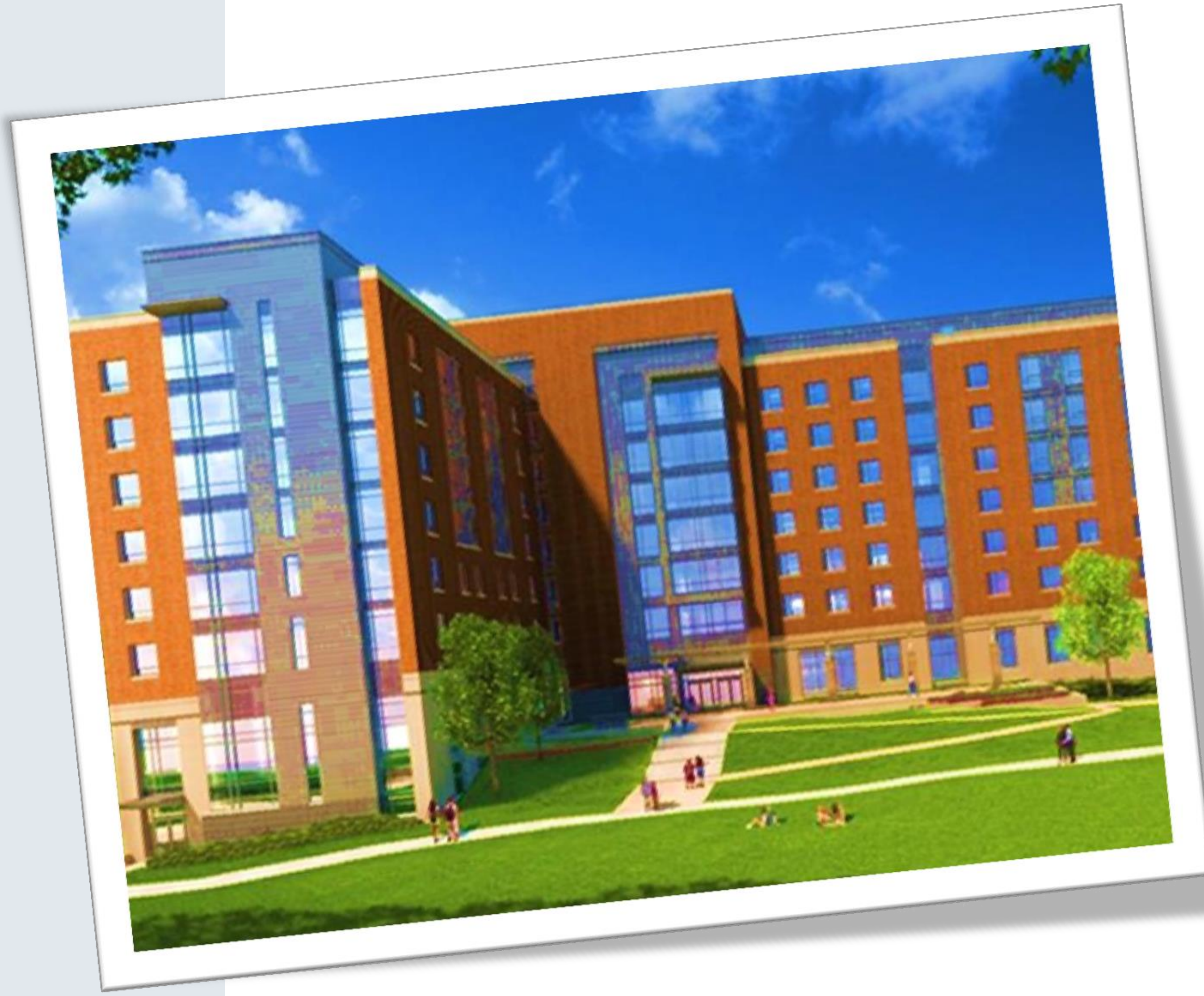
TYPICAL MEMBER SPOT CHECKS FOR GRAVITY LOADS AND ALTERNATE SYSTEMS TYPICAL BAY DESIGN STUDY



Prince Frederick Hall

The University of Maryland

College Park, MD

Christopher Cioffi

AE Senior Thesis- Structural

Advisor: Heather Sustersic

October 18, 2013

Table of Contents

The Pennsylvania State University

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Heather Sustersic,

The following is the technical three submittal of my structural thesis. It contains spot checks for gravity loads, and three different alternate gravity systems. It shows spot analysis of the existing two way slab and gravity checks for a typical interior and exterior column. Also this tech report shows the design of a one way slab, post tensioned two way slab and a steel composite deck for a typical bay. I hope that you enjoy reading my findings and I look forward to hearing back from you.

Thank you,

Sincerely,

Christopher Cioffi

PRINCE FREDERICK HALL

College Park, Maryland 20742

General Information

- Occupancy: Multi-use Dormitory
- Construction dates: May 2012- Aug 2014
- Approximate Size: 185,500sf
- Stories: 7 Stories, 1 underground

Christopher Cioffi Structural Option

Primary Project Team

- Owner: The University of Maryland
- Architect: WDG Architecture PLLC
- Contractor: Clark Construction LLC
- Civil Engineer: Site Resources Inc.
- Structural Engineer: Cagley & Associates Inc.

Structure

- Construction material: Cast in place concrete
- Slab System: 2-way slab with typical depth 9 inches
- Lateral System: 7 Concrete shear walls
- Columns: Multi- sized square concrete columns, rebar reinforced.
- Foundation: Mix of spread footers and strip footers.

MEP Systems

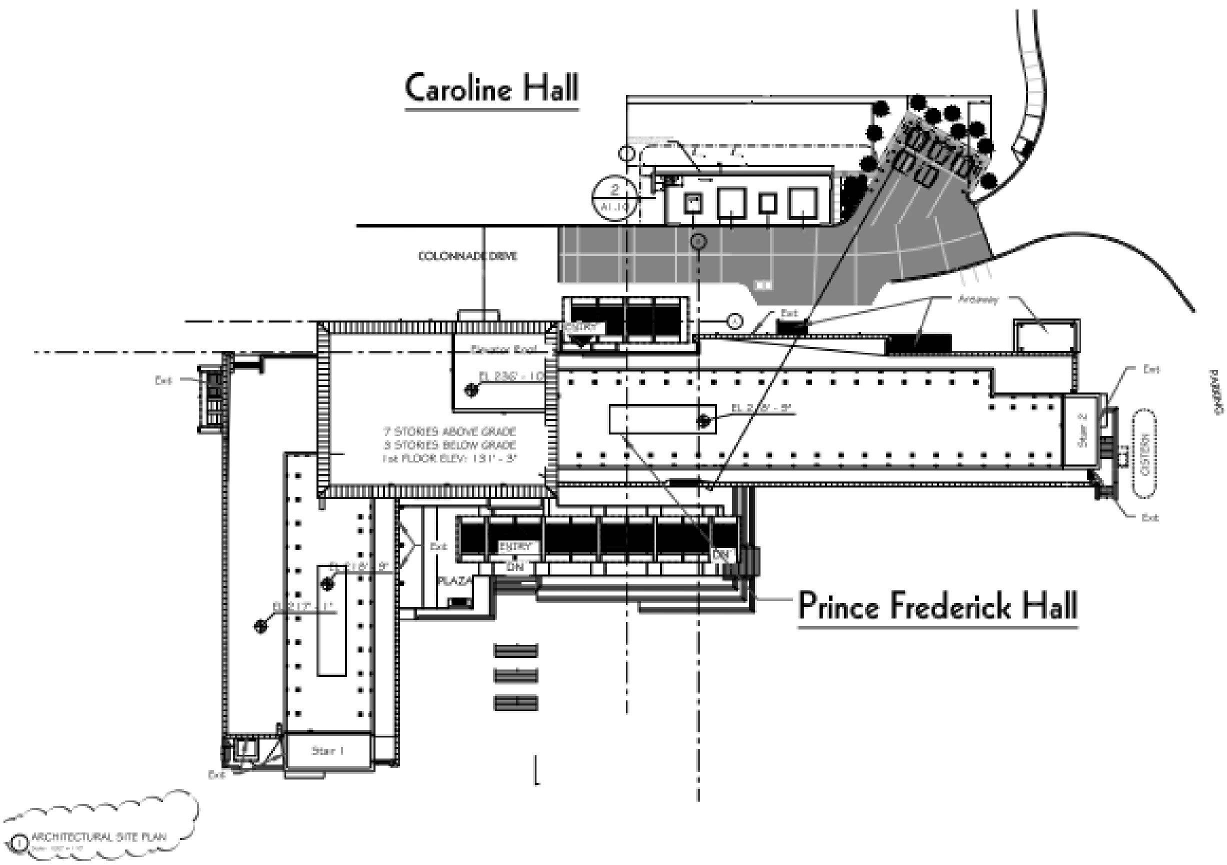
Mechanical: There are 2 roof top units and 6 air handling units. The 6 air handling units run with economizers and distribute to the VAV system for ventilation. There are also 2 main chillers for the building.
Lighting: Lighting for public corridor spaces is controlled by occupancy sensors to reduce power consumption. Outdoor lighting system turns on at dusk to conserve as well. Typical lighting in the dorm rooms are fluorescent ceiling fixtures controlled by the occupant.
Fire Suppression: The whole building is protected with fully-automatic sprinkler standpipes. Dry stand pipes are used in areas where it can become below forty degrees Fahrenheit.

CPEP SITE: http://www.engr.psu.edu/ae/thesis/portfolios/2014/cjc5333/index.html

Executive Summary

The purpose of this technical report is to establish an understanding of Prince Frederick Hall's structural and existing features. Prince Frederick Hall is nestled at the heart of The University of Maryland's campus and is a multi-use dormitory building consisting of living and office spaces. This document provides an overview of all the structural components designed by Cagley & Associates Inc. including general floor framing, structural slabs, shear wall, and the foundation system. Integration of all structural components is explained and elaborated upon. The pictures and images (unless otherwise noted) are the property of The University of Maryland and WDG Architecture PLLC and are being used solely for educational purposes.

