}\right)\left(\tfrac{32.625}{12}\right)\right) + 0.386(11.33' \times 30')$$

$V_u = 187^k$

$\phi V_c = 358^k \geq V_u = 187^k$ ✓

$\boxed{\text{18" edge slab passes for punching shear at edge column.}}$

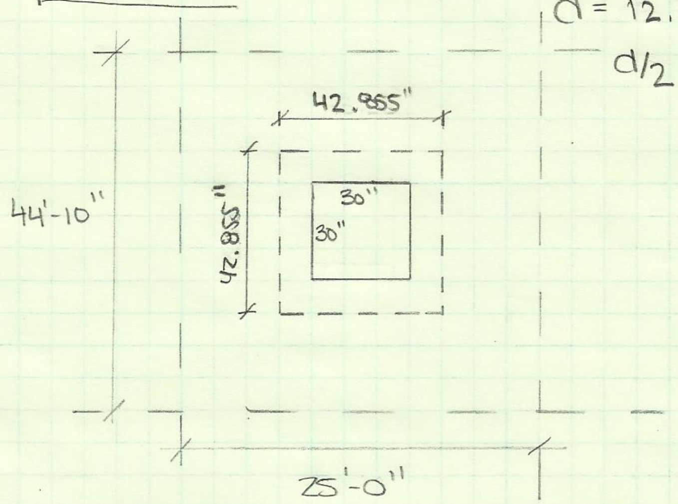
Interior Column: (not in typical bay - Column E4)

$q_u = 386$ PSF

$d = 14'' - 3/4'' - 0.79/2$

$d = 12.855''$

$d/2 = 6.428''$




$b_o = (42.855'')(4) = 171.42''$

$\beta = 30''/30'' = 1.0$

$\alpha_s = 40$

$$V_c = \min \left| \begin{array}{l} (2 + 4/1.0) = 6 \\ \frac{(40)(13.25)}{173} + 2 = 5.06 \\ 4 \end{array} \right.$$

$V_c = 4(1.0)\sqrt{5000\,psi}\,(171.42'')(13.25'')/1000$
$= 642.4^k$

$\phi V_c = 0.75(642.4^k)$

$\phi V_c = 481.8^k$

$$V_u = (0.386)\left[44.83' \times 25' - \left(\frac{43.25}{12}\right)^2\right]$$

$$V_u = 427.6^k$$

$$\phi V_c = 481.8^k \geq V_u = 427.6^k \checkmark$$

$$\boxed{\text{14" Slab passes for punching shear at interior column}}$$

Check one-way shear:

@ Column G1 30' direction

$V_u = (30')(15.833' - 24"/12 - 16.855"/12)(.386\text{ ksf})$
$= 143.5^k$

$V_c = 2\lambda\sqrt{f'c}\ b_w \cdot d$ $\qquad d = 18" - 3/4 - 0.79/2 = 16.855"$

$= 2(1.0)\sqrt{5000\text{ psi}}\,(30' \times 12)(16.855)/1000$
$= 858.2^k$

$\phi V_c = 0.75(858.2)$
$= 643^k$

$\phi V_c = 643^k \geq V_u = 143.5^k$ ✓

@ Column G1 15'-10" direction

$V_u = (4.5')(30/2 - 1 - 16.855/12)(0.446\text{ ksf})$
$+ (11 + 4/12)(30/2 - 1 - 16.855/12)(0.386\text{ ksf})$
$= 80.17^k$

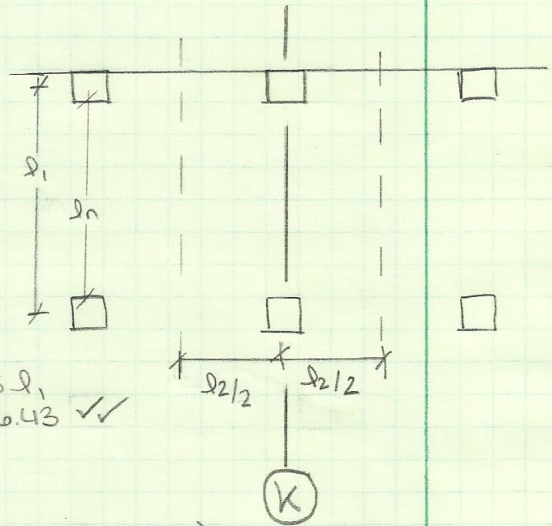
$V_c = 2\lambda\sqrt{f'c}\ b_w \cdot d$
$= 2(1.0)\sqrt{5000\text{ psi}}\,(15.833 \times 12)(16.855)/1000$
$= 452^k$

$\phi V_c = 0.75(452^k)$
$= 339.7^k$

$\phi V_c = 339.7^k \geq V_u = 80.17^k$ ✓

$\boxed{\text{One-way shear strength is adequate}}$

## Check Moment Capacity:




Along Column line K :

$\ell_2 = 30'$ $\quad \ell_1 = 40.67'$

$\ell_n = 40 + \frac{8}{12} - {}^{12''}/_{12''} - {}^{6''}/_{12''}$
$= 39'-2''$
$= 39.167'$

$\ell_n = 39.167'$ $\qquad \ell_n \geq 0.65\,\ell_1$
$\qquad 39.167 \geq 26.43$ ✓✓

$q_u = 386$ PSF

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8} \qquad (\text{ACI 318-11 §13.6.2})$$

$$= \frac{(.386\,\text{ksf})(30')(39.167')^2}{8}$$

$M_o = 2221$ k-ft

Slab w/ out beams btw int supports :

$M_{int}^- = 0.7\,(2221 \text{ k-ft}) \rightarrow M_{int}^- = 1554.7$ k-ft

$M^+ = 0.52\,(2221 \text{ k-ft}) \rightarrow M^+ = 1155$ k-ft

$M_{ext}^- = 0.26\,(2221 \text{ k-ft}) \rightarrow M_{ext}^- = 577.5$ k-ft

Distribute to Column Strip and Middle Strip :

$$\%^- \text{ int. column strip} = 75 + 30\left(\frac{\cancel{\alpha_{f_1} \ell_2}^{\,0}}{\ell_1}\right)(1 - \ell_2/\ell_1)$$

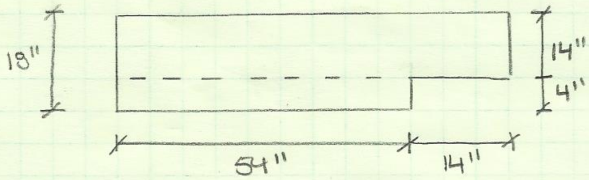
$\%^-_{int.col} = 75\%$

$M^-_{int.col} = 1166$ ft-k

$$\%^- \text{ ext. column strip} = 100 - 10\,\beta_t + 12\,\beta_t\left(\frac{\alpha_{f_1} \cdot \ell_2}{\ell_1}\right)\left(1 - \frac{\ell_2}{\ell_1}\right)$$

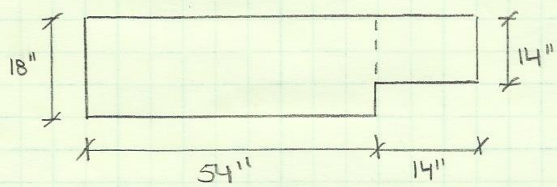
$$\beta_t = \frac{\cancel{E_{cb}} \cdot C}{2\,\cancel{E_{cs}} \cdot I_s} = \frac{C}{2 I_s}, \quad I_s = 15549 \text{ in}^4$$

$$C = \Sigma\,(1 - 0.63\,x/y)\,\frac{x^3 \cdot y}{3}\;,\quad x < y$$




$$C = (1 - 0.63\,\tfrac{4}{54})\left(\frac{4^3 \cdot 54}{3}\right) + (1 - 0.63\,\tfrac{14}{68})\left(\frac{14^3 \cdot 68}{3}\right)$$

$$C = 55{,}228 \text{ in}^4$$




$$C = (1 - 0.63\,\tfrac{18}{54})\left(\frac{18^3 \cdot 54}{3}\right) + (1 - 0.63\,\tfrac{14}{14})\left(\frac{14^3 \cdot 14}{3}\right)$$

$$C = 87669 \text{ in}^4$$

$$C_{MAX} = \left|\begin{array}{l} 55228 \text{ in}^4 \\ 87669 \text{ in}^4 \end{array}\right. \rightarrow \underline{C = 87669 \text{ in}^4}$$

$$\beta_t = \frac{87669 \text{ in}^4}{2(15549 \text{ in}^4)} = 2.82$$

$$\alpha_f = 1.69$$

$$\%^-\text{ext. col. strip} = 100 - 10(2.82) + 12(2.82)\left(\frac{1.69 \cdot 30}{40.67}\right)\left(1 - \frac{30}{40.67}\right)$$

$$\%^-\text{ext. col.} = 82.9\%$$

$$\underline{M^-_{ext.col} = 478.7 \text{ ft-k}}$$

$$\%^+_{cop.} = 60 + 30\left(\cancel{\frac{\alpha_{f1} \cdot \ell_2}{\ell_1}} \xrightarrow{} 0\right)(1.5 - \ell_2/\ell_1)$$

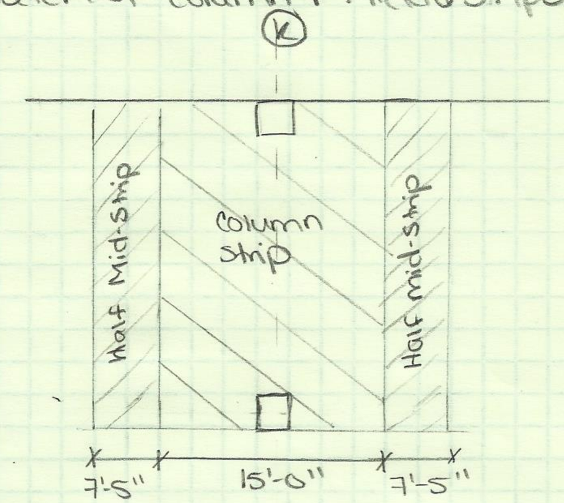
$$\underline{M^+_{cop.} = 693 \text{ k-ft}}$$

Summary of Moments for Span @ Col. Line K

$M^-_{int.col} = 1166$ ft-k
$M^-_{ext.col} = 479$ ft-k
$M^+_{col} = 693$ ft-k

$M^-_{int.mid} = 389$ ft-k
$M^-_{ext.mid} = 98.5$ ft-k
$M^+_{mid} = 462$ ft-k

Sketch of Column & Middle Strips:




$d = 14" - 3/4" - 0.79/2$
$= 12.86"$

Check Reinforcement for Bending Capacity:

$M^+$ column strip $= 693$ ft-k

As → #5 @ 6" + 3 #5 → 33 #5

$A_s = 33(0.31 \text{ in}^2) = 10.23 \text{ in}^2$

$$a = \frac{(10.23 \text{ in}^2)(60{,}000 \text{ psi})}{0.85(5000 \text{ psi})(15\times12)} = 0.802$$

$\phi M_n = 0.9(10.23)(12.86 - 0.802/2)(60{,}000 \text{ psi})$
$= 6882601$ in-lb
$= 573.6$ ft-k

$\phi M_n = 573.6^{'k} < M^+_{col} = 693^{'k}$ ✗

$\boxed{\text{Inadequate Section @ C.S. Midspan}}$

Try Mid. col. strip and mid strip together @ $M_u^+$:

$M_u^+ = 1155'^k$

$A_s \rightarrow 10.23 in^2 + (31)(0.31 in^2) = 19.84 in^2$

$a = \frac{(19.84)(60{,}000)}{0.85(5000)(30\times12)} = 0.778$ in

$\phi M_n = 0.9(19.84)(12.86 - 0.778/2)(60{,}000)/12\cdot1000$
$= 1113$ ft-k

$\phi M_n = 1113$ ft-k $\leq M_u^+ = 1155'^k$ X

Although the section still appears to be inadequate, including the col. strip and middle strip as one calculation shows that the strips must resist the $M_u^+$ together. Therefore, the rest of the moment checks will combine the col. strip and middle strip!