Site Plan and Location of Building



LIST OF DOCUMENTS USED

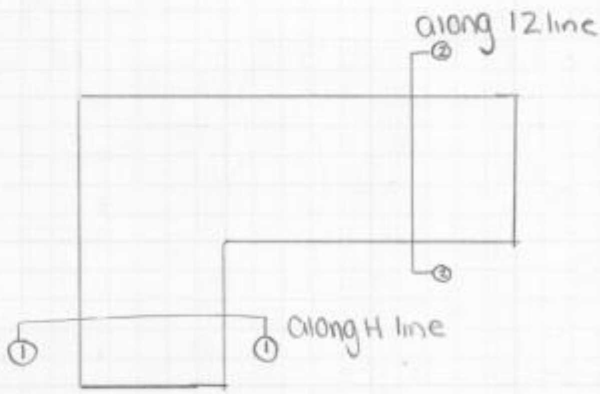
Documents Used for Analysis and Design

- ASCE 7-08
- ASCE 7-10
- ACI 318-08
- ACI 318-11
- AISC 14th Edition
- PCA Design Aids
- Reinforced Concrete Mechanics and Design, Sixth Edition. Wight

GRAVITY LOAD calculations

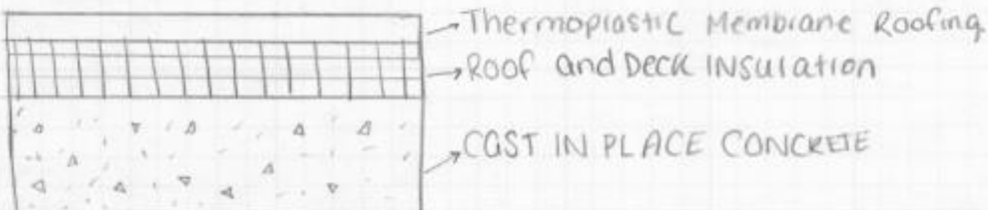
Roof Design (LOADS)

see page B of APPENDIX for detailed roof plan
C for section cuts
D for roof materials



ROOF PLAN OUTLINE

TYPICAL CUT 1.



deadload of roofing (AISC 4th edition pg 17-26

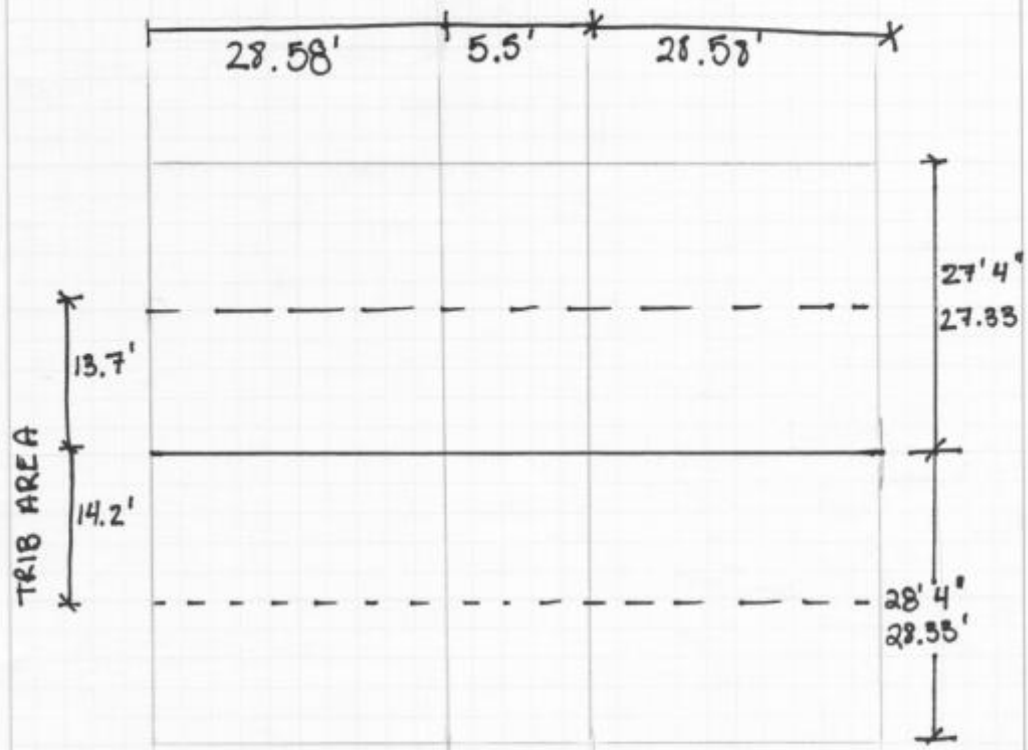
material	weight psf
plastic Membrane	1
deck insulation	2
Mechanical	8 (MEP from ceiling below)

≈ 10 psf

CHRISTOPHER CIOFFI | GRAVITY LOADS | structural thesis | 9/27

dead load roof = 10 psf [STRUCTURAL NOTES S.1.00]

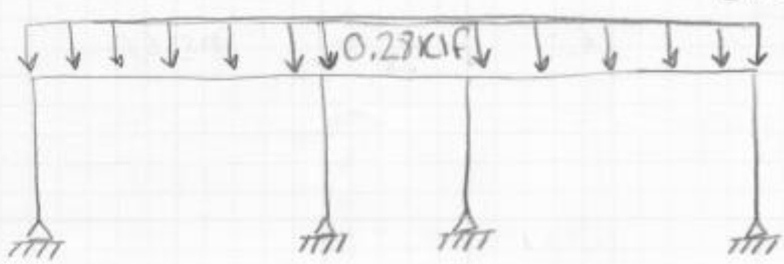
plan view



in feet

$(13.7 + 14.2)(10\,psf) = 279\,\frac{ft\,lb}{ft^2} = 279\,plf$

$= 0.28\,klf$



20

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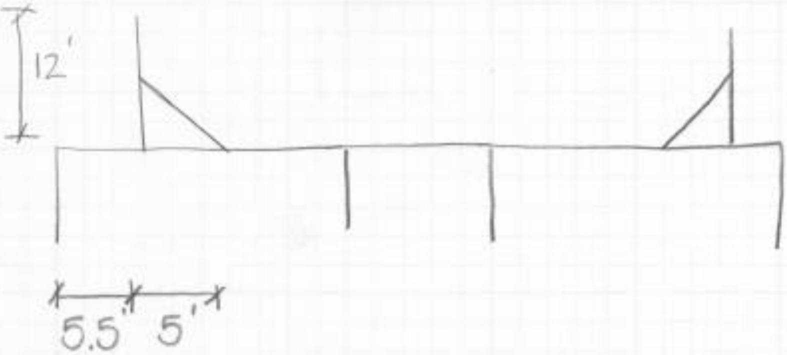
Roof Screen Dead Load Approximation:

see appendix E for roof screen details

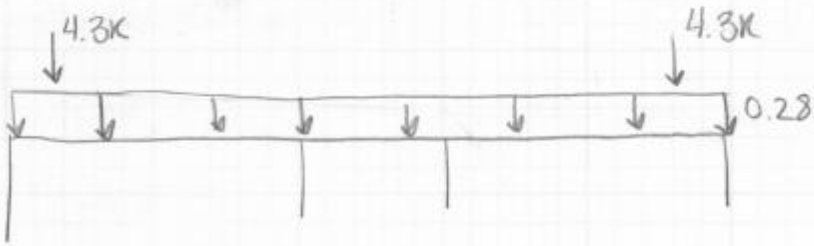


metal decking is about 3psf

$(3psf)(12ft) = 36plf = 0.36klf$

each support is about 6' apart.

$(0.36)(1\ \ ) = 4.3K$



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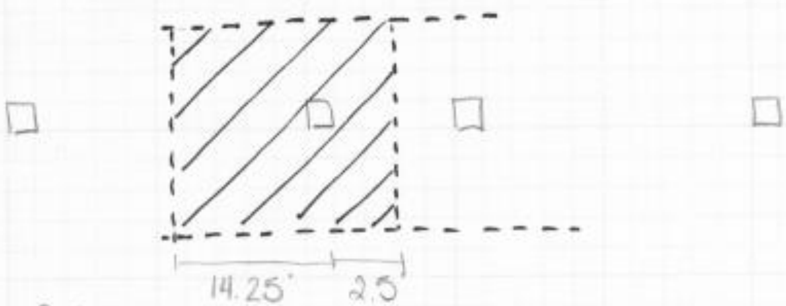
<u>live load Design</u>:

live load on roof (ASCE 7-05 § 4.9.1)

max loads

$L_r = L_o(R_1)(R_2)$ where $12 \leq L_r \leq 20$

L_o - table 4-1 L_o = 20psf (flat roof)



either of these

A_T → max trib area for a column on line H

$A_T = (27.9)(17) = 474.3 ft^2$

see Appendix E for R1 equation

$R_1 = 1.2 - 0.001A_T$

$R_1 = 1.2 - 0.001(474.3)$

$R_1 = 0.73$

F is # of inches of rise per foot.

8" min slab deck to 14" max slab deck at a typ span goes for 14.35'

$\frac{rise}{span} = \frac{0.5'}{14.35'} \times 32 = 1.1$

$R_2 = 1.0$ F ≤ 4.0 due to minimal slope

$L_r = 20(0.73)(1.0)$

$L_r = 14.6$ psf

L_r + 10psf for workers = 24.6psf

22

CHRISTOPHER CIOFFI	gravity loads		9/27

IN STRUCTURAL NOTES S1.00 Roof live load
-30psf min (ponding or snow load is used when greater than 30 psf)

-design with 30psf

$(13.7 + 14.2)(30psf) = 837psf$

feet

$= 0.837 klf$

23

Roof Snow loads:

$\tan\theta = \frac{0.5}{14.4}$

$\theta < 5° \rightarrow$ flat roof snow load.

→ Found that roof is flat from S1.00 of structural notes
See appendix F.

snow load P_f (calculated in psf).

$$P_f = 0.7 C_e C_t I P_g \quad (eq\ 7\text{-}1)$$

see appendix F for the following equations/ ASCE 7 tables.

P_g - ground snow load (table 7-1 ASCE 7) = 30

$P_g > 20\ lb/ft^2 \longrightarrow P_f = 20I$

C_e - exposure factor (table 7-2 ASCE 7) = 1.0

C_t - thermal factor (table 7-3 ASCE 7).
$C_t = 1.0$

I - importance factor (table 7-4 ASCE 7) = 1.0

Balanced load Assumption

$P_f = 0.7 C_e C_t I P_g$

$P_f = 0.7(1.0)(1.0)(1.0)(30)$

$P_f = 21\ lbs/ft^2$

min value
$P_f = 20(I)$
$= 20(1.0)$
$= 20\ lbs/ft^2$

snow load sect 1 = (13.7 + 14.2)(21) = 0.59 klf

24

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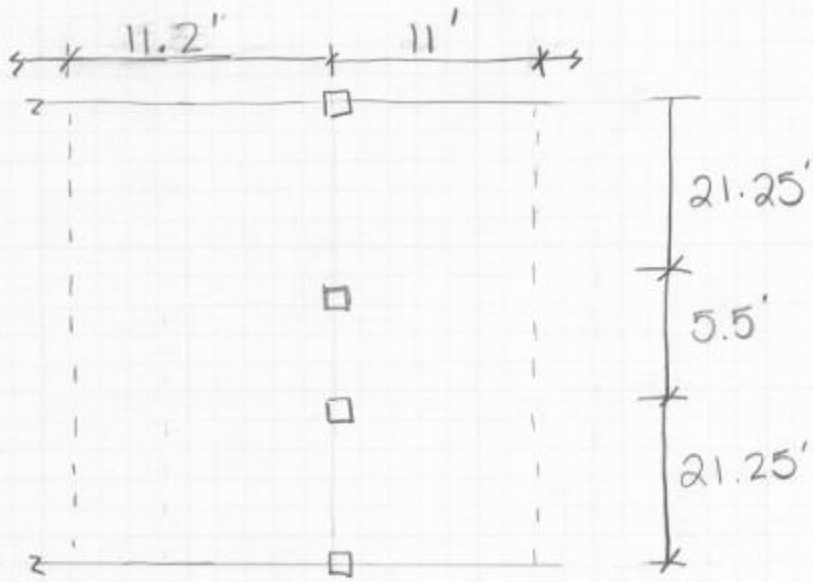
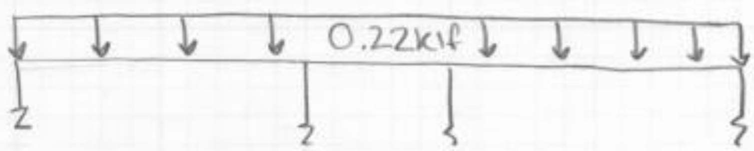
SECTION 2

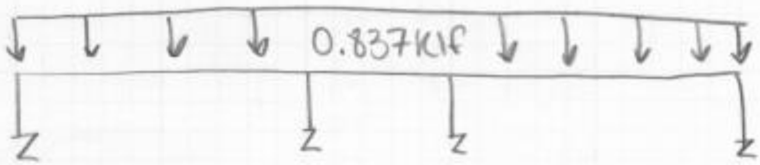


dead load roof

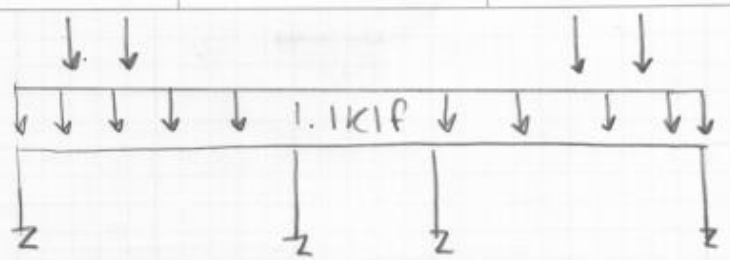
$(11.2 + 11)(10psf) = 222\,plf$
ft
$= 0.22\,klf$



live load roof = 0.837 klf



live load will be the same as the other section 25
The greater area will control.



$T_L = 0.22\,klf + 0.837\,klf = 1.1\,klf$

Snow load on roof:

$(11.2 + 11)(21\,psf) = 466.2\,plf$

feet

$= 0.47\,klf$

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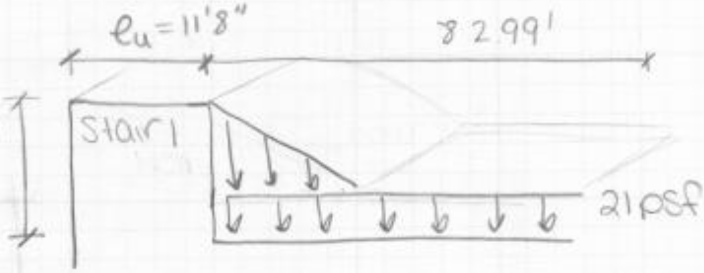
Snow Drift on Roof:



82.99 ≥ 15' drift require

$C_e = 1.0$ table 7-2

$C_t = 1.0$ table 7-3

$C_s = 1.0$

$I_s = 1.0$ table 7-4

flat roof snow load $P_f = 21\,psf$

snow density $\gamma =$

$$\gamma = 0.13P_g + 14 \leq \text{ground snow load}$$

$$P_f = 0.7\, C_e C_T I_s P_g$$

$$21 = 0.7(P_g)$$

$$\boxed{P_g = 30}$$

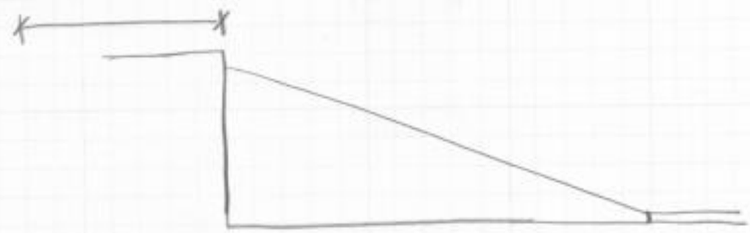
$$\gamma = 0.13(30) + 14 = 17.9$$

$$P_s = C_s P_f = 1.0(21) = 21\,psf$$

$$h_b = P_s/\gamma = 21/17.9 = 1.17'$$

$\frac{h_c}{h_b} \geq 0.2$ $\quad \frac{11'}{1.17} > 0.2$ yes

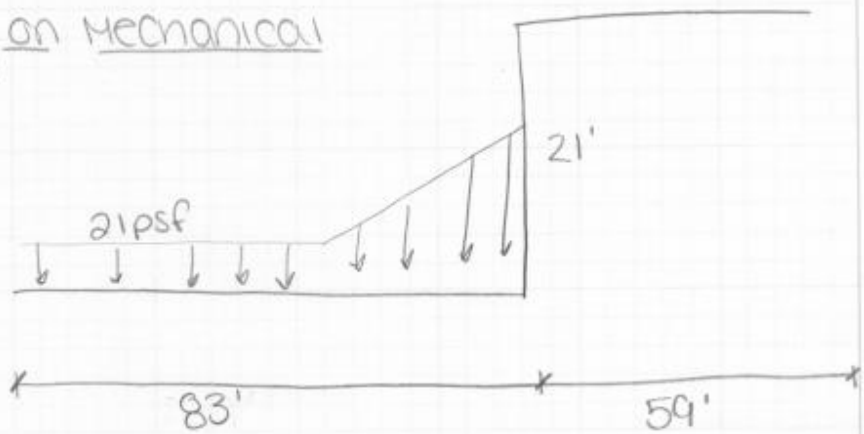
27

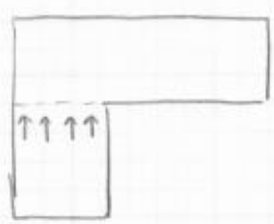


$l_u < 25$ feet.

Ignore the drift.

Drift on Mechanical area:





$\gamma = 0.13 P_g + 14$

$\gamma = 0.13(30) + 14$

$= 17.9$

$P_f = 0.7 C_e C_T I_s P_g$

$21 = 0.7 P_g$

$P_g = 30$

$h_b = C_s P_f$

$= 1.0(21)$

$= 21 psf$

28

$h_b = P_s / \gamma = 21 / 17.9 = 1.17'$

$h_r = h_c + h_b$

$\frac{h_c}{h_b} \geq 0.2 \qquad \frac{20}{1.17}$ yes

windward:

$l_u > 25$ ✓

$h_d = 0.75\left(0.43\, l_u^{1/3} (P_g + 10)^{(1/4)} - 1.5\right)$

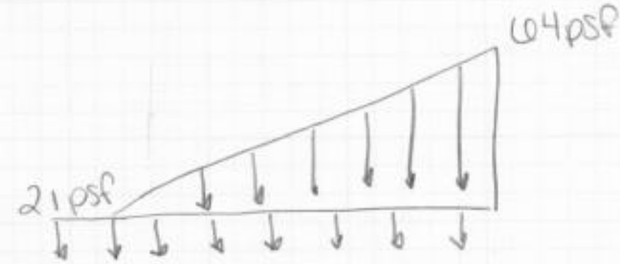
$h_d = 2.4\text{ft}$

$W = 4(2.4\text{ ft}) = 9.6 \text{ feet}$

$P_d = h_d \gamma = 2.4(17.9) = 43 \text{ psf}$

$P_d + P_f = 43 + 21 = 64 \text{ psf}$

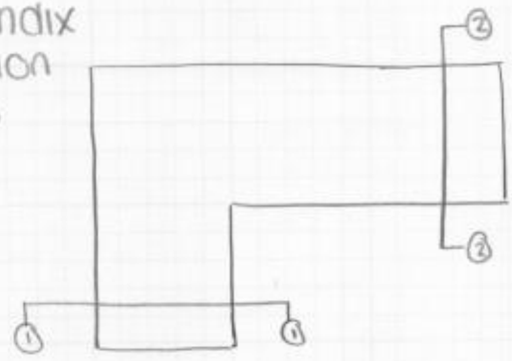
$\text{gradient} = \frac{P_d}{W} = \frac{43}{9.6} = 4.5$



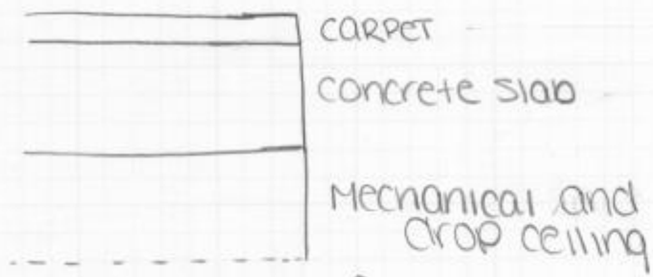
29

GRAVITY LOADS TYPICAL floor (3rd to 7th)

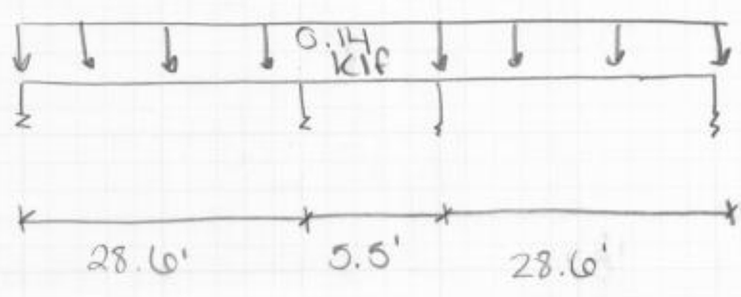
See Appendix I for section locations



Section 1 dead loads (same as roof section)



super imposed = 5psf

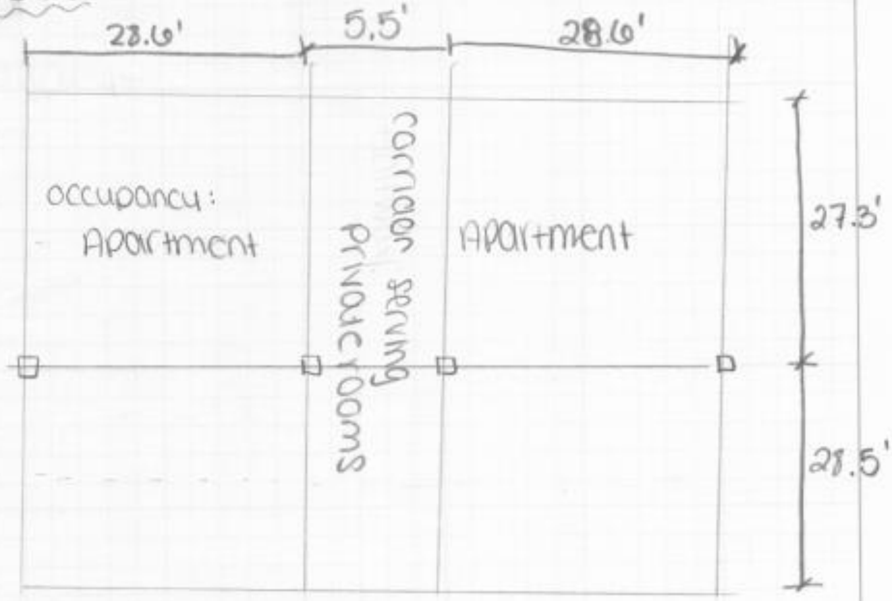


TRIB AREA:

(13.7 + 14.2)(5psf) = 139.5psf
feet
= 0.14klf

30

live load:



(ASCE7 Occupancy live loads)

Apartments = 40 + 10psf (partitions)

corridors serving private rooms = 40 psf

$\underbrace{(13.7 + 14.2)}_{\text{feet}}(50\text{psf}) = 1395\text{ plf}$
$= 1.395\text{ klf}$

$\underbrace{(13.7 + 14.2)}_{\text{feet}}(40) = 1116\text{ plf}$
$= 1.116\text{ klf}$

31

live load reductions:

$K_{LL}A_T > 400 ft^2$

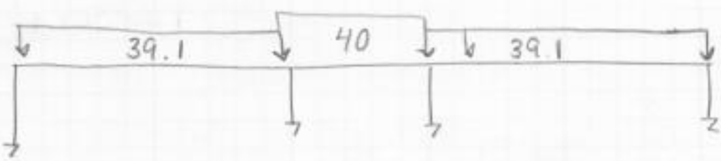
$A_T = (27.3/2 + 28.5/2)(28.6) = 798 sf$

K_{LL} - two way slab = 1.0

Live load can be reduced for Apartment load.