Column Strip + Middle Strip @ Interior Column, $M^-_{int}$:

$M^-_{int} = 1555'^k$

$A_s^- = \#6 @ 16'' + 45 \#6$
$= (22+45)(0.44 in^2) = 29.48 in^2$

$a = \frac{(29.48 in^2)(60{,}000 psi)}{0.85(5000)(30\times12)} = 1.15$ in

$\phi M_n = 0.9(29.48)(12.86 - 1.15/2)(60{,}000)/12\cdot1000$
$= 1629.7$ ft-k

$\phi M_n = 1629.7$ ft-k $\geq M^-_{int} = 1555$ ft-k X

$\boxed{\text{Adequate capacity for moment strength}}$

Column Strip + Middle Strip @ Ext. Column, $M^-_{ext}$:

$M^-_{ext} = 577.5^{'k}$

As → #6 @ 16", 20-#6

$As = (22+20)(0.44\ in^2) = 18.48\ in^2$

$a = \dfrac{(18.48\ in^2)(60000\ psi)}{0.85(5000)(30\times12)} = 0.725\ in$

$\phi Mn = 0.9(18.48\ in^2)(12.86 - 0.725/2)(60{,}000)/12\cdot1000$

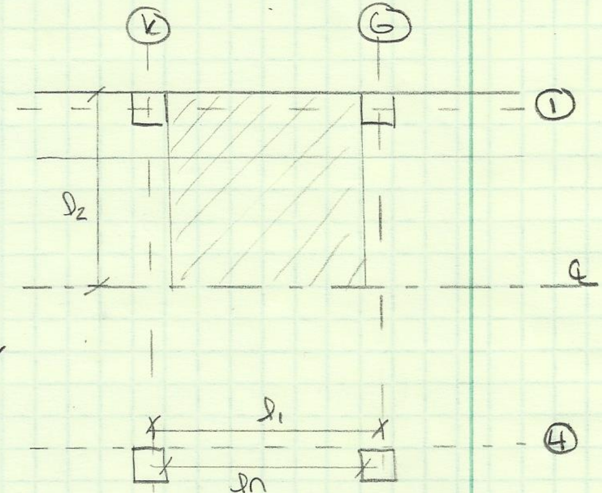
$\phi Mn = 1039\ ft\text{-}k$

$\phi Mn = 1039\ ft\cdot k \geq M^-_{ext} = 577.5\ ft\text{-}k$ ✓

$\boxed{\text{Adequate for moment strength for slab @ ext. column}}$

Check Moment Capacity:

Along Columnline ①:




$l_2 = 20.83$ ft

$l_1 = 30$ ft

$l_n = 30\text{ ft} - 24''/12$
$= 28$ ft

$l_n \geq 0.65\, l_1$
$28' \geq 0.65(30) = 19.5'$ ✓

$q_u = 386$ PSF

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{(0.386\text{ ksf})(20.83')(28')^2}{8}$$

$M_o = 788$ ft-k

Slab w/ out Beams btw int supports - Interior Span:

$M_u^- = 0.65(788^{\text{'k}}) = 512.2^{\text{'k}}$

$M_u^+ = 0.35(788^{\text{'k}}) = 275.8^{\text{'k}}$

Check $M_u^-$ of combined C.S. and M.S.:

$M_u^- = 512.2^{\text{'k}}$

$A_s^- \rightarrow$ #6 @ 16 + 2-#6

$A_s = (15+2)(0.44\text{ in}^2) = 7.48\text{ in}^2$

$$a = \frac{(7.48\text{ in}^2)(60{,}000\text{ psi})}{0.85(5000)(20.83\times 12)} = 0.422\text{ in}$$

$\phi M_n = 0.9(7.48)(12.86 - 0.422/2)(60{,}000)/12{,}000$
$= 425.8^{\text{'k}}$

$\phi M_n = 425.8^{\text{'k}} < 512.2^{\text{'k}} = M_u^-$ ✗

$\boxed{\text{Slab is inadequate for moment capacity (except edge beam was not accounted for)}}$

Check $M_u^+$ of combined C.S. and M.S.:

$M_u^+ = 275.8^{1k}$

$A_s \rightarrow$ #5 @ 6"

$A_s = 41(0.31) = 12.71\ in^2$

$$a = \frac{(12.71\ in^2)(60{,}000\ psi)}{0.85(5000)(20.83 \times 12)} = 0.718\ in$$

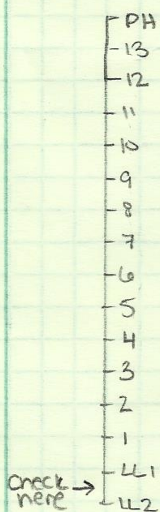
$$\phi M_n = 0.9(12.71)(12.86 - 0.718/2)(60{,}000)/12{,}1000$$
$$= 715\ ft\text{-}k$$

$\phi M_n = 715^{1k} > M_u^+ = 275.8^{1k}$ ✓

Slab is adequate at midspan for moment capacity

## Column Gravity Checks:

Interior Column – Column E-4 – 30"x48", 22-#10 bars




Trib Area @ each level = $(50\,ft/2 \times 35.33\,ft/2)$
$= 442\ ft^2$

Pent House:

Dead = 156 PSF $(442\ ft^2)$ = 68.95 K
Roof Live = 20 PSF $(442\ ft^2)$ = 8.84 K

Levels 13 to 3:

Dead = (200 PSF)(221) + (150)(221) = 77.35K
Live = (60 PSF)(442) = 26.52 k

Level 2:

Dead = (200 PSF)$(442\ ft^2)$ = 88.4k
Live = (60 PSF)$(442\ ft^2)$ = 26.52 k

Level 1:

Dead = (150 PSF)$(442\ ft^2)$ = 66.3 k
Live = (75 PSF)$(442\ ft^2)$ = 33.15k

Influence Area = $(442\ ft^2)(2)$ = $884\ ft^2$

Live Load Reduction:

$$L = 80\left(0.25 + \frac{15}{\sqrt{884}}\right) = 0.75$$

max 0.4 (for two or more floors above)

L = 80 (0.75) = 60 PSF
L = 100 (0.75) = 75 PSF } used above!

L = 40 (0.75) = 30 PSF – used below!

Lower level 1:

Dead = (200)(442) = 88.4k
Live = (30)(442) = 13.26k

$$DL = 68.95^k + 77.35^k(11) + 88.4^k + 66.3^k + 884^k$$

$$+ \frac{150\ PCF}{1000}\left[\left(\frac{30}{12}\times\frac{48}{12}\times 54.51\right) + \left(\frac{24}{12}\times\frac{24}{12}\times 144.19\right)\right]$$

$$\underline{DL = 1331^k}$$

$$LL = 26.52^k(12) + 33.15^k + 13.26^k$$

$$\underline{LL = 364.7^k}$$

$$\underline{L_R = 8.84^k}$$

$$P_u = 1.2D + 1.6L + 0.5L_R$$

$$P_u = 1.2(1331^k) + 1.6(364.7^k) + 0.5(8.84^k)$$

$$\boxed{P_u = 2185^k}$$

Column Strength: §10.3.6.2 - ACI 318-11

Nonprestressed members, tie reinforcement

$$\phi P_{n,max} = 0.8\phi\left[0.85 f'_c (A_g - A_{st}) + f_y (A_{st})\right]$$

$\phi = 0.65$

$A_g = 30" \times 48" = 1440 \text{ in}^2$

$A_{st} = 22\,(1.27 \text{ in}^2) = 27.94 \text{ in}^2$

#10 bars

$f'_c = 6000$ PSI

$$\phi P_{n,max} = 0.8(0.65)\left[0.85(6000\text{ psi})(1440\text{ in}^2 - 27.94\text{ in}^2) + 60{,}000\text{ psi}\,(27.94\text{ in}^2)\right]$$

$\phi P_{n,max} = 4617^k$

$P_u = 2185^k \leq \phi P_n = 4617^k$ ✓

$\boxed{\text{Interior Column E-4 Passes for compression gravity loads}}$

Exterior Column - Column K-1 - 30"x30", 24-#11 bars

Pent House: Trib Area = 625 $ft^2$

Dead = (156 PSF)(625 $ft^2$) = $97.5^k$
Roof live = (20 PSF)(625 $ft^2$) = $12.5^k$

Levels 13 to 3:

Dead = (250 PSF)(4.5' x 30') + (200 PSF)(30' x 15.83')
= 128.73 k
Live = (625 $ft^2$)(53.6 PSF) = $33.5^k$

Levels 2 and Ground: None

Lower Level 1:

Dead = (150 PCF)(625 $ft^2$)(18"/12) = $130^k$
Live = (26.8 PSF)(625 $ft^2$) = $16.75^k$

Live Load Reduction: $A_{infl}$ = 1250 $ft^2$

$$L = 80 \left| \begin{array}{l} 0.25 + \frac{15}{\sqrt{1250}} = 0.67 \\ \\ \text{max } 0.4 \end{array} \right.$$

$$\left.\begin{array}{l} L = 53.6 \text{ PSF} \\ L = 40(0.67) = 26.8 \text{ PSF} \end{array}\right\} \text{used above!}$$

$DL = 97.5^k + 128.73^k(11) + 130^k$

$+ \frac{150}{1000} PCF\left(\frac{30}{12}\times\frac{30}{12}\times 28.17' + \frac{24}{12}\times\frac{24}{12}\times 146.2'\right)/1000$

$DL = 1758^k$

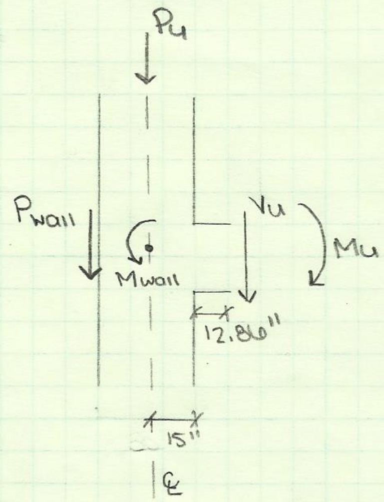
$LL = 33.5^k(11) + 16.75^k = 385.25^k$

$L_R = 12.5^k$

$P_u = 1.2(1758^k) + 1.6(385.25^k) + 0.5(12.5^k)$

$\boxed{P_u = 2732 \text{ k}}$

Find Moment to be applied to Column centerline:




$P_u = 2732^k$

$V_u = 80.17^k$

$\bar{M}_{u_{ext}} = 577^{'k}$

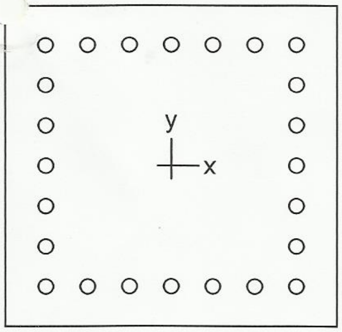
$M_{v,u} = 80.17^k(12.86'' + 15'')/12$
$= 186.1^{'k}$

$P_{wall} = (0.118^{k/ft})(30\ ft) = 3.54^k$
$M\ wall = (3.54^k)(15''/12) = 4.43\ ft\text{-}k$

$\left.\begin{array}{l} P_u = 2736^k \\ M_u = 758.67^{'k} \end{array}\right\}$ Column loads

* The interaction diagram was found using spColumn to save time and obtain accurate results. The 7000 psi interaction diagram can be seen on the following page. These results were compared to those for the same column with 4000 psi concrete. The values were reasonable in comparison to the design guide.