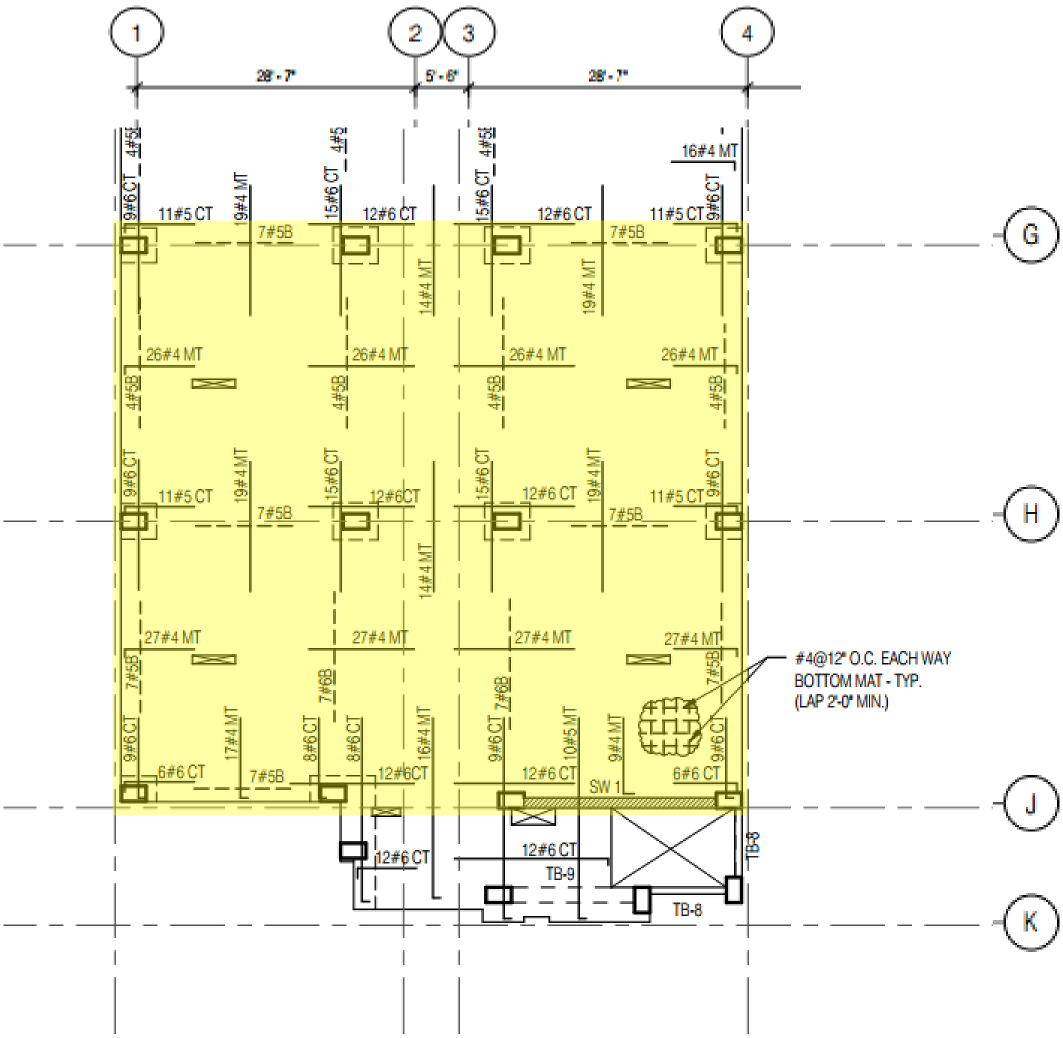
$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

$$= 50\left(0.25 + \frac{15}{\sqrt{1.0(798)}}\right) = 39.1 psf$$

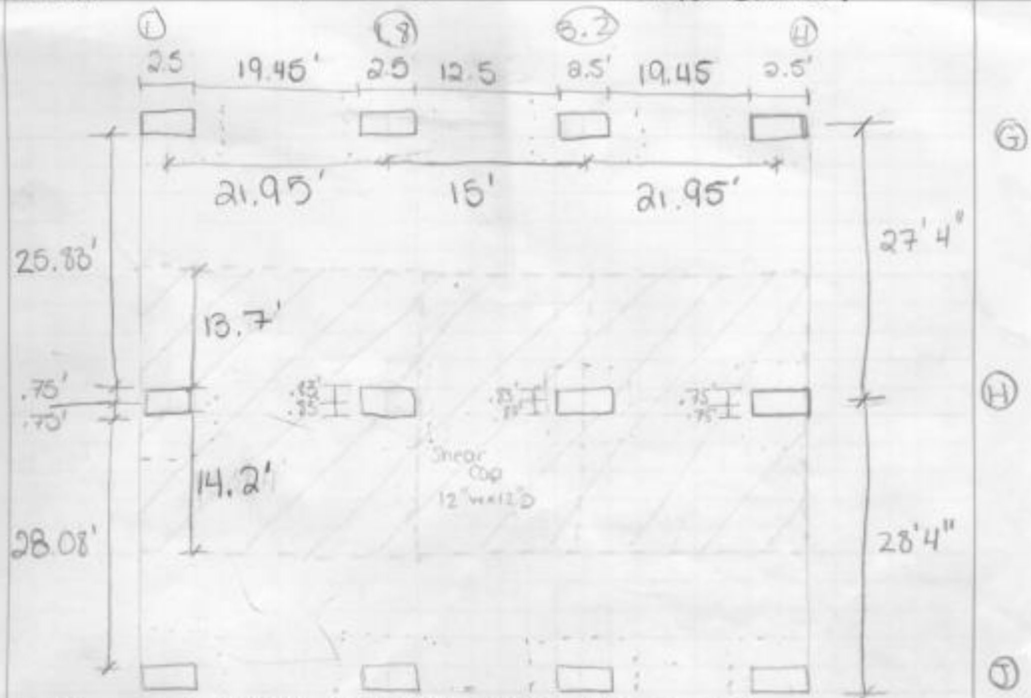


32

Typical Floor Bay Being Analyzed



CHRISTOPHER CIOFFI | TECH 3 | TWO-WAY Slab CHECK



TWO-Way Slab CHECK: (at floor 6)

Column H1 = 30"x18"

Column H1.8 = 30"x20"

Column H3.2 = 30"x20"

Column H4 = 30"x18"

Slab thickness - 8" concrete

#4 reinforcing @ 12" O.C. Each way BTM mat TYP.
LAP 2'0" min.

f_y = 5000 psi , f_y = 60 ksi

DEAD LOAD (exterior panel H1 to H1.8).

- concrete = $150\,psf\left(\frac{8''}{12''}\right) = 100\,psf$
- MEP = 10 psf

TWO-WAY SLAB SPOT CHECKS

CHRISTOPHER Cioffi | TECH 3 | Two way slab check

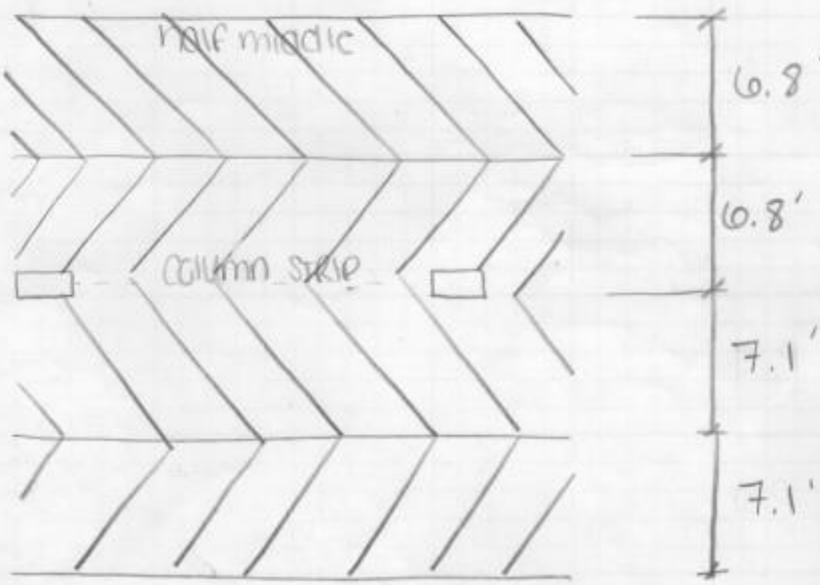
live loads:

appartments = 40 + 10 psf = 50psf

$q_u = 1.2(110) + 1.6(50) = 212$ psf

$l_n = 21.95'$

$l_2 = 14.2 + 13.7 = 27.8$



h_{min} exterior panels $= \frac{l_n}{30} = \frac{19.45 \times 12}{30} = 7.78 < 8"$ thick GOOD

h_{min} interior $= \frac{l_n}{36} = \frac{12.5 \times 12}{36} = 4.2" < 8"$ thick GOOD

ACI 318-08

$M_0 = \frac{q_u l_2 l_n^2}{8} = \frac{(0.212)(27.8)(21.95^2)}{8} = 354.9$ k-ft

Christopher Cioffi | TECH3 | Two way slab Check

positive and negative moments:

$M^-_{(interior)} = 0.7M_o = 0.7(-354.9) = -248.46'k$

$M^+ = 0.52M_o = 0.52(354.9) = 184.55'k$

$M^-_{(exterior)} = 0.26(M_o) = 0.26(-354.9) = -92.27'k$

column strip moments:

$\frac{\ell_2}{\ell_1} = \frac{27.8}{21.95} = 1.27$

$\alpha_{F1}\frac{\ell_2}{\ell_1} = 0 \rightarrow$ no beams parallel to H.

$M^-_{(interior)} = 0.7(-248.46) = -174.13'k$

$M^+ = 0.6(184.55) = 110.73'k$

$M^-_{(exterior)} = 1.0(-92.27) = -92.27'k$

$\beta_t = \frac{E_{cb}C}{2E_{cs}I_s} = 0$ no exterior beam.

middle strip moments:

$M^-_{(interior)} = 0.3(-248.46) = -74.54$

$M^+ = 0.4(184.55) = 73.82$

$M^-_{exterior} = 0$

column strip:

$b = (7.1 + 6.8) \times 12) = 166.8$

$h = 8''$ thick

$d = 8'' -$ CLR cover $- \frac{1}{2}$ bar diameter

$= 8'' - 1.0 - \frac{1}{2}(0.5) = 6.75''$

TWO-WAY SLAB SPOT CHECKS

Christopher Cioffi | TECH 3 | Two-Way Slab Check

z

$$\varepsilon_y = \frac{\varepsilon_c}{c}(d-c) = \frac{0.003}{0.823}(6.69-0.823)$$

$$= 0.21 > 0.005 \quad \Phi = 0.9$$

$$\Phi M_n = \Phi A_s F_y \left(d - \frac{c}{2}\right)$$

$$= 0.9(7.77)(60)\left(6.69 - \frac{0.823}{2}\right)$$

$$= 2634.33/12 = 219.53'k$$

$$219.53'k > 110.73'k \quad \text{GOOD} ✓$$

Reinforcing for end column strip:

$$A_s = 28(0.2) + 7(0.31) = 7.77$$

$$a = \frac{A_s F_y}{0.85 f'c\, b} = \frac{(7.77)(60)}{(0.85)(5)(166.8)} = 0.66$$

$$c = \frac{a}{B} = \frac{0.66}{0.8} = 0.825 \quad d = 6.69$$

$$\varepsilon_y = \frac{\varepsilon_c}{c}(d-c) = \frac{0.003}{0.953}(6.25-0.825) = 0.014 > 0.005 \quad \Phi = 0.9$$

$$\Phi M_n = \Phi A_s F_y (d - c/2)$$

$$= (0.9)(9.01)(60)(6.625 - 0.953/2)$$

$$= 2991.5/12 = 249.3'k$$

$$249.93'k > -92.27'k \quad \text{good} ✓$$

Christopher Cioffi | TECH 3 | Two way slab check

# of Bars $= [(6.8\times2)+(7.1\times2)]\times12/12 = (28)$

(28) #4 @12" on Bottom

Reinforcing for interior column strip:

$A_s = 28(0.2) + 7(0.31)$
$= 7.77 in^2$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{7.77(60)}{(0.85)(5)(166.8)} = 0.66$

$C = \frac{a}{\beta} = \frac{0.66}{0.8} = 0.825$ $\quad d = 8" - 1.0 - \frac{1}{2}(0.625)$
$d = 6.69$

$\varepsilon_y = \frac{\varepsilon_c}{c}(d-c) = \frac{0.003}{1.15}(6.25 - 0.825)$
$= 0.014 > 0.005 \quad \Phi = 0.9$

$\Phi M_n = \Phi A_s F_y (d - \frac{c}{2})$
$= 0.9(10.88)(60)(6.625 - 1.15/2)$
$= 2991.5/12 = -249.3$

-249.30 'K > -174.13 good ✓

Reinforcing for center column strip:

$A_s = 28(0.2) + 7(0.31) = 7.77 in^2$ (#5)

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{(7.77)(60)}{(0.85)(5)(166.8)} = 0.658$

$C = \frac{a}{\beta} = \frac{0.658}{0.8} = 0.823$ $\quad d = 8" - 1.0 - \frac{1}{2}(0.625)$
$= 6.69"$

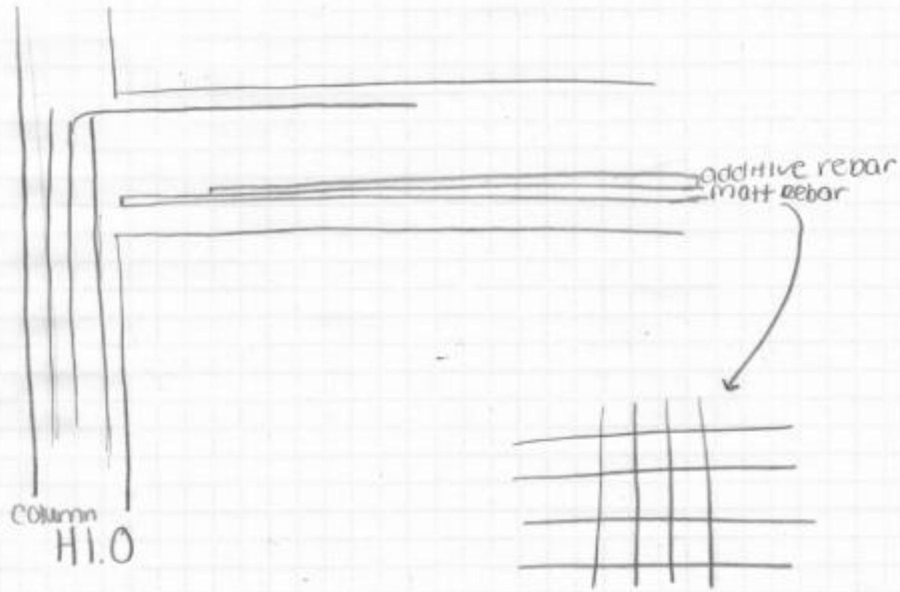
$$\varepsilon_y = \frac{\varepsilon_c}{c}(d - c) = \frac{0.003}{1.15}(6.25 - 0.8225)$$

$$= 0.014$$

$$\phi M_n = \phi A_s f_y (d - c/2)$$

$$= 0.9(10.88)(60)(6.625 - 1.15/2)$$

$$-249.3'K > -174.13 \quad \text{good} \checkmark$$



Exterior TYP PANEl PASSES ALL Moment Checks

SHEAR CAP CHECK

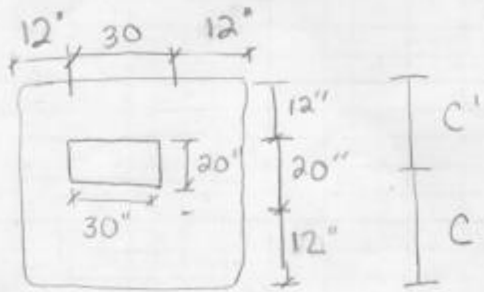
CHRISTOPHER Cioffi | TECH 3 | Shear cap CHECK

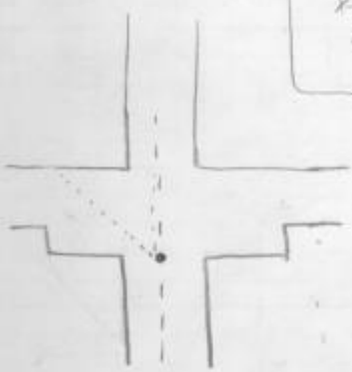
SHEAR CAP CHECK: {*documents unclear - analized with 2 different diminsions (BOTH PASS)}

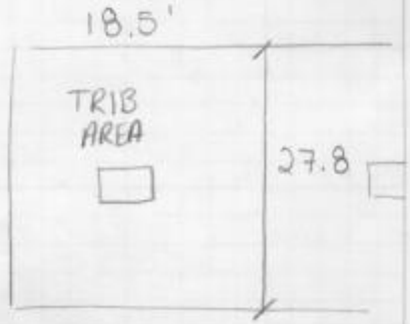
ACI 318-08 table 9.5c

$$\text{thickness} > \frac{\ell_n}{33} = \frac{23.58 \times 12}{33} = 8.6"$$

Punching shear:







self weight =

$$1.2\left[\frac{12}{12}(.150) + 10\,\text{pcf}\right] + 1.6(50)$$

$$= .272\ \text{ksf}$$

$$V_u = 0.272\left[(18.5 \times 27.8) - \frac{(2a + C_1)(2a + C_2)}{144}\right]$$

SHEAR CAP CHECK

Christopher Cioffi TECH3 Shear cap Check

$V_u = 0.272[514.3 - (44 \times 54/144)] = 135.4k$

$b_o = (54 + 44) \times 2 = 196 in$

$$V_c = \begin{bmatrix} 2 + 4/\beta \\ \frac{\alpha_s d}{b_o} + 2 \\ 4 \end{bmatrix} \lambda\sqrt{f'c}\, b_o d$$

$V_c = (2 + 4/(30/20))(\sqrt{5000})(196)(7.25) = 468.91$

$V_c = \left(\frac{40(7.25)}{196} + 2\right)(\sqrt{5000})(196)(7.25) = 349.63^k$

$V_c = 4\sqrt{5000}(196)(7.25) = 401.92^k$

$\Phi V_c = 349.63 \times 0.75 = 262.2^k > 135.4^k$

Slab is okay for punching shear

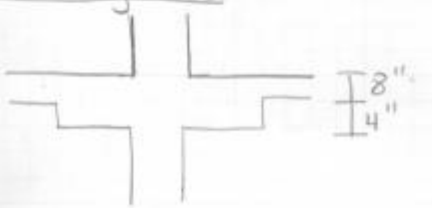
SHEAR CAP CHECK

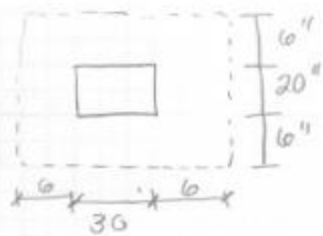
Christopher Cioffi | TECH 3 | Shear Cap Check

Shear caps

thickness $> \frac{ln}{33} = \frac{23.58 \times 12}{33} = 8.6''$

punching shear:





self weight =

$1.2\left[\frac{12}{12}(150) + 10\right] + 1.6(50)$

$= 0.272$ ksf

TRIB AREA $= (18.5)(27.8) = 514.3$

$V_u = 0.272(514.3 - (32 \times 42)/144) = 137.4k$

$b_o = (32 + 42) \times 2 = 148 in$

$V_c = \begin{bmatrix} 2 + 4/B \\ \frac{\alpha_s d}{b_o} + 2 \\ 4 \end{bmatrix} \sqrt{f'c}\, b_o\, d$ → $= \sqrt{f'c}\, b_o\, d$

$= \sqrt{5000}(148)(7.25)$

$= 75872.6$

$= 2 + \frac{4}{30/20}$

$= \frac{40(7.25)}{146} + 2$

$= 4$ → controls

$(4)(75872.6)/1000$

$= 303.5k$

$\Phi V_c = 303.5(0.75) = 227.62k > 137.4k$

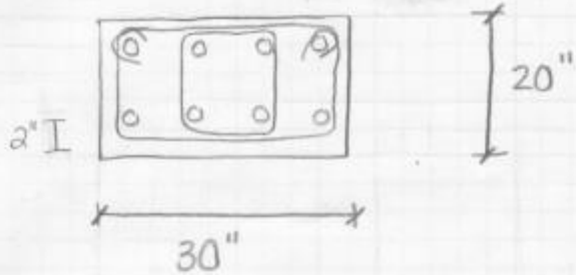
okay for punching shear ✓

INTERIOR COLUMN

Christopher Cioffi | TECh 3 | Interior column check gravity load

Spot Check - Column H1.8

analized at floor 4



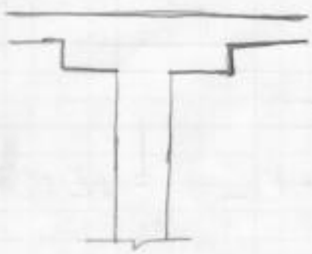
INFORMATION

8 #9 bars vert

#3 bars spaced @ 18" O.C

f_y = 60 ksi

2" min cover.



roof slopes 14" - 8" → avg for design.