Column k-1 Interaction




30 x 30 in

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

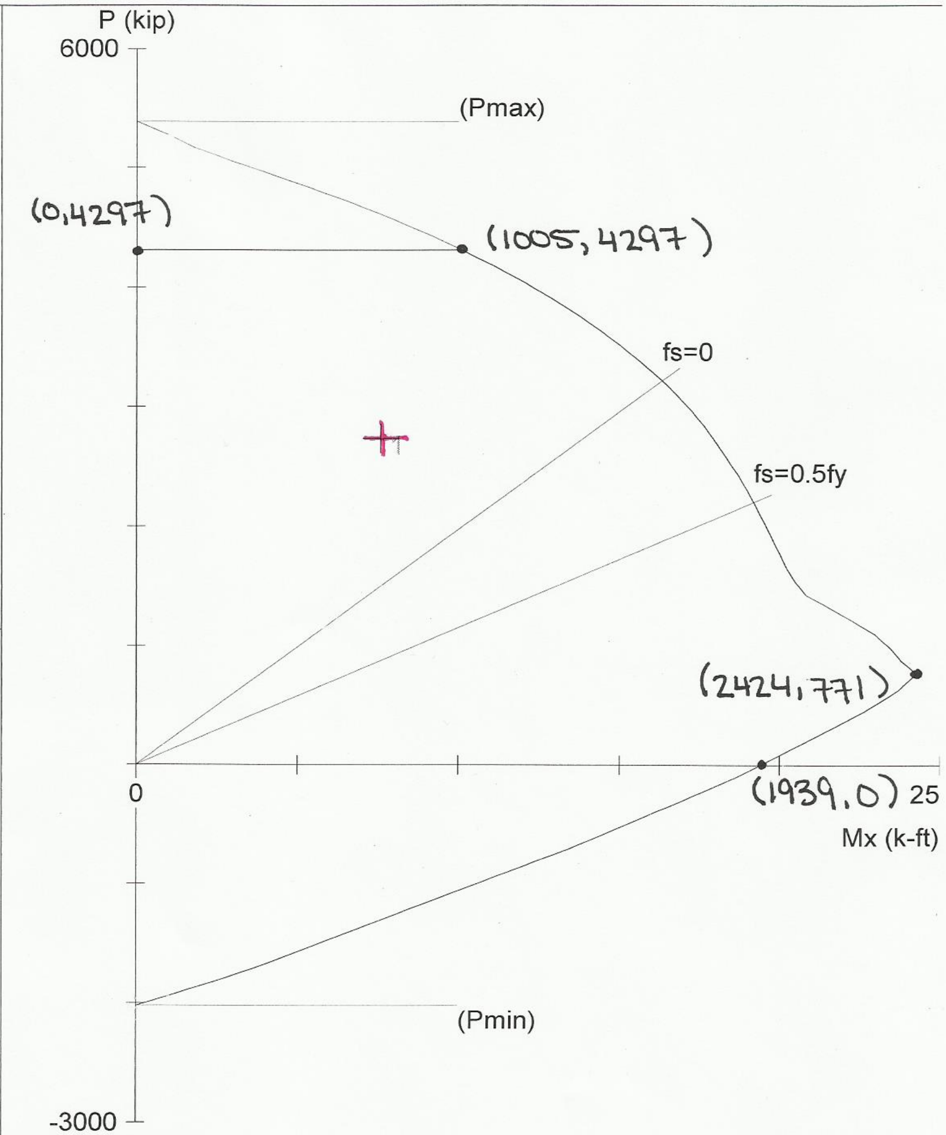
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/08/13

Time: 21:16:21




spColumn v4.81. Licensed to: Penn State University. License ID: 59919-1033951-4-22545-2CF68

File: e:\fall 2013\ae 481w\tech 3\column k-1 spslab.col

Project: Tech Report 3

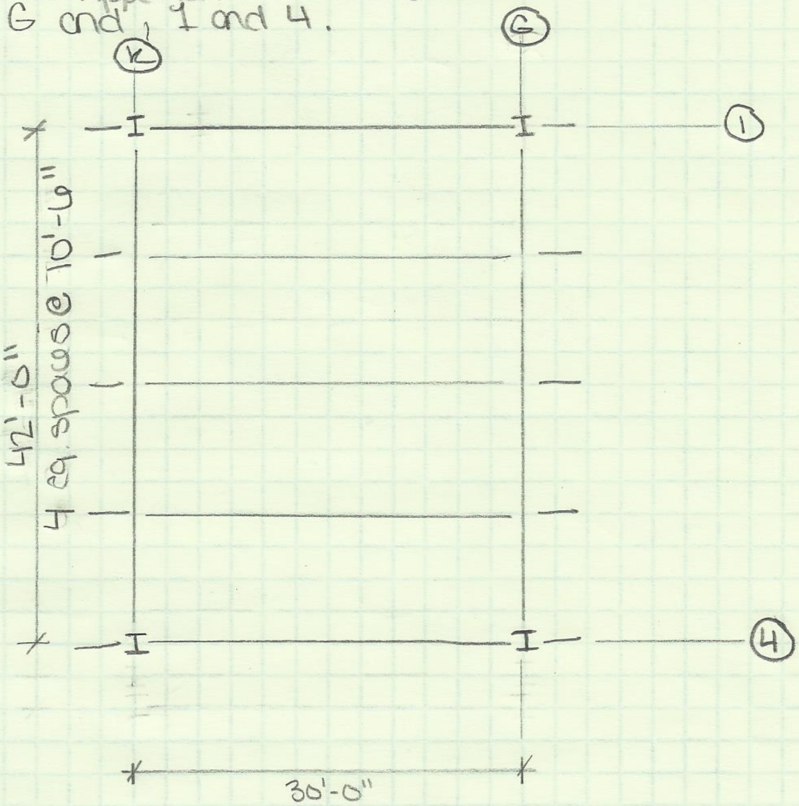
| | | | |
|---|---|---|---|
| Column: Column k-1 | | Engineer: AMS | |
| f'c = 7 ksi | fy = 60 ksi | Ag = 900 in^2 | 24 #11 bars |
| Ec = 4769 ksi | Es = 29000 ksi | As = 37.44 in^2 | rho = 4.16% |
| fc = 7 ksi | | Xo = 0.00 in | Ix = 67500 in^4 |
| e_u = 0.003 in/in | | Yo = 0.00 in | Iy = 67500 in^4 |
| Beta1 = 0.7 | | Min clear spacing = 2.36 in | Clear cover = 3.00 in |
| Confinement: Tied | | | |
| phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65 | | | |

# ALTERNATIVE FLOOR SYSTEM

# DESIGN #1: NON-COMPOSITE STEEL

# Non-Composite Steel System - Design 1

- Same as typical bay used for gravity checks see Appendix A. Btw column lines K and G and, 1 and 4.




## Deck Design:

2 HR fire rating - Unprotected deck → 4½" topping

LL = 80 PSF

MISC DL = 23 PSF

Superimposed live load = 103 PSF

Span = 10.5 ft

Try: 2VLI18 with t=4.50 in, total thickness=6.5"
NW

Check: 2VLI18 w/ 4.5" topping, 6.5" total thickness

1. Unshored Construction - 3 span = 10'-11" > 10'-6" ✓

   Shoring not required

2. $W_{LL} + W_{misc,DL} \leq$ Superimposed Live Load

   80 PSF + 23 PSF ≤ 203 PSF (@ 10'-6" span)

   103 ≤ 203 PSF ✓

   Deck is adequate for strength

$\boxed{\text{Use 2VLI18 with 4.5" NW Topping}}$

Determine New LOADS:

Deck = 69 PSF
Framing Allowance = 10 PSF
Superimposed dead = 23 PSF

$\}$ DL = 102 PSF

LL = 80 PSF (for office space)

Determine Joist Design:

Is live load reduction possible?

$K_{LL}A_T = 30' \times 2(10.5')$
$= 630\ ft^2 > 400\ ft^2 \rightarrow$ Yes

$L = 80 \left| \begin{array}{l} \left(0.25 + \frac{15}{\sqrt{630}}\right) = 0.848 \\ \\ \text{max}\ \ 0.5 \end{array} \right.$

$L = 80(0.848) = \underline{67.81\ PSF}$

Design load:

$W_u = 1.2(102\ PSF) + 1.6(67.81\ PSF)$
$= 231\ PSF$

$\omega_u = (231\ PSF)(10.5')/1000$
$= 2.43\ k/ft$

$\underline{\omega_u = 2.43\ ^{k}/_{ft}}$

Beam length, $l$: $l = 30\ ft$

Design Moment:

$M_u = \frac{\omega l^2}{8} = \frac{(2.43\ k/ft)(30')^2}{8}$

$\underline{M_u = 273.4\ ft\text{-}k}$

Try W16x40: $\phi M_n = 274\ ft\text{-}k$, $I_x = 518\ in^4$

$\phi M_n = 274^{'k} \geq M_u = 273.4^{'k}$

Check deflections:

$\Delta_{LL} \leq L/360$

$\omega_{LL} = (67.81\ PSF)(10.5') = 712\ lb/ft = 0.712\ ^{k}/_{ft}$

$\Delta_{max} = \frac{5\omega_{LL}L^4}{384\ EI} = \frac{5(0.712\ ^{k}/ft)(30\ ft)^4\ (1728)}{384(29000\ ksi)(518\ in^4)}$

$\underline{\Delta_{max} = 0.864\ in}$

Deflection check for joist cont.:

$$\Delta_{LL} \leq \frac{(30')(12)}{360} = 1.0 \text{ in}$$

$$\Delta_{LL} = 0.864 \text{ in} \leq 1.0 \text{ in} \checkmark$$

$$\boxed{\text{Use W16x40 for joists spaced at 10.5' o.c.}}$$

Check SW:
$(40 \text{ lb/ft})/10.5 = 3.81 \text{ PSF}$
$3.81 \text{ PSF} < 10 \text{ PSF} \checkmark$

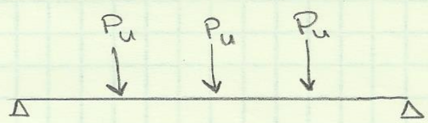
## Girder Design:

Live load reduction: $K_{LL}A_T = 2(30' \times 42') = 2520 \text{ ft}^2$

$$L = 80 \left| \begin{array}{l} (0.25 + 15/\sqrt{2520}) = 0.549 \\ \\ \text{max } 0.5 \end{array} \right.$$

$$L = 80(0.549) = \underline{43.92 \text{ PSF}}$$

Design loads:




$$W_u = 1.2(102 \text{ PSF}) + 1.6(43.92 \text{ PSF}) = 192.7 \text{ PSF}$$

$$W_u \text{ on joist} = (192.7 \text{ PSF})(10.5')/1000 = 2.02 \text{ k/ft}$$

$$P_u = (2.02 \text{ k/ft})(30 \text{ ft}) = 60.6^k$$

Beam length: $l = 42'\text{-}0''$

Design Moment: (using Table 3-22a, of Steel Manual, 14th ed.)

$$M_u = aPL = 0.5(60.6^k)(42')$$
$$= 1272.6 \text{ ft-k}$$

Try W30 x 108:

$\phi M_n = 1300$ ft-k, $I_x = 4470$ in$^4$

$\phi M_n = 1300^{'k} \geq M_u = 1272.6^{'k}$ ✓

Check deflections:

$\Delta_{LL} \leq L/360$

$\Delta_{LL} \leq 42(12)/360 = 1.4$ in

$$\Delta_{max} = \frac{0.05 P l^3}{EI}$$

Live load unfactured

$P = (43.92 \text{ PSF})(10.5')(30')/1000 = 13.83^k$

$$\Delta_{max} = \frac{0.05(13.83^k)(42 ft)^3}{(29000 \text{ ksi})(4470 \text{ in}^3)}(1728)$$

$\Delta_{max} = 0.683$ in

$1.4'' > \Delta_{max} = 0.683$ in ✓

$\boxed{\text{Use W30 x 108 for each girder}}$

Check SW:

$(108 \text{ lb/ft})/30 + 3.81$
$= 7.41 \text{ PSF} < 10$ PSF ✓

Determine Edge Joist design:

Live load reduction:

$K_{LL}A_T = 30' \times (10.5') = 315\ ft^2 < 400\ ft^2$

No LL Reduction

$L = 80\ PSF$

Design load:

$DL = 102\ PSF$
$LL = 80\ PSF$
$DL_{wall} = 118\ PLF$

$w_D = (102\ PSF)(10.5'/2)/1000 = 0.536\ k/ft$
$w_L = (80\ PSF)(10.5'/2)/1000 = 0.42\ k/ft$
$w_{D,wall} = 0.118\ k/ft$