TRIB AREA = (18.48)(27.9) = 515.6 sf

roof Loads:

dead = 10 psf (515.6) = 5.16 k

Live = cannot reduce

= 30 psf (515.6) = 15.47 k

slab = $\left(\frac{8+14}{2}\right)\left(\frac{1}{12}\right)$(515.6)(150)

= 70.90 k

Shear cap = $\left(\frac{4}{12}\right)$(54×44/144)(150) = .825 k

Column self = $\left(\frac{30\times20}{12}\right)$(12.67)(150) = 95 k

floor #7 Loads:

dead = 10(515.6) = 5.2 k

live (Reduced) = $K_{LL}A_T$ > 400 sf $L_r = L_o(0.25 + 15/\sqrt{K_{LL}A_T})$

K_{LL} = 4.0   = 50(0.25 + 15/$\sqrt{2062.4}$)

= 45.5(515.6) = 23.46 k   = 45.5 psf

slab = $\left(\frac{8}{12}\right)$(515.6)(150) = 51.6 k

shear cap = $\left(\frac{4}{12}\right)$(54×44/144)(150) = 0.825 k [conservative]

column self $\left(\frac{30\times20}{12}\right)$(10.67)(150) = 80.03 k

floors (6 - 5) Loads:

dead = 5.20(2) = 10.4

live = 23.46(2) = 46.92 k

slab = 51.6(2) = 103.2 k

TYPICAL MEMBER SPOT CHECK FOR GRAVITY LOADS

INTERIOR COLUMN

Christopher Cioffi | TECH 3 | interior column check gravity Load

shear cap = 0.825(2) = 1.65

column self = 80.03(2) = 160.06

Total load = 1.2 [5.16 + 70.9 + .825 + 95 + 10.4 + 51.6 + 0.825 + 5.2 + 51.56 + 103.2 + 1.65 + 160.06]

+1.6 [15.47 + 23.46 + 46.92]

= 701.8 + 137.4 = 839.2

$\Phi P_n = 0.8\Phi[0.85f'c(A_g - A_{st}) + f_yA_{st}]$

$= 0.8(0.65)[0.85(4)(600-8) + 60(8)]$

= 1296.3K

839.2 < 1296.3 kips good ✓

analized at 2nd floor :-

floors 3 and 4 loads

dead = 10(515.6) = 5.2K

live reduced = 45.5(515.6) = 23.46

slab = $(\frac{8}{12})$(515.6)(150) = 51.6K

shear cap = 0.825K

column self = 80.03

1.2(2)(137.66) + 1.6(2)(23.46) = 405.5

839.2 + 405.5 = 1244.7K

$\Phi P_n = 0.8\Phi[0.85f'c(A_g - A_{ST}) + f_yA_{st}]$

$= 0.8(0.65)[0.85(5)(600-8) + 60(8)]$

= 1557.92

1244.7K < 1557.92 good ✓

INTERIOR COLUMN

Christopher Cioffi	TECH3	interior column check gravity load

analized at floor 1:

2nd floor loads:

Same as typ floors except column

$A_s = 12(1.56) = 18.72$

$30 \times 30 = 900 in^2 = A_g$

column self $= \left(\frac{30\times30}{12}\right)(150)(19) = 213k$

$57.63 + 213 = 270.6k$

Total loads $= 1.2(270.6) + 1.6(23.46) + 1244.7 = 1606k$

$\Phi P_n = 0.8(0.65)[0.85(5)(900 - 18.72) + 60(18.72)]$

$= 2531.7k$

$1606k < 2531.7k$ good ✓

analized at ground:

1st floor load:

dead $= 10(515.6) = 5.2k$

live $= 100(0.25 + 15/\sqrt{2062.4}) = 58.02(515.6) = 29.92k$

slab $= \left(\frac{8}{12}\right)(515.6)(150) = 51.6k$

column self $= \left(\frac{30\times30}{12}\right)(13)(150) = 146.3k$

drop panel $= (6.25\times10)(150)(4/12) = 3.125k$

Total load $= 1606 + 1.2(206.2) + 1.6(29.92)$

$= 1606 + 206.5 + 47.87$

$= 1901.3k$

$\Phi P_n = 0.8(0.65)[0.85(5)(900 - 18.72) + 60(18.72)]$

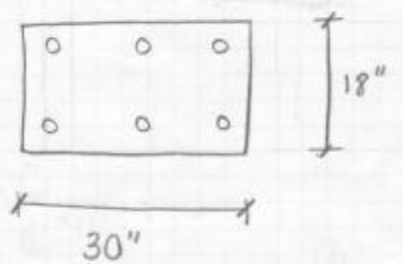
$= 2531.7k$

$1901.3k < 2531.7k$ good ✓

INTERIOR COLUMN

Christopher Cioffi | TECH 3 | Exterior column Check gravity load

Spot check exterior column H1
evaluated at floor 4



INFORMATION

6 #9 bars vert

#3 bars spaced @18" O.C

$f_y = 60$ ksi

2" min cover

$A_g = 540\ in^2$

Trib area = (13.7 + 14.2)(12.23) = 341.2 sf

Roof Loads:

dead = 10 psf (341.2) = 3.4K

live = cannot reduce - parapet snow drift negligible see tech 2

= 30(341.2) = 10.23K

slab = $\left(\frac{8+14}{2}\right)\left(\frac{1}{12}\right)(341.2)(150)$ = 46.92K

shear cap = $\left(\frac{4}{12}\right)(42 \times 42)\left(\frac{1}{144}\right)(150)$ = 0.613k

column self = $\left(\frac{30 \times 18}{12}\right)(12.67)(150)$ = 85.52k

floors 7-5 loads:

dead = 10psf (341.2) = 3.4k

Live load → reduction $K_{LL}A_T$ > 400sf

4.0(341.2) > 400sf

$L_r = 50\ (0.25 + 15/\sqrt{1364.8}) = 32.8$ psf (341.2) = 11.19k

slab = $\left(\frac{8}{12}\right)(341.2)(150)$ = 34.12k

Shear cap = $\left(\frac{4}{12}\right)(42 \times 42)\left(\frac{1}{144}\right)(150)$ = 0.613K

column self = $\left(\frac{30 \times 18}{12}\right)(10.63)(150)$ = 71.75K

Total load = 1.2(136) + 1.6(10.23) + 1.2(3)(109) + 1.6(11.19) = 589k

TYPICAL MEMBER SPOT CHECK FOR GRAVITY LOADS

INTERIOR COLUMN

Christopher Cioffi	TECH 3	Exterior column check gravity load

$\Phi P_n = 0.85(0.65)[0.85(4)(540) - 60(6 \times 1.0)]$
$= 815.5k$

$589 < 815.5k$

<u>evaluated at floor 28</u>

<u>floors 4-3 Loads:</u>

dead = 10 psf (341.2) = 3.41k

live = 32.8(341.2) = 11.19k

Slab = $(8/12)(341.2)(150) = 34.12K$

Shear cap = $(4/12)(42 \times 42)(\frac{1}{144})(150) = 0.613k$

column self = $(\frac{30 \times 18}{12})(10.63)(150) = 71.75k$

Total load = 589 + 1.2(2)(109.89) + 1.6(2)(11.19)
= 888K

$\Phi P_n = 0.85(0.65)[0.85(5)(540) - 60(6 \times 1.0)]$
$= 1069.1k$

888k < 1069.1 k good ✓

<u>evaluated at floor 1:</u>

dead = 3.41k
live = 11.19
Slab = 34.12K
Shear cap = 0.613K
column self = $(\frac{30 \times 18}{12})(19)(150) = 128.3k$
Total load = 888 + 1.2(166) + 1.6(11.19) = 1105K
$\Phi P_n = 0.8(0.65)(0.85(5)(540) - 60(6 \times 1)]$
$= 1069.1k$
1105 K < 1069.1K

INTERIOR COLUMN

Christopher Cioffi	TECH 3	Exterior Column Check gravity load

evaluated at ground:

1st floor Load:

dead = 3.41

live = (100 (0.25 + 15/√13(64.8)) = 65.6(341.2) = 22.3k

slab = 431.2

drop panel = (5.5)(10)(4112)(1/144)(150) = .19k

column self = (30x18/12)(13')(150) = 87.7k

Total load = 816.2 + 1.2(522.5) + (1.6)(22.3)

1478 < 1069.1K NOT GOOD

column failed test

possible reasons →

- was too conservative with concrete weight – shear caps.
- too conservative in general

Alternative System:
One Way Slab Design
With Reinforced Beams

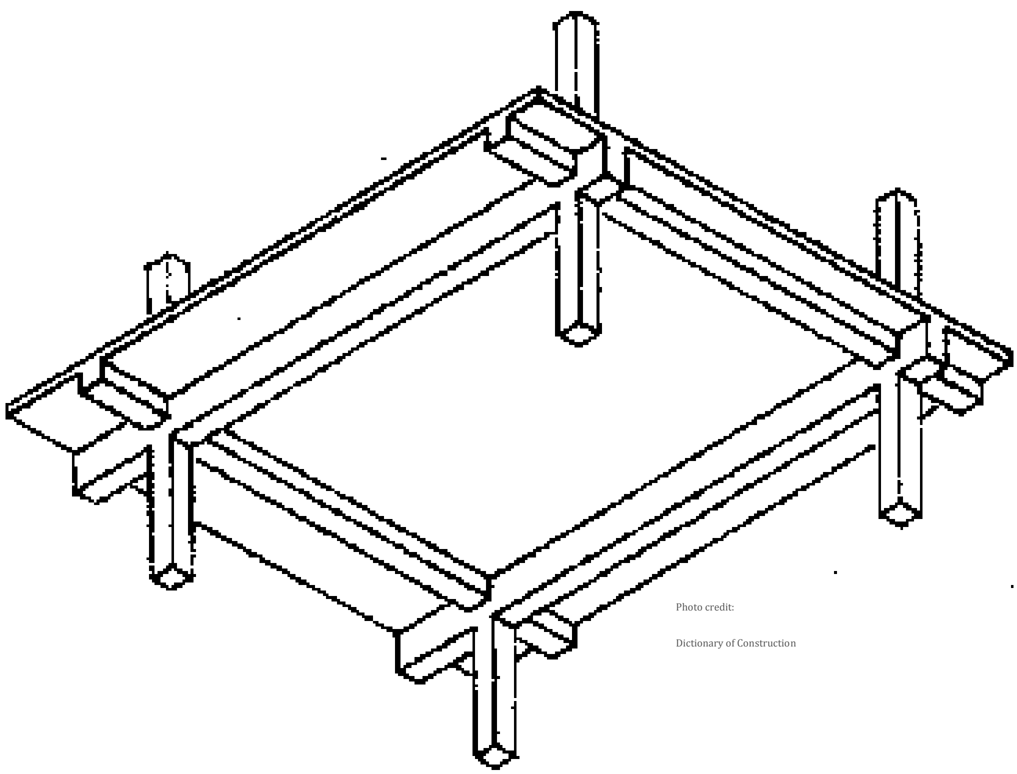
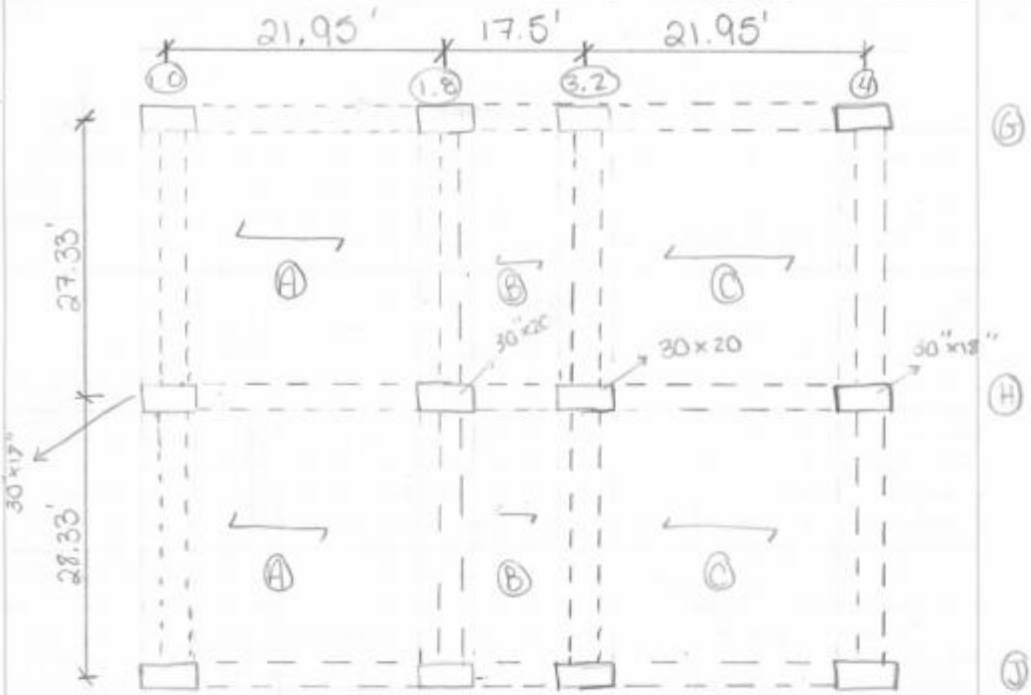