$w_u = 1.2(0.536 + 0.118) + 1.6(0.42)$
$w_u = 1.46\ k/ft$

Beam length: $l = 30\ ft$

Design Moment:

$$M_u = \frac{w_u l^2}{8} = \frac{(1.46\ k/ft)(30')^2}{8}$$

$M_u = 164.25\ ft\text{-}k$

Try W16x26:

$\phi M_n = 166\ ft\text{-}k, \quad I_x = 301\ in^4$

$\phi M_n = 166^{'k} > M_u = 164.25^{'k}$

Check Deflections:

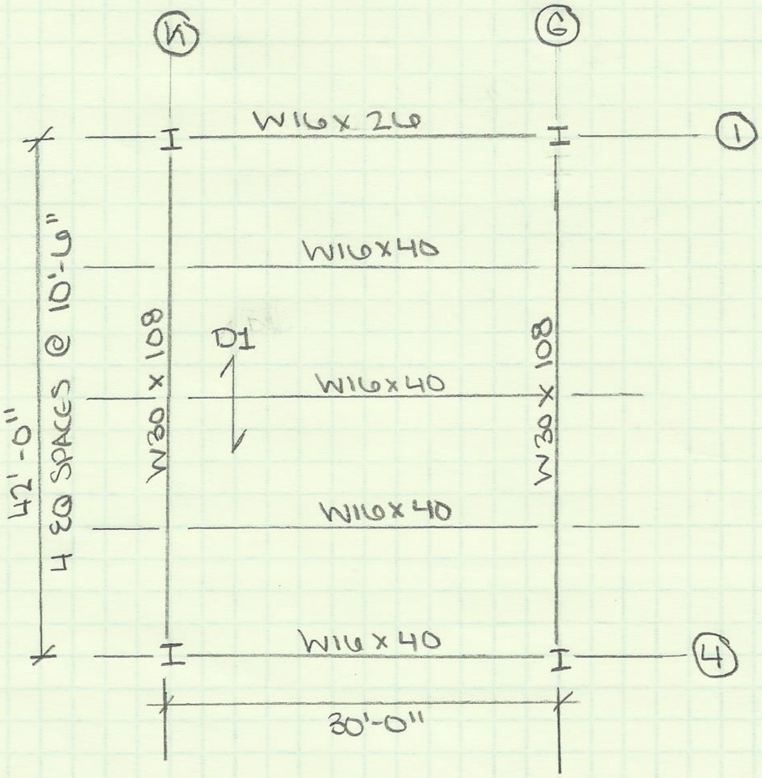
$$\Delta_{LL} \le L/360 = \frac{30(12)}{360} = 1.0\ in$$

$$\Delta_{max} = \frac{5(0.42\ k/ft)(30 ft)^4(1728)}{384(29000\ ksi)(301\ in^4)}$$
$= 0.877\ in$

$1.0\ in > 0.877\ in$ ✓

$\boxed{\text{Use W16 x 26 for edge Joist}}$

Steel Design #1 Summary: Non-Composite




D1: 2VLI 18, 4.5 inch NW concrete topping

# ALTERNATIVE FLOOR SYSTEM

# DESIGN #2: COMPOSITE STEEL

## COMPOSITE STEEL SYSTEM - DESIGN 2

- Same layout as non-composite system (See Pg. 20)

Deck: 2 VLI 18 with 4.5 in NW topping → 6.5" total thickness

LOADS: DL = 102 PSF
LL = 80 PSF

Determine Joist Design:

LL = 67.81 PSF (reduced on pg. 22)

Design load:

$W_u = 1.2(102\ PSF) + 1.6(67.81\ PSF) = 230.9\ PSF$

$W_u = (230.9\ PSF)(10.5')/1000 = \underline{2.42\ k/ft}$

Beam length:

$l = 30\ ft$

Design Moment:

$$M_u = \frac{W_u l^2}{8} = \frac{(2.42\ k/ft)(30\ ft)^2}{8}$$

$\underline{M_u = 272.3\ ft\text{-}k}$

If $a \approx 1' \rightarrow Y_2 = 6.5'' - \frac{1}{2}'' = 6''$

Try $Y_2 = 6''$:

W14x26, $\Sigma Q_n = 173 \rightarrow \phi M_n = 275^{1k} \geq 272.3^{1k}$ ✓

Studs $= 173/17.2 = 10.1 \rightarrow 11 \times 2 = 22\ \frac{studs}{beam}$

W12x26, $\Sigma Q_n = 259 \rightarrow \phi M_n = 291^{1k} \geq 272.3^{1k}$ ✓

Studs $= 259/17.2 = 15 \rightarrow 15 \times 2 = 30\ \frac{studs}{beam}$

W14x22, $\Sigma Q_n = 283 \rightarrow \phi M_n = 294^{1k} \geq 272.3^{1k}$

Studs $= 283/17.2 = 17 \times 2 = 34\ \frac{studs}{beam}$

Impractical

Note:
W14x26 is most economical here; however, W12x26 limits beam depth by two inches. Because the existing slab is 14" in most locations, the W14 is acceptable and will be investigated further.

Determine $b_{eff}$:

$$b_{eff} = \min \left| \begin{array}{l} span/8 \\ \tfrac{1}{2}(\text{dist to adjacent beam}) \end{array} \right. + \min \left| \begin{array}{l} span/8 \\ \tfrac{1}{2}(\text{dist. adj. bm}) \end{array} \right.$$

$$= 2 \times \left| \begin{array}{l} (30')(12)/8 = 45 \text{ in} \\ \tfrac{1}{2}(10.5 \times 12) = 63 \text{ in} \end{array} \right.$$

$b_{eff} = 90$ in

W14x26, 22 studs:

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{22(17.2)}{0.85(4)(90 in)} = 1.23 \text{ in} > 1.0'' \ \times$$

Try W14x26: 28 studs

$\Sigma Q_n = 226$, $\phi M_n = 300'^k$

Studs $= 226/17.2 = 13.1 \rightarrow 14 \times 2 = 28$ studs/beam

$$a = \frac{14(17.2)}{0.85(4)(90)} = 0.787'' < 1'' \ \checkmark$$

Check Unshored Strength:

W14x26, $\phi M_p = 151$ ft-k, $I_x = 245 \text{ in}^4$

$w_u = 1.4(69 \text{ psf} \times 10.5) + 1.4(26) = 1.05$ k/ft

$w_u = 1.2(69 \times 10.5 + 26) + 1.6(20 \times 10.5) = 1.22$ k/ft

$$M_u = \frac{(1.22)(30)^2}{8} = 137.3 \text{ ft}\cdot\text{k} \le \phi M_p = 151'^k \ \checkmark$$

Check wet concrete deflection:

$$W_{wc} = 69(10) + 26 = 0.716 \text{ klf}$$

$$\Delta_{wc} = \frac{5(0.716)(30)^4(1728)}{384(29000)(245)} = 1.32 \text{ in}$$

$$\Delta_{wc,max} = \frac{L}{240} = \frac{30(12)}{240} = 1.5 \text{ in}$$

$$\Delta_{wc} = 1.32 \text{ in} < \Delta_{wc,max} \checkmark$$

Check LL deflection:

$$W_{LL} = (67.81 \text{ PSF})(10.5 \text{ ft})/1000 = 0.712 \text{ klf}$$

$$I_{LB} = 722 \text{ in}^4$$

$$\Delta_{LL} = \frac{5(0.712 \text{ k/ft})(30)^4(1728)}{384(29000)(722)} = 0.620 \text{ in}$$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{(30)(12)}{360} = 1 \text{ in}$$

$$\Delta_{LL} = 0.620 \text{ in} \leq \Delta_{LL} = 1.0 \text{ in} \checkmark$$

$\boxed{\text{Interior Joists will be W14x26 with 28 studs spaced evenly along the beam}}$

Determine Edge Joist Design:

LL = 80 PSF

$W_u = 1.2(102\text{ PSF}) + 1.6(80\text{ PSF}) = 250.4\text{ PSF}$

$w_u = (250.4\text{ PSF})(10.5/2)/1000 + 118/1000 = 1.43\ ^{k}/_{ft}$

wall load

Beam length:

$l = 30\text{ ft}$

Design Moment:

$$M_u = \frac{w_u l^2}{8} = \frac{(1.43)(30)^2}{8} = \underline{160.9^{\text{'k}}}$$

Find beff:

$$b_{eff} = \min\left|\begin{array}{l} \text{span}/8 = \frac{30(12)}{8} = 45'' \\ 3'' + 6'' = 9'' \end{array}\right. + \min\left|\begin{array}{l} 45'' \\ \tfrac{1}{2}(10.5 \times 12) = 63'' \end{array}\right.$$

half-flange width

6" slab overhang assumed

$b_{eff} = 9'' + 63'' = \underline{72''}$

If $a \approx 1''$, $Y_2 = 6.5'' - 1/2 = 6''$

Try $Y_2 = 6''$:

$$1.0 \geq \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} \rightarrow \Sigma Q_n \leq 1.0(0.85)(4)(72) \leq 244.8$$

W10x15 , $\Sigma Q_n = 194$ , $\phi M_n = 170^{\text{'k}}$

Studs = 194 / 17.2 = 11.3 → 12 x 2 = 24 studs/beam

W10x17 , $\Sigma Q_n = 150$ , $\phi M_n = 161^{\text{'k}}$

Studs = 150/17.2 = 8.7 → 9 x 2 = 18 studs/beam

W12x14 , $\Sigma Q_n = 163$ , $\phi M_n = 166^{\text{'k}}$

Studs = 163/17.2 = 9.5 → 10 x 2 = 20 studs/beam

W12x14 with 20 studs is the most economical by inspection. I will continue with this option.

Try W12x14 : 20 studs , $\phi M_n = 166^{1k} \geq M_u$ ✓

$$a = \frac{10(17.2)}{0.85(4)(72)} = 0.7'' < 1.0'' \checkmark$$

Check Unshored Strength:

W12x14 , $\phi M_n = 65.3^{1k}$ , $I_x = 88.6 \text{ in}^4$

$W_u = 1.4(69 \times 10.5/2) + 1.4(14) = 0.527 \text{ k/ft}$

max $W_u = 1.2(69 \times \frac{10.5}{2} + 14) + 1.6(20 \times 10.5/2) = 0.62 \text{ k/ft}$

$W_u = 0.62 \text{ k/ft}$

$$M_u = \frac{(0.62^{1k})(30)^2}{8} = 69.75^{1k} > \phi M_n \text{ X}$$

Try W12x16 : $\Sigma Q_n = 156$ , $\phi M_n = 176^{1k}$

Studs = 156/17.2 = 9.1 → 10x2 = 20 studs/beam

→ W10x17 = more economical

Try W10x17 : 18 studs, $\phi M_n = 161^{1k}$

$$a = \frac{9(17.2)}{0.85(4)(72)} = 0.63'' < 1'' \checkmark$$

Check Unshored strength:

W10x17, $\phi M_n = 70.1^{1k}$ , $I_x = 81.9 \text{ in}^4$

$W_u = 1.2(69 \times \frac{10.5}{2} + 17) + 1.6(20 \times 10.5/2) = 0.623 \text{ k/ft}$

$M_u = 70.09^{1k} \leq \phi M_n = 70.1^{1k}$ ✓

Check Wet Concrete Deflection:

$$W_{wc} = 69\left(\frac{10.5}{2}\right) + 17 = 0.379 \text{ klf}$$

$$\Delta_{wc} = \frac{5(0.379)(30)^4(1728)}{384(29000)(81.9)} = 2.91''$$

$$\Delta_{max} = 1.5'' < \Delta_{wc} = 2.91'' \quad \times$$

* Try W12x16 to avoid camber

Try W12x16 : 20 studs/beam, $\phi M_n = 176^{'k}$

$$a = \frac{10(17.2)}{0.85(4)(72)} = 0.703'' < 1'' \checkmark$$

Check Unshored Strength:

W12x16, $\phi M_n = 75.4^{'k}$, $I_x = 103 \text{ in}^4$

$$W_u = 1.2(69 \times 10.5/2 + 16) + 1.6\left(20 \times \frac{10.5}{2}\right) = 0.62^{k}/ft$$

$$M_u = 69.75^{'k} < \phi M_n = 75.4^{'k} \checkmark$$

Check Wet Concrete Deflection:

$$W_{wc} = 69(10.5/2) + 16 = 0.378^{k}/ft$$

$$\Delta_{wc} = \frac{5(0.378)(30^4)(1728)}{384(29000)(103)} = 2.3''$$

$$\Delta_{wc} = 2.3'' > \Delta_{max} = 1.5'' \quad \times$$

* Would need camber, after looking at other beams for wet concrete deflection off to the side, try W12x26

Try W12x26: $\Sigma Q_n = 198$, $\phi M_n = 262$

Studs $= 198/17.2 = 11.5 \rightarrow 2(12) = 24$ studs

Unshored Strength: Will be OK ✓

Wet Concrete deflection: $W_u = 0.388^{k}/ft$

$$\Delta_{wc} = \frac{5(0.388^{k}/ft)(30)^4(1728)}{384(29000)(204 \text{ in}^4)} = 1.20''$$

$$\Delta_{wc} = 1.20'' < \Delta_{max}\ 1.5'' \checkmark$$

Check LL deflection:

$$W_{LL} = (80\ PSF)(^{10.5}/_{2})/1000 = 0.42\ klf$$

$$I_{LB} = 586\ in^4$$

$$\Delta_{LL} = \frac{5(0.42\ klf)(30)^4(1728)}{384(29000)(586)} = 0.45''$$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{12(30)}{360} = 1''$$

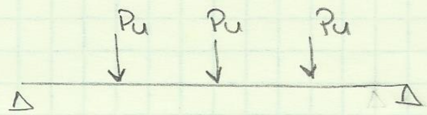
$$\Delta_{LL} = 0.45'' < \Delta_{max} = 1'' \checkmark$$

$\boxed{\text{Exterior Joist will be W12x26 with 24 studs spaced evenly along the beam}}$

Determine Girder design:

$L = 43.92$ PSF (reduced on page 23)

Design loads:




$P_u = 60.6^k$

Beam length: $l = 42'-0''$

Design Moment:

$M_u = 0.5(60.6^k)(42') = \underline{1272.6 \text{ ft-k}}$

If $a \approx 1''$, $Y_2 = 6'' \rightarrow$ Try $Y_2 = 6''$

Determine beff:

$$b_{eff} = 2 \times \min \begin{cases} (42' \times 12)/8 = 63'' \\ \tfrac{1}{2}(30' \times 12) = 180'' \end{cases}$$

$\underline{b_{eff} = 126''}$

Try $Y_2 = 6''$:

$1.0 \geq \dfrac{\Sigma Q_n}{0.85(4)(126)} \rightarrow \Sigma Q_n \leq 428.4$

$\underline{W30\times90}$, $\Sigma Q_n = 329$, $\phi M_n = 1480^{'k}$

studs $= 329/17.2 = 19.1 \rightarrow 20 \times 2 = 40$ studs/bm

$a = \dfrac{20(17.2)}{0.85(4)(126)} = 0.80'' < 1''$ ✓

Try W30x90 : 40 studs, $\phi M_n = 1480^k$

Check Unshored strength:

W30x90, $\phi M_n = 1060^{'k}$, $I_x = 3610 \text{ in}^4$

$W_{u,joists} = 1.2(69 \times 10.5 + 26) + 1.6(20 \times 10.5)$
$= 1.24 \text{ k/ft}$

$P_u = (1.24 \text{ k/ft})(30 \text{ ft}) = 37^k$

M max due to $P_u$ = $0.5(37)(42)$
$= 777 \text{ k-ft}$

M max due to SW = $\dfrac{1.2(90/1000)(42)^2}{8} = 23.8 \text{ k-ft}$

$M_u = 801 \text{ ft-k} < \phi M_n$ ✓

Check wet concrete deflection:

$W_{wc,joists} = 69(10.5) + 26 = 0.75 \text{ k/ft}$

$P_{wc} = (0.75^k/\text{ft})(30') = 22.5^k$

$\Delta_{wc} = \dfrac{0.05 P l^3}{EI} = \dfrac{0.05(22.5)(42)^3(1728)}{29000(3610)}$

$= 1.38''$

$\Delta_{max} = \dfrac{L}{240} = \dfrac{42(12)}{240} = 2.1''$

$\Delta_{wc} < \Delta_{max}$ ✓

Check Live load deflection: $I_{LB} = 5880 \text{ in}^4$

$W_{LL,joists} = (43.92 \text{ psf})(10.5)/1000 = 0.461 \text{ k/ft}$

$P_{LL} = 0.461^k/\text{ft}\,(30') = 13.8^k$

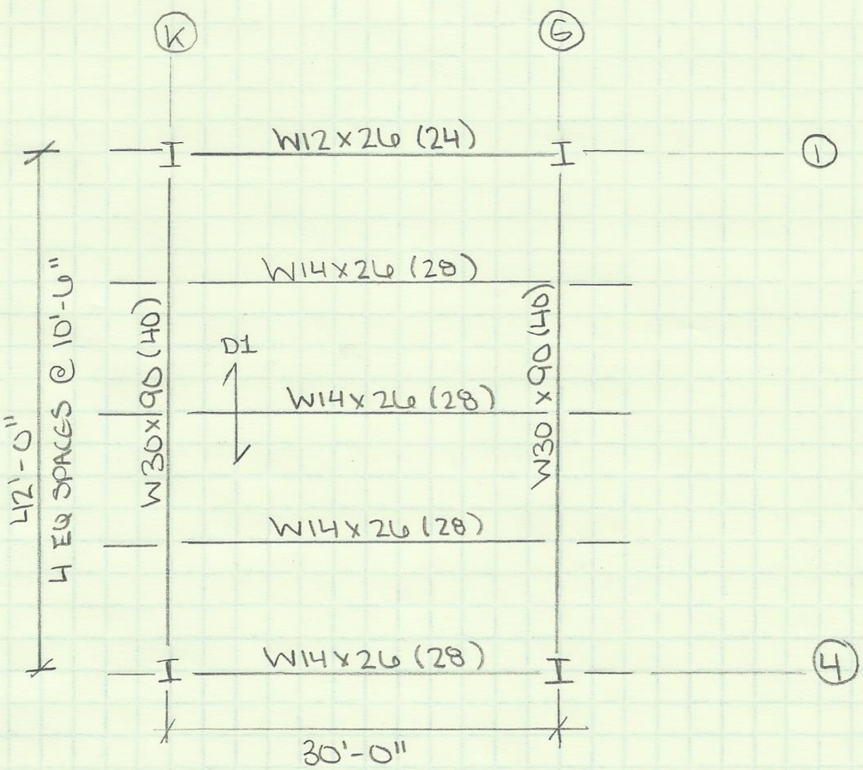
$\Delta_{LL} = \dfrac{0.05(13.8^k)(42)^3(1728)}{29000(5880 \text{ in}^4)} = 0.518''$

$\Delta_{LL,max} = \dfrac{L}{360} = \dfrac{12(42)}{360} = 1.4''$

$\Delta_{LL} < \Delta_{LL,max}$ ✓

Use W30x90 with 40 studs spaced evenly along the beam for the Girder

Steel Design # 2 Summary: Composite



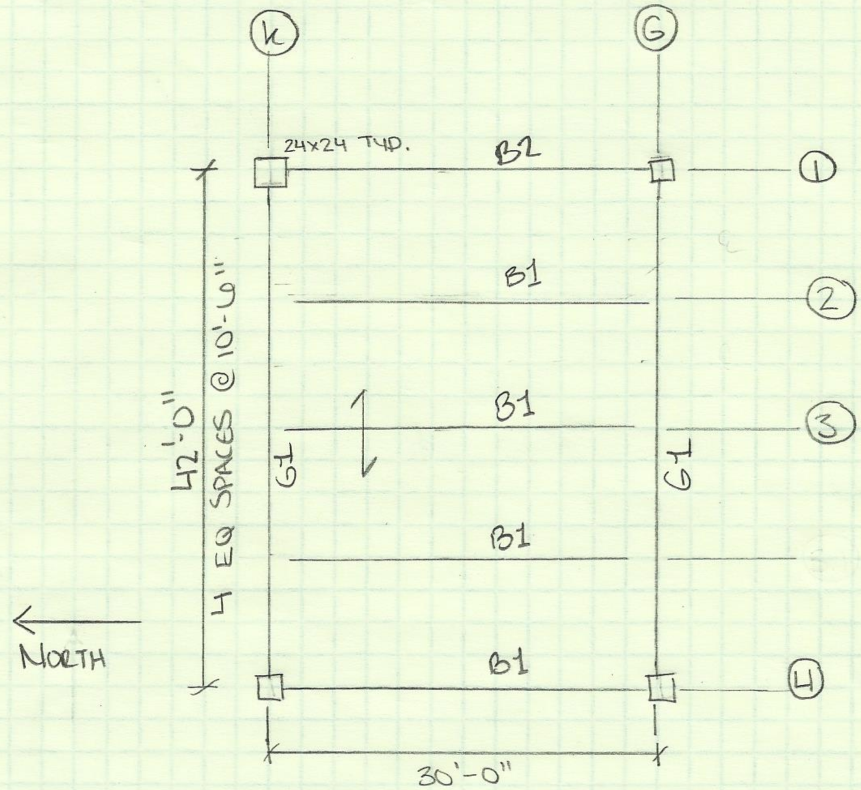

D1: 2VLI 18, 4.5 inch NW Concrete topping

# ALTERNATIVE FLOOR SYSTEM

# DESIGN #3: ONE-WAY CONCRETE SLAB

## One Way Slab With Beams - Design 3

Possible Layout: (Similar to steel design)




Determine One-Way Slab Thickness:

- ACI 318-11 Table 9.5a cannot be used because this space has partitions.
- Referenced Table A-9 on Page 1094 of Reinforced Concrete Mechanics and Design 6E by Wright and MacGregor.

Supporting partitions → One end continuous → $h_{min} = l/13$

Note: 1 end continuous used here b/c its the worst case. I will design the slab to be the same thickness for ease of calculation.

$h_{min} = \ell/13 = \frac{10.5(12)}{13} = 9.69" \rightarrow \underline{10"}$

Determine Load w/ 10" Slab:

LL = 80 PSF

DL = 150 PCF (10"/12") + 23 PSF = 148 PSF

$W_u = 1.2(148) + 1.6(80) = \underline{306 \text{ PSF}}$

Design Moments:

Looking @ 1 ft Strip ...

$M_u = \frac{W_u \ell^2}{8}$

$w_u = (306 \text{ PSF})(1 \text{ ft}) = 306 \text{ lb/ft} = 0.306 \text{ k/ft}$

* Using moment coefficients to estimate moments. $\ell$ will be used in place of $\ell_n$ to be conservative (beam sizes unknown)

$M_{u_1}^- = -\frac{W_u \ell^2}{24} = -\frac{(0.306)(10.5)^2}{24} = \underline{-1.41 \text{ k-ft}}$

$M_{u_{12}}^+ = \frac{W_u \ell^2}{14} = \underline{+2.41 \text{ k-ft}}$

$M_{u_2}^- = \max \left| \begin{array}{c} \frac{W_u \ell^2}{10} \\ \frac{W_u \ell^2}{11} \end{array} \right. = \underline{-3.37 \text{ k-ft}}$

$M_u^+, \text{midspans} = \frac{W_u \ell^2}{16} = \underline{+2.11 \text{ k-ft}}$

$M_u^-, \text{supports} = \frac{W_u \ell^2}{11} = \underline{-3.07 \text{ k-ft}}$

* The maximum negative and positive moments will be used to design the slab and reinforcement. *

$\left. \begin{array}{l} M_u^+ = 2.41 \text{ k-ft} \\ M_u^- = -3.37 \text{ k-ft} \end{array} \right\}$ use for design

Design Slab Reinforcement:

$$A_s = \frac{M_u}{4d}$$

#4 bars assumed based on CRSI Handbook

$$d = 10'' - 3/4'' - \tfrac{1}{2}(0.5'') = 9''$$

$$A_s^+ = \frac{M_u^+}{4d} = \frac{2.41\ k\text{-}ft}{4(9'')} = 0.067\ in^2/ft$$

$$A_s^- = \frac{M_u^-}{4d} = \frac{(3.37\ k\text{-}ft)}{4(9'')} = 0.094\ in^2/ft$$

As,min for temperature and shrinkage:

$$\rho_{min} = 0.0018 \quad (\text{ACI 318-11 §7.12.2.1})$$

$$\rho = \frac{A_s}{b \cdot h} \rightarrow A_s = 0.0018(12'')(10'')$$

$$A_s = 0.216\ in^2/ft$$

* this value will control reinforcement design in both directions b/c of temp. and shrink requirements. It will also control both top and bottom bars for $A_s^-$ and $A_s^+$.

Bar Spacing: (§10.6.4)

$$S_{max} = \min \left| \begin{array}{l} 15\left(\frac{40{,}000}{2/3(60{,}000)}\right) - 2.5(3/4'') = 13.125'' \\ 12\left(\frac{40{,}000}{2/3(60{,}000)}\right) = 12'' \end{array} \right.$$

$$S_{max} = 12''$$

#4 @ 11" o.c. ? $\quad$ As,provided $\rightarrow \frac{0.2\ in^2}{11''} \times 12 = 0.218\ in^2/ft$

As,provided = $0.218\ in^2/ft$ > Asmin = $0.216\ in^2/ft$ ✓

Verify Moment Capacity:

$$a = \frac{(0.218\ in^2)(60{,}000)}{0.85\,(4000)(12)} = 0.321\ in$$

$$M_n = (0.218)(60{,}000)(9'' - 0.321/2)/1000 \cdot 12 = 9.63\ ft\text{-}k$$

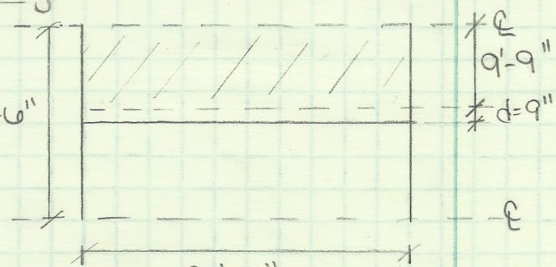
$$\phi M_n = 0.9(9.64^{'k}) = 8.67^{'k} > M_{u\,max} = 3.37^{'k}\ \checkmark$$

$\boxed{\text{USE \#4 bars @ 11" o.c. @ bottom of slab @ midspan and top of slab over supports w/ 10" slab}}$

Check One Way Shear Capacity:

Vu @ d from support:

$$V_u = (9' + 9/12)(30)(0.300) = 89.5^k$$




$$V_c = 2\lambda\sqrt{f'c}\ b_w \cdot d = 2(1.0)\sqrt{4000}\,(30 \times 12)(9'')/1000$$

$$V_c = 409.8^k$$

$$\phi V_n = 1.0\ \phi V_c = 0.75(1.0)(409.8^k) = 307.4^k$$

Slabs

$$\phi V_n = 307.4^k \geq V_u = 89.5^k\ \checkmark$$

| TYPICAL SLAB REINFORCING | | | |
|---|---|---|---|
| Slab Thickness | E-W | | N-S |
| | TOP | BOTTOM | |
| * 10" | #4@11" | #4@11" | #4 @ 11" for temp. & shrinkage |

* 4000 psi, normal weight concrete slab

## Beam Design :

### Design loads :

LL = 67.81 PSF (from Pg. 22)

DL = 148 PSF

$W_u = 1.2(148) + 1.6(67.81) = 286 \text{ PSF}$

$w_u = 0.286 \times 10.5 = 3.0 \text{ k/ft}$

### Design Moment :

l used to be conservative for preliminary sizing

$$M = \frac{w_u l^2}{8} = \frac{(0.286 \times 10.5')(30')^2}{8} = 337.8 \text{ ft-k}$$

$$M_u = 1.1(337.8^{\text{'k}}) = \underline{371.6^{\text{'k}}}$$

allowance for beam selfweight

### Estimate Beam Size:

Try $b = 4/5 d$

$b \cdot d^2 = 20 M_u$

$(4/5 d) \cdot d^2 = 20(371.6)$

$d = 21''$

$b = 17''$

$h = d + 2.5 = 21'' + 2.5'' = 23.5 \rightarrow 24''$

USE $h = 24''$ and $b = 17''$

### Check SW Effects:

$$w_{sw} = \frac{(24 \times 17)}{144} \times 150 / 1000 = 0.425 \text{ k/ft}$$

$w_u = 3.0 \text{ k/ft} + 1.2(0.425 \text{ k/ft}) = 3.51 \text{ k/ft}$

$$M_u = \frac{(3.51)(30)^2}{8} = 394.9 \text{ ft-k}$$

### Required Steel:

$$A_s = \frac{M_u}{4d} = \frac{394.9^{\text{'k}}}{4(21'')} = 4.7 \text{ in}^2 \rightarrow (6)\ \#8 \text{ bars}$$

$A_{s\,prov} = 6(0.79) = 4.75 \text{ in}^2$

actual d

Check As,min:

$$A_{s,min} = \max \begin{cases} \dfrac{3\sqrt{f'c}}{f_y} \cdot b_w \cdot d = \dfrac{3\sqrt{4000}}{60{,}000}(17)(22.75) = 1.22\,in^2 \\ \dfrac{200\, b_w \cdot d}{f_y} = \dfrac{200(17)(22.75)}{60{,}000} = 1.29\,in^2 \end{cases}$$

$$A_{s,min} = 1.29\,in^2 \le A_{s,prov} = 4.75\,in^2 \checkmark$$

Check As,max:

$$A_{s,max} = 0.85\beta_1 \frac{f'c}{f_y}\left(\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_y}\right) b_w \cdot d$$

$$= (0.85)^2 \frac{(4000)}{60000}\left(\frac{0.003}{0.003+0.004}\right)(17)(22.75)$$

$$A_{s,max} = 7.98\,in^2 \ge A_{s,prov} = 4.75\,in^2 \checkmark$$

Check $\phi M_n$: $d = 24'' - 1\frac{1}{2}'' - 0.25'' - \frac{1}{2}(1.0) = 22.75''$

$$a = \frac{(4.75)(60{,}000)}{0.85(4000)(17'')} = 4.93''$$

$$\beta_1 = 0.85 \rightarrow c = 4.93''/0.85 = 5.8$$

$$\varepsilon_s = \frac{0.003}{5.8}(22.75 - 4.93/2) = 0.010 \geq 0.00207 \checkmark$$

$$M_n = A_s f_y (d - a/2)$$
$$= (4.75)(60{,}000)(22.75 - 4.93/2)/1000 \cdot 12$$
$$= 481.8 \text{ ft-k}$$

$$\phi M_n = 0.9(481.8)$$

$$\phi M_n = 433.6'^k \geq M_u = 394.9'^k \checkmark$$

Check Min + Max # of Bars:

Using Table A.7 adapted from ACI Reference 3.8:

- 3/4" Aggregate Size
- 17" = bw
- # 8 bars

} 6 bars allowed @ bw=16" so 6 good for bw = 17"

(6) #8 bars ✓ ok for Max # of bars
-also, spacing ok by ACI 318-11 §3.3.2

Using Table A.8 adapted from ACI §10.6.4:

- 2" cover including stirrup
- 17" = bw
- # 8 bars

} 3 bar Minimum

6 bars > 3 bars ✓

Check Shear Capacity:

$V_u = (3.51 k/ft)(15' - 22.75/12) = 46.0^k$

$V_c = 2\lambda\sqrt{f'c}\, b_w \cdot d$
$= 2(1.0)\sqrt{4000}\,(17'')(22.75'')/1000$
$= \underline{48.92^k}$

$V_c = 48.92^k \angle V_u = 46.0^k \rightarrow$ Need Stirrups

Shear Strength Required by Shear. Reinf:

$V_s = 46^k/0.75 - 48.92 = \underline{12.41^k}$

$12.41^k \leq 8\sqrt{4000}\,(17)(22.75)/1000 = 196^k$ ✓

Max Shear Reinf. Spacing:

$V_s = 17.75^k \leq 4\sqrt{4000}\,(17)(22.75)/1000 = 97.8^k$ ✓

then... $S_{max} = \min \left| \begin{array}{l} d/2 = 22.75''/2 = 11.375'' \\ 24'' \end{array} \right.$

$\underline{S_{max} = 11''}$

Min. Shear Reinforcing:

$A_{v,min} = \max \left| \begin{array}{l} 0.75\sqrt{f'c}\, b_w \cdot s / f_{yt} \\ 50\, b_w \cdot s / f_{yt} \end{array} \right.$

$= \max \left| \begin{array}{l} 0.75\sqrt{4000}\,(17)(11)/60{,}000 = 0.148\ in^2 \\ 50(17)(11)/60{,}000 = 0.156\ in^2 \end{array} \right.$

$\underline{A_{v,min} = 0.156\ in^2} \rightarrow \underline{2 \text{ legs of } \#3}$
$A_v = 0.22$

Design of Shear Reinf:

$S = A_v \cdot f_{yt} \cdot d / V_s = (0.22)(60{,}000)(22.75) / 17.75^k \cdot 1000$
$= 16.9'' \geq S_{max} = 11''$

* Use 11" for stirrup spacing *
for ⊔ #3.

Distance where $\phi V_n$ occurs:

$$W_u = 3.51\ ^{k}/ft \rightarrow V_{max} = 52.65^{k}$$

$$0.5\phi V_c = 0.5(0.75)(48.92) = 18.35^{k}$$

$$18.35 = 52.65 - 3.51(\ell_v)$$

$$\ell_v = 9.77'$$

$$2" + (n-1)(11") \geq 9.77(12)$$

$$11n - 11 \geq 115.24$$

$$n \geq 11.5$$

$n = 12$ Stirrups

$\boxed{(12)\ \#3 \times \sqcup\ @\ 11" \text{ from 2" away from face of each support}}$

<u>Beam Design Summary</u>: B1

| |
|---|
| 17" x 24" NW CONC., 4000 PSI |
| (6) #8 longitudinal, bottom bars |
| (12) #3 x $\sqcup$ @ 11" @ 2" away from the face of each support |

GIRDER DESIGN:

Design loads:

$LL = 43.92$ PSF

JOIST SW

$W_{u,JOIST} = 10.5'(1.2(148)+1.6(43.92))/1000 + 1.2(0.425)$

$W_{u,joist} = 3.11$ k/ft

$P_u = (3.11^k/ft)(30\ ft) = 93.3^k$

Design Moment:

$M_u = 0.5PL = 0.5(93.3^k)(42')$

$M_u = 1959$ ft-k

$M_u = 1.1(1959^{'k}) = \underline{2155^{'k}}$

Estimate Beam Size:

Try $b = 4/5 d$

$(4/5 d)d^2 = 20(2155) \rightarrow 38'' = d$

$h = 38 + 2.5 = 40.5 \rightarrow 41''$

$b = 4/5(41'') = 32.8'' \rightarrow 33''$

* Use $h = 41''$ and $b = 33''$

Check SW Effects:

$W_{sw} = \frac{(41 \times 33)}{144}(150)/1000 = 1.41$ k/ft

$W_{u,sw} = 1.2(1.41) = 1.69$ k/ft

$M_{u,sw} = \frac{(1.69)(42)^2}{8} = 372.6$ ft-k

$M_u = 1959^{'k} + 372.6^{'k} = 2332^{'k}$

Required Steel:

$A_s = \frac{2332}{4(38)} = 15.34\ in^2 \rightarrow$ (10) #11 bars

$A_{s,prov} = (10)(1.56) = 15.6\ in^2$

Check As,min:

actual d

$$A_{s,min} = \max \begin{cases} \dfrac{3\sqrt{4000}}{60{,}000}(33)(38.545) = 4.02 \text{ in}^2 \\ \dfrac{200(33)(38.545)}{60{,}000} = 4.24 \text{ in}^2 \end{cases}$$

$$A_{s,min} = 4.24 \text{ in}^2 \leq A_{s,prov} = 15.6 \text{ in}^2 \checkmark$$

Check As,max:

$$A_{s,max} = 0.85(0.85)\frac{4000}{60{,}000}\left(\frac{0.003}{0.003+0.004}\right)(33)(38.545)$$

$$A_{s,max} = 26.26 \text{ in}^2 \geq A_{s,prov} = 15.6 \text{ in}^2 \checkmark$$