Photo credit:

Dictionary of Construction

Christopher Cioffi | TECH 3 | Design: 1 way slab



INFORMATION:

$f'c = 5000$ psi

for deflection control [table 9.5(a)]

slab min thickness

endspan (a and c) = $\ell/24$

$\frac{\ell}{24} = \frac{21.95(12)}{24} = 10.9''$ TRY 11" thick slab

interior (b)

$\frac{\ell}{28} = \frac{17.5(12)}{28} = 7.5''$

Design for panel (A)

$W_{DL} = 10\text{ psf} + 150\text{ psf}\left(\frac{11''}{12''}\right) = 147.5\text{ psf}$

$W_{LL} = 50\text{ psf}$ for apartments

Christopher Ciotti	TECH 3	Design: 1 way slab

Load combination (ASCE 7-10 §2.3.2)

1.2D + 1.6L

→ controls

$W_T = 1.2(147.5) + 1.6(50) = 257 \text{ psf}$

TRY #4 bars

$d = h - \text{clear cover} - d_{\#4}/2 = 11'' - 1.5 - \frac{0.5}{2}$

$d = 9.25''$

beam design: [1.0G - 1.0H]

$w = (257)(21.95)/1000 = 5.64 \text{ k/ft}$

$M_u = (5.64)(28.33 - (\frac{18}{12}))^2 / 8 = 507.5 \text{ ft-k}$

$507.5 \times 1.1 = 558.25 \text{ 'k}$

↑ estimate eff of self wt is 10%

beamsize → $bd^2 = 20 M_u$

$b = \frac{4}{5} d$ $\qquad \frac{4}{5} d^3 = 20(558.25)$

$d = 24''$

$b = \frac{4}{5}(24)$

$= 19.2 \approx 19.5''$

$h = 24 + 2.5 = 26.5$

$W_{self} = \frac{26.5(19.5)}{144} \times (150) = .538$

$W_u = 5.64 + 1.2(.538) = 6.29 \text{ k}$

$M_u = \frac{(6.29)(26.8)^2}{8} = 564.7 \text{ 'k}$

$bd^2 = 20 M_u$

$(19.5)(24^2) = 20(564.7)$

$11232 < 11294$ ✓ good

Christopher Cioffi	TECH 3	Design: 1 way slab

Required steel:

$A_s = \frac{M_u}{4d} = \frac{564.7}{4(24)} = 5.8\,in^2$

TRY 6 #9 rebar $= 6 \times 1.0 = 6 in^2$

$d = 26.5'' - 1.5_{CLR} - \frac{1.28}{2} = 24.36$

nominal moment

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6.0(60)}{0.85(5)(19.5)} = 4.34''$

$c = a/\beta_1 = 4.34/0.8 = 5.425$

$\varepsilon_s = 0.003\left[\frac{24.36 - 5.425}{5.425}\right] = 0.01047 > .005$ okay ✓

use $\phi = 0.9$

$\phi M_n = 0.9(A_s)(f_y)(d - a/2)$
$= 0.9(6.0)(60)(24.36 - 4.34/2)$
$= 7189.6/12 = 599.13$ 'k

$\phi M_n > M_u$

$599.13 > 588.25$ okay

min area of steel:

$A_{s,min} = \frac{200}{f_y} bd = \frac{200}{60{,}000}(19.5)(24.36) = 1.58\,in^2$

$A_s = 6 in^2 > 1.58 in^2$ good

Check max reinforcing ratio:

$\rho_{max} = 0.85\beta_1(f'_c/f_y)(\varepsilon_u/\varepsilon_u + \varepsilon_y)$
$= 0.85(0.8)(5/60)(0.003/0.003 + 0.005)$
$= 0.0213$

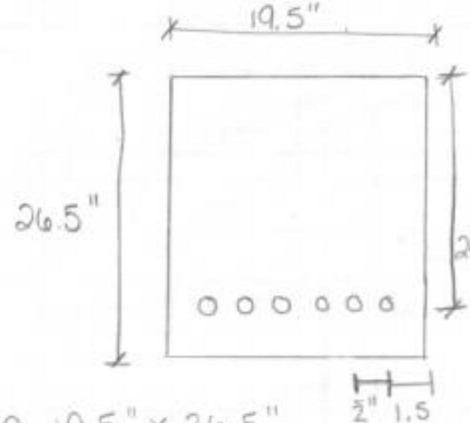
Christopher Cioffi | TECH 3 | Design: 1 way Slab

$\rho = A_s / bd = 6.0 / (19.5)(24.36)$

$= 0.0126$

$\rho < \rho_{max}$

$0.0126 < 0.0213$ good ✓



$1.128" \times 6 = 6.768"$

$+ 2(1.5) = 3"$

9.768

$19.5 - 9.768 = 9.7"$

$9.7/5 = 1.9 \approx 2"$

use a 19.5" × 26.5" with 6 #9 rebar

<u>Slab design:</u>

$T = 11"$ TRY #4 bars

unit strip method ↴

using a 1' width method

$A_{s_{min}} = 0.002(12")(11") = 0.264$

↳ ACI 318-11 § 7.12.2.1

$A_s / 1ft = A_{rebar}\left(\frac{12"}{spacing}\right)$

$0.264 = 0.2(12"/spacing)$

spacing = 11"

$S_{max} \begin{vmatrix} 3t \\ 18" \end{vmatrix} = \begin{matrix} 3(11) = 33" \\ 18" \end{matrix}$ 11" okay ✓

ACI 318-11 - §10.5.4

Christopher Cioffi | TECH 3 | Design: 1way slab

Spacing crack control

$$S = 15\left(\frac{40{,}000}{f_s}\right) - 2.5C_c$$

$$= 15(40{,}000/60{,}000) - 2.5(0.75)$$

C_c - distance from tension rebar to nearest surface $= 8.125''$

$$S = 12\left(\frac{40{,}000}{f_s}\right)$$

$$= 12(4/6) = 8'' \longrightarrow \text{controls}$$

TRY #4 rebar at 8" O.C.

$A_s = 0.20\,in^2$

$\frac{12}{8} \times 0.2 = 0.3\,in^2/ft$

$\rho = A_s/bd = 0.3/(12'')(11'') = 0.0023$

$\phi = 0.9$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.3)(60)}{0.85(5)(12)} = 0.353$

$d = 11'' - 0.75 - \frac{0.5}{2} = 10''$

$\phi M_n = \phi A_s f_y (d - a/2)$

$0.9(0.3)(60)(10 - 0.353/2) = 159.14/12$

$= 13.26\,k\text{-}ft$

$W = 1.2\left[\left(\frac{11}{12}\right)(150)(1') + 10psf(1')\right] + 1.6[50(1')]$

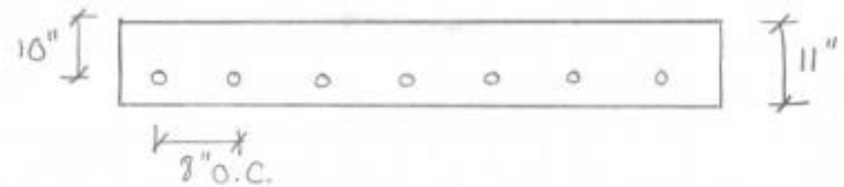
$W = 0.257\,klf$

$M_u = \frac{0.257(21.95)^2}{8} = 15.48'k$

$\phi M_n < M_u$ good ✓✓

Christopher Cioffi | TECH 3 | Design: 1 way Slab

use an 11" slab with #4 Rebar 8" O.C.



Deflection:

Beam → $I = \frac{(19.5)(26.5)^3}{12} = 30240.6$

$\Delta_{TL} = \frac{5w\ell^4}{384EI}$

$w_T = \left[10 + \frac{11}{12}(150)\right](21.95) + \frac{19.5(26.5)}{144}(150)$

$= 3.77$

$E = 150^{1.5}\sqrt{f'c}$

$= 150^{1.5}\sqrt{5}$

4108

$\Delta_{TL} = \frac{5(3.77)(28.08)^4}{384(30240.6)(4108)} \times 1728$

$= 0.424$

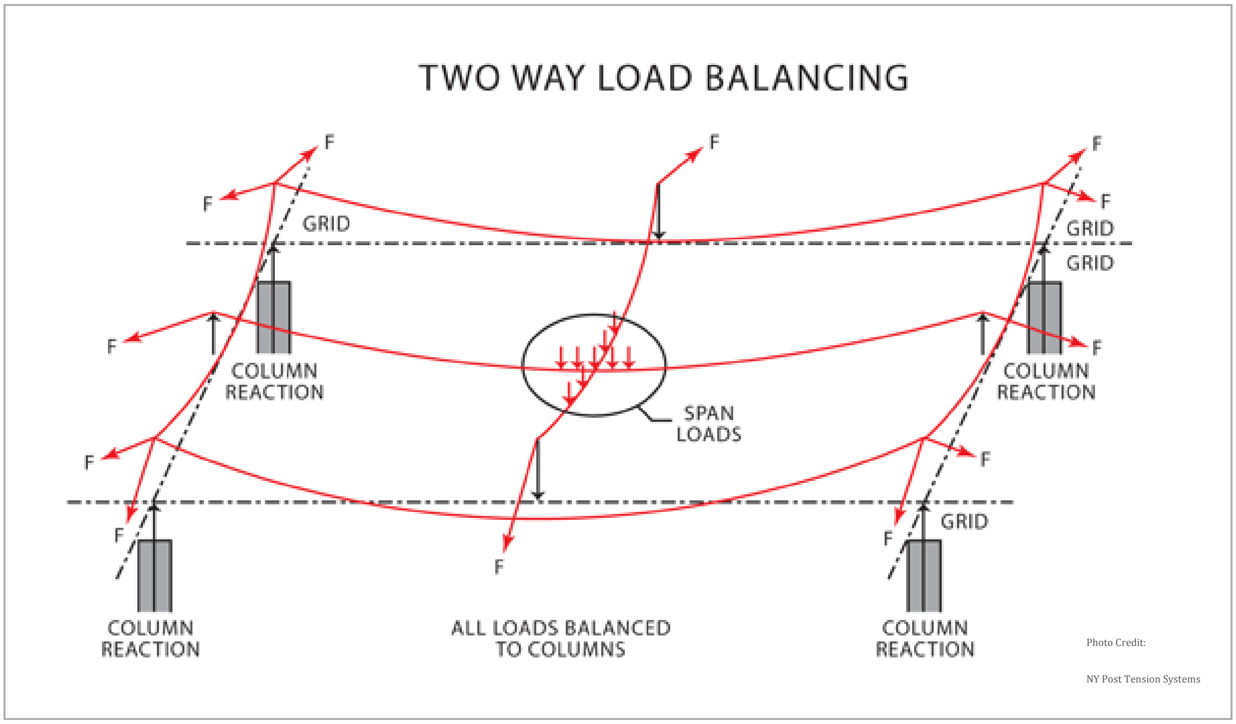
$\frac{\ell}{480} = \frac{28.08 \times (12)}{480} = 0.702"$