Check $\phi M_n$: $d = 41'' - 1\tfrac{1}{2}'' - 0.25'' - \tfrac{1}{2}(1.41) = 38.545''$

$$a = \frac{(15.6)(60{,}000)}{0.85(4000)(33)} = 8.34 \text{ in}$$

$$\beta_1 = 0.85 \rightarrow c = a/\beta_1 = 8.34/0.85 = 9.81''$$

$$\varepsilon_s = \frac{0.003}{9.81}(38.545 - 8.34/2) = 0.0105 \geq 0.00207 \checkmark$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$= (0.9)(15.6)(60{,}000)(38.545 - 8.34/2)/12{,}000$$
$$= 2413 \text{ ft-k}$$

$$\phi M_n = 2413'^k \geq M_u = 2332'^k \checkmark$$

Check Min & Max bars:

Bar Spacing / Max # bars: Table A.7

- 3/4" Aggregate
- 33" = $b_w$
- #11 bars

in 30" beam, 9 allowed so additional 3" for additional bar should be adequate

Bar Spacing:

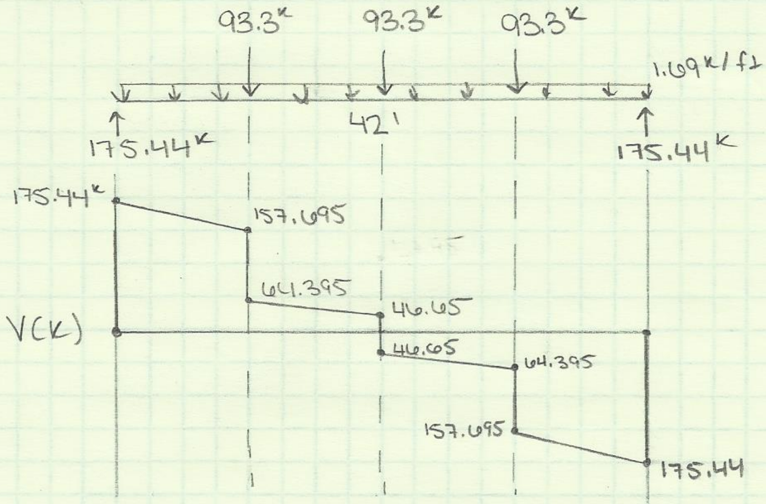
$$S_{min} = \max \begin{cases} d_b = 1.41'' \\ 1'' \\ 4/3\, S_a = 4/3(3/4) = 1'' \end{cases}$$

$$S_{min} = 1.41'' \times 2 = 2.82'' < 3'' \checkmark$$

Minimum # of bars: Table A.8

4 bars < 10 bars ✓

## Check Shear Capacity:




$V_u @ d = 175.44^k - 1.69(38.545)/12 = \underline{170.0\ k}$

$V_c = 2(1.0)\sqrt{4000}(33'')(38.545'')/1000 = \underline{160.9^k}$

$\phi V_n = 0.5(0.75)(160.9^k) = 60.3^k < V_u \rightarrow$ need stirrups

$V_s = 170/0.75 - 160.9^k = \underline{65.8^k} \leq 8\sqrt{4000}(33)(38.455)/1000 = 642\ k$ ✓

$V_s \leq \frac{1}{2}(642^k) = 321^k > 65.8^k$ ✓

then... $S_{max} = \min \left| \begin{array}{l} d/2 = 38.545/2 = 19.3'' \\ 24'' \end{array} \right.$

$\underline{S_{max} = 19''}$

Min Shear Reinf:

$A_{v,min} = \max \left| \begin{array}{l} 0.75\sqrt{4000}(33)(19)/60{,}000 = 0.496\ in^2 \\ 50(33)(19)/60{,}000 = 0.523\ in^2 \end{array} \right.$

$A_{v,min} = 0.523\ in^2 \rightarrow$ 3 legs #4
$A_v = 0.6\ in^2$

Design Reinf:

$S = (0.6)(60{,}000)(38.545)/65.8 \cdot 1000 = 21.1\ in > S_{max} = 19''$

* USE 19" spacing of #4 *

Distance where $\phi V_n$ occurs:

$\phi V_n = 60.3^k$

$60.3 = 64.395 - 1.69(l_v)$

$l_v = 2.42\ ft \rightarrow +10.5 = 12.92\ ft$ (from bm end)

$2" + (n-1)(19") \geq 12.92\ (12)$

$n \geq 9.1$

$n = 10$ stirrups

<u>(10) #4 x @ 19"</u>

GIRDER <u>DESIGN Summary</u>: G1

| |
|---|
| 33" x 41" NW CONC., 4000 PSI |
| (10) #11 longitudinal, bottom bars |
| (10) #4 x @ 19" @ 2" away from support |

## Edge Beam Design: B2

Design loads:

$L = 80$ PSF (no reduction)

$DL = 148$ PSF

$DL_{wall} = 118$ PLF

$W_u = 1.2(148) + 1.6(80) = 305.6$ PSF

$W_u = (305.6 \text{ PSF})(10.5'/2) + 1.2(118 \text{ Plf})$
$= 1746$ lb/ft
$W_u = \underline{1.75 \text{ k/ft}}$

Design Moment:

$M = \frac{w_u l^2}{8} = \frac{(1.75 \text{ lb/ft})(30 \text{ ft})^2}{8} = 196.9$ ft-k

$M_u = 1.1(196.9'^k) = \underline{216.59'^k}$

Estimate Beam Size:

Try $b = 4/5 d$

$b \cdot d^2 = 20 M_u$

$(4/5) d^3 = 20(216.59)$

$d = 18"$
$b = 15"$

$h = d + 2.5 = 18" + 2.5" = 20.5 \rightarrow 21"$

use $h = 21"$ and $b = 15"$

Check SW Effects:

$w_{sw} = \frac{(21 \times 15)}{144} \times 150/1000 = 0.328$ k/ft

$w_u = 1.75$ k/ft $+ 1.2(0.328$ k/ft$) = 2.14$ k/ft

$M_u = \frac{(2.14 \text{ k/ft})(30^2)}{8} = \underline{241.2 \text{ ft-k}}$

Required Steel :

$$A_s = \frac{M_u}{4d} = \frac{241.2^{\,1k}}{4(18'')} = 3.35\ in^2 \rightarrow (5)\ \#8\ \text{bars}$$

$A_{s,provided} = 3.95\ in^2$

Check $\phi M_n$ : $d = 21'' - 1.5'' - 0.5'' - \frac{1}{2}(1.0) = 18.5''$

$$a = \frac{(3.95)(60{,}000)}{0.85(4000)(15'')} = 4.65''$$

$$\beta_1 = 0.85 \rightarrow c = 4.65''/0.85 = 5.47''$$

$$\varepsilon_s = \frac{0.003}{5.47}\left(18.5 - \frac{4.65}{2}\right) = 0.0089 > 0.00207 \checkmark$$

$$M_n = A_s f_y (d - a/2)$$
$$= (3.95)(60{,}000)(18.5 - 4.65/2)/1000 \cdot 12$$
$$= 319.5\ ft\text{-}k$$

$$\phi M_n = 0.9(319.5^{1k}) = 287.5\ ft\text{-}k$$

$$\phi M_n = 287.5^{1k} \geq M_u = 241.2^{1k} \checkmark$$

Check Min & Max # of Bars:

Table A.7 : Max Bars

- 3/4" Aggregate
- 15" = bw
- # 8

} 5 bars allowed for bw = 14" and 6 bars allowed for bw = 16"...

so... (5) #8 bars ✓ ok for max # of bars and spacing by ACI 318-11 § 3.3.2

Table A.8 : Min bars

- 2" cover with stirrup
- bw = 15"
- # 8 bars

} 2-3 bars minimum

5 bars > 3 bars ✓

Check As,min:

$$A_{s,min} = \max \begin{cases} \dfrac{3\sqrt{4000}}{60{,}000}(15)(18.5) = 0.878 \text{ in}^2 \\ \dfrac{200(15)(18.5)}{60{,}000} = 0.925 \text{ in}^2 \end{cases}$$

$$A_{s,min} = 0.925 \text{ in}^2 \leq A_{s,prov} = 3.95 \text{ in}^2 \checkmark$$

Check As,max:

$$A_{s,max} = 0.85(0.85)\frac{(4000)}{60{,}000}\left(\frac{0.003}{0.003+0.004}\right)(15)(18.5)$$

$$A_{s,max} = 5.73 \text{ in}^2 \geq A_{s,prov} = 3.95 \text{ in}^2 \checkmark$$

Check Shear Capacity:

$V_u = (2.14\,k/ft)(15' - 18.5/12) = 28.8^k$

$V_c = 2\lambda\sqrt{f'_c}\,b_w \cdot d$
$= 2(1.0)\sqrt{4000}(15'')(18.5'')/1000$
$= 35.10$

$\phi V_n = (0.5)(0.75)(35.10) = 13.16^k$

$\phi V_n = 13.16^k < V_u = 28.8^k$

Shear Strength Required by Shear Reinf:

$V_s = \frac{28.8}{0.75} - 35.10 = \underline{3.3^k}$

$3.3^k \leq 8\sqrt{4000}(15)(18.5)/1000 = 140^k$ ✓

Max spacing of shear reinforcing:

$V_s = 3.3^k \leq 4\sqrt{4000}(15)(18.5)/1000 = 70^k$ ✓

then... $S_{max} = \min \begin{cases} d/2 = 18.5/2 = 9.25'' \\ 24'' \end{cases}$

$\underline{S_{max} = 9''}$

Min Shear Reinforcing:

$A_{v,min} = \max \begin{cases} 0.75\sqrt{4000}(15)(9)/60{,}000 = 0.107\,in^2 \\ 50(15)(9)/60{,}000 = 0.1125\,in^2 \end{cases}$

$A_{v,min} = 0.1125\,in^2 \rightarrow$ 2 legs of #3

$\underline{A_v = 0.22\,in^2}$

Design of Shear Reinforcement:

$S = A_v \cdot f_{yt} \cdot d/V_s = (0.22)(60{,}000)(18.5)/3.3 \cdot 1000$
$= 74'' \geq S_{max} = 9''$

* use 9" for stirrup spacing for ⊔ #3

Distance where $\phi V_n$ occurs:

$$V_{max} = (2.14\ ^{k}/_{ft})(30')/2 = 32.1^{k}$$

$$0.5\phi V_c = 13.16^{k}$$

$$13.16 = 32.1 - 2.14\ \ell v$$

$$\ell v = 8.85\ ft$$

$$2'' + (n-1)(9'') \geq 8.85(12)$$

$$(n-1)(9) \geq 104.2$$

$$(n-1) \geq 11.578$$

$$n \geq 12.58$$

$$n = 13 \text{ stirrups}$$

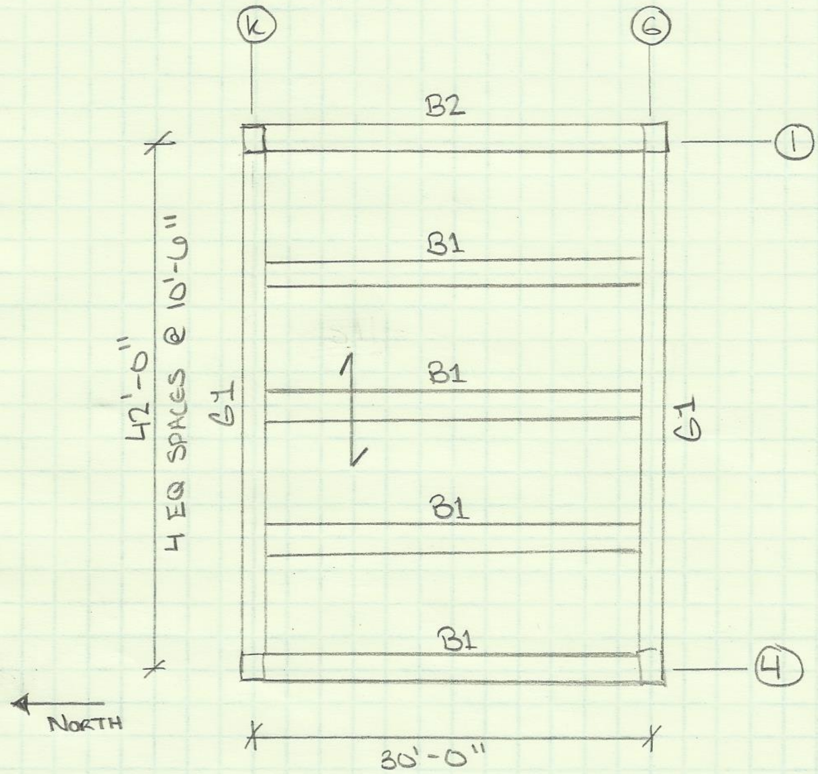
| |
|---|
| (13) #3 × @ 9" from 2" away from face of each support |

Beam Design Summary: B2

| |
|---|
| 15" × 21" NW, 4000 PSI |
| (5) # 8 longitudinal, bottom bars |
| (13) #3 × @ 9" @ 2" away from face of each support |

# One way Concrete Slab w/ Beams

## Design # 3 Summary:




| Slab Schedule | | | |
|---|---|---|---|
| Thickness | E-W | | N-S |
| | Top | Bottom | |
| 10" | #4@11" | #4@11" | #4@11" for temperature and shrinkage |

**Notes:**

1. 4000 PSI Normal Weight concrete
2. Rebar $f_y = 60{,}000$ psi and $f_{yt} = 60{,}000$ psi

| BEAM SCHEDULE | | | |
|---|---|---|---|
| Beam Label | Size, bxh | Longitudinal Bottom Reinf. | Shear Reinforcement |
| B1 | 17"x24" | (6) #8 | #3 @ 11" |
| B2 | 15"x21" | (5) #8 | #3 @ 9" |
| G1 | 33"x41" | (10) #11 | #4 @ 19" |

# COST ANALYSIS OF SYSTEMS

## COST ANALYSIS :

- The following costs were found using 2013 R.S. Means Square Foot cost data for the appropriate assembly. Details of each assembly will be given
- Cost Multiplier for San Diego, CA → 1.03 for commercial construction

FLOOR Design #1: Non-Composite Steel:

R.S. Means : B1010 254

Includes : Steel beams
Steel girders
Composite Steel deck
Concrete Slab w/ WWF

used for interpolation
Bay Size : 42' x 30'
Total Load : 80 PSF + 103 PSF = 183 PSF

Superimposed load : 80 PSF (LIVE) + 23 PSF (SUPERIMPOSED dead) = 103 → 125 PSF

| Bay Size | SUPERIMPOSED LOAD (PSF) | TOTAL (PSF) LOAD | TOTAL COST/SF |
|---|---|---|---|
| 30 x 30 | 125 | 182 | 32.40 |
| 30 x 35 | 125 | 183 | 34.35 |
| 30 x 42 | 125 | 183 | ? |

Interpolate Cost Based on bay length :

$$\frac{42-30}{X-32.4} = \frac{35-30}{34.35-32.4} \rightarrow X = 37.08 \times \underset{\text{location multiplier}}{1.03}$$

$$30 \times 42\text{, Total load} = 183\text{ PSF} \rightarrow \boxed{\$38.19/SF}$$

FLOOR DESIGN #2: Composite Steel

R.S. Means: B1010 256

Includes: Composite beams
Welded shear studs
Composite deck
Concrete Slab w/ WWF

used for interpolation
Bay Size: 42' x 30'
Total Load: 183 PSF

Superimposed Load: 103 PSF → 125 PSF
(round up for table)

| Bay Size | Superimposed Load (PSF) | TOTAL LOAD (PSF) | TOTAL COST / SF | TOTAL COST PER SF @ 183 PSF |
|---|---|---|---|---|
| 30 x 30 | — | 168 | 22.75 | 23.51 |
| | | 252 | 27 | |
| 30 x 35 | — | 169 | 23.40 | 24.03 |
| | | 254 | 27.20 | |
| 30 x 42 | — | 183 | — | ? |

$$\frac{27 - x}{252 - 183} = \frac{27 - 22.75}{252 - 168} \rightarrow x = 23.51$$

$$\frac{27.2 - x}{254 - 183} = \frac{27.2 - 23.4}{254 - 169} \rightarrow x = 24.03$$

location multiplier

$$\frac{42 - 30}{x - 23.51} = \frac{35 - 30}{24.03 - 23.51} \rightarrow x = 24.76 \times 1.03$$

30 x 42, TOTAL LOAD = 183 PSF → $\boxed{\$25.50 / SF}$

FLOOR DESIGN #3: Beams and One-way slab

R.S. Means : B1010 219

Includes: Concrete beam
Rebar
Concrete slab
Rebar
Monolithically Poured system

Total Load: 148+80 = 228 PSF

Slab Thickness: 10 inches

| Slab Thickness | Total Load (PSF) | Total Cost / SF | TOTAL COST/SF for TL = 216 PSF |
|---|---|---|---|
| 8" | 196 | 19.65 | 20.89 |
| | 254 | 21.90 | |
| 9" | 213 | 21.70 | 22.04 |
| | 272 | 23.05 | |
| 10" | 228 | — | ? |

$$\frac{21.90 - x}{254 - 228} = \frac{21.90 - 19.65}{254 - 196} \rightarrow x = 20.89$$

$$\frac{23.05 - x}{272 - 228} = \frac{23.05 - 21.70}{272 - 213} \rightarrow x = 22.04$$

$$\frac{10 - 8}{x - 20.89} = \frac{9 - 8}{22.04 - 20.89} \rightarrow x = \$23.19/SF \times 1.03$$

10" slab, TOTAL LOAD = 216 PSF → $\boxed{\$23.89/SF}$

## Original Design: Two-Way Flat Plate

R.S. Means: B1010 222

Includes: flat plate, uniform slab
rebar
drop panels*

* although the current system doesn't use drop panels, no flat-plate system exists in R.S. means. Also, my system has a substantial edge beam that will be somewhat compensated for with the drop panel assumption.

Total Load: 200 PSF + 80 PSF = 280 PSF

Slab Thickness: 14"

| Slab Thickness | TOTAL LOAD (PSF) | Total Cost Per SF | Total Cost/ PSF @ 280 PSF |
|---|---|---|---|
| 11.5" | 231 | 19.05 | 19.65 |
| | 284 | 19.70 | |
| 12" | 240 | 19.50 | 19.90 |
| | 290 | 20.00 | |
| 14" | 280 | — | ? |

$$\frac{19.7 - x}{284-280} = \frac{19.7-19.05}{284-231} \rightarrow x = 19.65$$

$$\frac{20.0 - x}{290-280} = \frac{20.-19.5}{290-240} \rightarrow x = 19.9$$

location multiplier

$$\frac{14-12}{x-19.90} = \frac{12-11.5}{19.9-19.65} \rightarrow x = 20.90 \times 1.03$$

14" flat plate slab, TOTAL LOAD = 280 PSF

$\boxed{\$21.53/SF}$

# SYSTEM WEIGHT CALCULATIONS

## Floor Design #1 : Non-Composite Steel

Beam & Girder weights:

$$W = (30')(26\,^{lb}/_{ft}) + 4(30')(40\,^{lb}/_{ft}) + 2(42')(108\,^{lb}/_{ft})$$

$$= 14652 \text{ lb}$$

Slab and Deck :

$$W = 69 \text{ PSF}$$

Total Weight :

$$W_T = \frac{14652 \text{ lb}}{(42' \times 30')} + 69 \text{ PSF}$$

$$\boxed{W_{TOTAL} = 80.6 \text{ PSF}}$$

## Floor Design #2: Composite Steel

Beam & Girder Weights:

$$W = 5(30')(26\,^{lb}/_{ft}) + 2(42')(90\,^{lb}/_{ft})$$

$$= 11460\ lb$$

Stud Weights:

$$W = (10\ lb)(40\times 2 + 28\times 4 + 24)$$

$$= 2160\ lb$$

Slab & Deck:

$$W = 69\ PSF$$

TOTAT WEIGHT:

$$W_{Total} = \frac{11460\ lb + 2160\ lb}{(30'\times 42')} + 69\ PSF$$

$$\boxed{W_{Total} = 79.8\ PSF}$$

## FLOOR Design #3: One-Way Slab on beams normal weight concrete

Beams & Girders:

$$W_{b1} = 4\times\frac{(17''\times 24'')}{144}(30' - 33''/12)(150) = 46325 \text{ lb}$$

$$W_{b2} = \frac{(15''\times 21'')}{144}(30' - 33''/12)(150) = 3941.4 \text{ lb}$$

$$W_g = 2\times\frac{(33''\times 41'')}{144}(42')(150) = 118387.5 \text{ lb}$$

Slab:

$$W_s = 150 \text{ PCF } (10''/12)\left(30'\times 42' - \frac{17''\times 30'}{12}(4) - \frac{15''\times 30}{12} - 2\times\frac{33''}{12}\times 42\right)$$

$$= 102687.5 \text{ lb}$$

Total weight:

$$W_{total} = 46325 \text{ lb} + 3941.4 \text{ lb} + 118387.5 \text{ lb} + 102687.5 \text{ lb}$$

$$= \frac{271341.4 \text{ lb}}{(42\times 30)}$$

$$\boxed{W_{total} = 215.4 \text{ PSF}}$$

Original Design : Flat Plate Slab

Slab + Edge Beam:

$$W_s = \left(14''/12\right)(150\ \text{PCF})(42' \times 30') + \left(4''/12\right)(30' \times 4.5')(150)$$

$$= 227250\ \text{lb}$$

$$W_{Total} = \frac{227250\ \text{lb}}{(42 \times 30)}$$

$$\boxed{W_{Total} = 180.4\ \text{PSF}}$$

# OVERALL SYSTEM COMPARISIONS

**Alyssa Stangl**
**Technical Report 3**
**Gravity System Comparisons**

| | | Gravity Floor Systems | | | |
|---|---|---|---|---|---|
| **Considerations** | | **Flat-Plate Concrete Slab** | **Non-Composite Steel** | **Composite Steel** | **One-Way Concrete Slab** |
| **Architectural Considerations** | | | | | |
| Maximum Depth | | 18" | 36.5" | 36.5" | 41" |
| Fire Protection Required? | | No | Yes | Yes | No |
| 2 hr Fire Rating achieved? | | Yes | Yes | Yes | Yes |
| **System Statistics** | | | | | |
| Cost Per Square Foot | | $21.53/SF | $38.19/SF | $25.50/SF | $23.89/SF |
| System Weight | | 180.4 PSF | 80.6 PSF | 79.8 PSF | 215.4 PSF |
| Are vibrations major concern? | | No | Yes | Yes | No |
| Durability | | Acceptable | Acceptable | Acceptable | Acceptable |
| **Future Design Considerations** | | | | | |
| Lateral System Options | Reinforced Concrete Shear Walls | Yes | Yes | Yes | Yes |
| | Steel Moment Frame | No | Yes | Yes | No |
| | Steel Braced Frame | No | No | No | No |
| | Concrete Moment Frame | No | No | No | Yes |
| Advantages | | Maximum floor to ceiling heights, cheapest system, no fire protection required | Light system weight, several options for lateral system | Lightest system weight, several options for lateral system, system used in LJC Tower I | Same material as existing system, cheapest of alternative systems |
| Disadvantages | | None | Most expensive system, fire protection required, large beam depth, vibrations | Higher cost than concrete systems, large beam depth, fire protection required | Heaviest system, large beam depth will significantly decrease floor to ceiling height |
| **Viable Option?** | | **N/A** | **Yes** | **Yes** | **No** |