$0.424" < 0.702"$ good ✓

$\frac{\ell}{360} = \frac{28.08(12)}{360} = 0.94"$

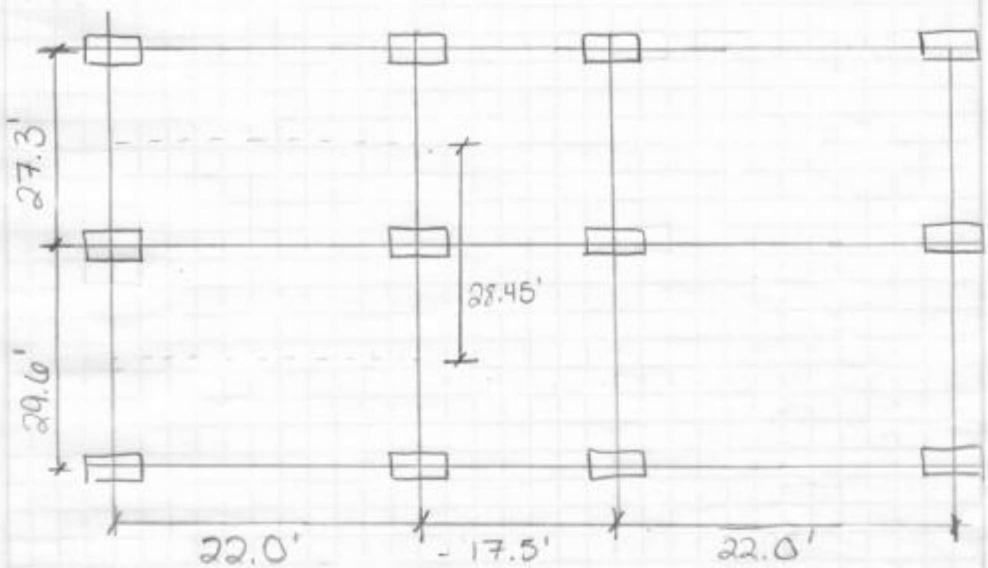
Alternative System:

Two- Way Post Tensioned Slab Design



Christopher Cioffi TECH 3 PT 2-Way slab design

Two way POST TENSIONED Design:



loads:

dead load - (framing) = self weight

dead SI = 5psf

live load = 40 psf + 10 psf(partitions) = 50psf

2 hour fire rating

Materials:

Normal weight concrete

f'_c = 5,000 psi

f'_{ci} = 3,000 psi

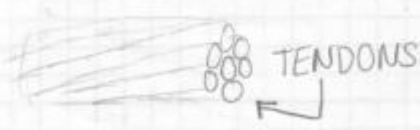
rebar → f_y = 60,000 psi

post tension ↴

unbonded tendons

1/2" ψ 7 wire strands

f_{pu} = 270 ksi



Christopher Cioffi TECh 3 PT 2-way slab design

estimated prestresses losses (ACI 318-11 §18.6).
=15 ksi

f_{se} (ACI 318-11 §18.5.1) - post-tensioning tendons at anchorage devices and couplers, immediately after force transfer.

$f_{se} = 0.7 f_{pu} = 0.7(270) - 15$ [estimated losses]
$= 174$ ksi

$P_{eff} = A_{PT} \times f_{se} = 0.153 in^2 (174 ksi) = 26.6$ k/tendon

<u>determine preliminary slab thickness:</u>

$\frac{L}{h} = 45$ $h = 29.6(12)/45 = 7.89''$
use 8" slab

longest span = 29.6 ft

<u>Loads:</u>

$W_{DL} = \frac{(8'')}{12}(150) = 100$ psf

$W_{SDL} = 5$ psf

$W_L = 50$ psf

Live load reduction → (ASCE 7-10 § 4.7.2).

$K_{LL}A_T \geq 400 ft^2$

$K_{LL} = 1.0$ (table 4-2).

exterior bay:

$A_T = (13.65 + 14.8)(22)$.
$= 625.9$

$L = L_o(.25 + 15/\sqrt{K_{LL}A_T})$
$= 42.5$ psf

interior bay:

$A_T = (13.65 + 14.8)(17.5)$
$= 497.9$

$L = L_o(0.25 + 15/\sqrt{K_{LL}A_T})$
$= 46$ psf

Christopher Cioffi | TECH 3 | PT 2-Way slab design

Section properties

design class: U (ACI 318-11 §18.3.3)

Gross cross sectional properties can be used for uncracked (§18.3.4)

$A = bh = (29.6)(12)(8) = 2842 in^2$

$S = \frac{bh^2}{6} = \frac{(29.6)(12)(8^2)}{6} = 3789 in^3$

set design parameters

allowable stresses: Class U

at time of jacking

$f'c = 3000 psi$

Compression: (ACI §18.4.1) = $0.6f'ci = 1800 psi$

Tension: $3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164 psi$

at service loads

$f'c = 5000 psi$

compression: (ACI §18.4.2 (a and b)) = $0.45f'c = 2250 psi$

Tension: (ACI §18.3.3) $f_t \leq 6\sqrt{f'c} = 6\sqrt{5000} = 424 psi$

Average precompression limit:

P/A = | 125 psi (min) | 300 psi (max) — ACI §18.12.4

Target load balances:

60-80% of DL (self weight)

used 75%

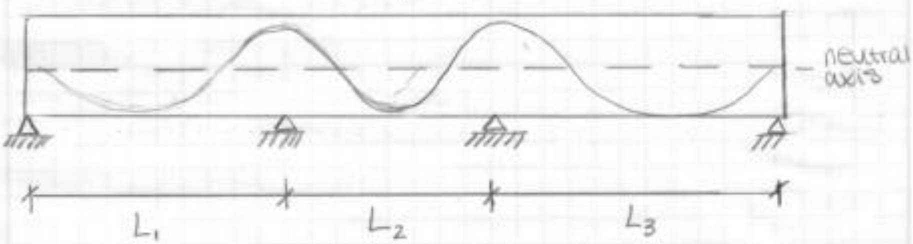
$0.75(W_{DL}) = 0.7(100) = 75 psf$

Cover requirements: (IBC 2012)

unrestrained slabs = 1 1/2" BTM = 3/4" TOP Restrained = 3/4" BTM

Christopher Cioffi TECH3 PT 2-way slab design

parabolic shape tendons



Tendon location	center of gravity location [measured from btm of beam]
exterior support	4.0"
interior support top	7.0"
interior span BTM	1.0"
ENDSPAN BTM	1.75"

$a_{int} = 7 - 1.0 = 6.0"$

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

prestressed force to balance 75% SW Dead load:

end span will govern the max required post-tensioning force

$W_b = 0.75 W_{DL} = 0.75(100)(28.45) = 2133.8$ plf
$= 2.13$ klf

Christopher Cioffi | TECH 3 | PT 2way slab design

force needed in tendons to counteract load in end bay

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

$P = 2.13(22)^2 / (8(3.75/12)) = 412 \text{ kips}$

<u>Check precompression allowance</u>

determine # of tendons

$\# \text{ of tendons} = 412 / (26.6)^{\rightarrow P_{eff}} = 15.5$

use 15 tendons - rounded down - compensated later

actual force for banded tendons:

$P_{actual} = (15 \text{ tendons})(26.6) = 399k$

The balanced load for end span (adjusted)

$w_b = (399/412)(2.13) = 2.06 k/ft$

determine actual precompression stresses

$P_{actual}/A = (399)(1000)/2842 in^2 = 140.4 psi$

Check

140.4 > 125psi min ✓ okay

< 300psi max ✓ okay.

<u>Check interior span force:</u>

$P = (2.13)(17.5^2)/(8(6/12)) = 163 \text{ Kips}$

$163k < 412 \text{ Kips}$

less force is required in the interior span.

For simplicity, force required for end spans will be continued into interior span

Christopher Ciotti: TECH 3 PT 2-Way Slab design

$W_b = (4/12)(8)(6/12) / 17^2$

$= 5.7$ k/ft

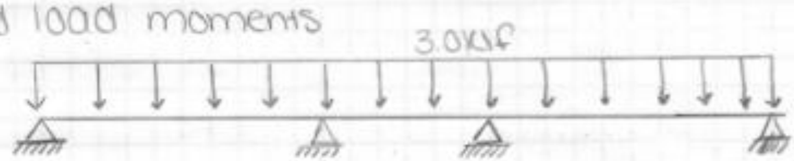
$W_b / W_{DL} < 100\%$ ✓

Interior frame effective prestress force

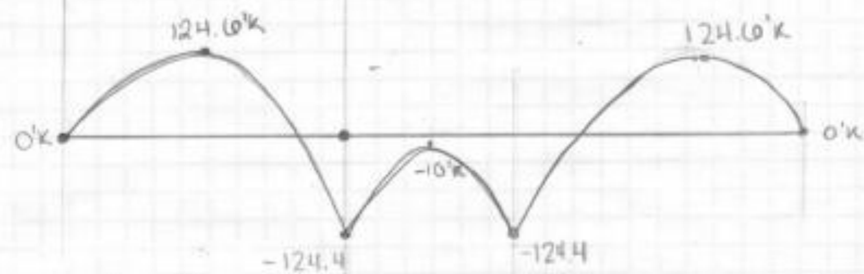
$P_{eff} = 399$k

SLAB STRESSES CHECK:

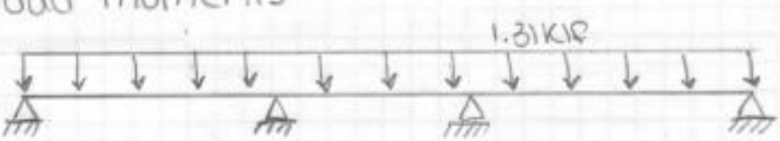
dead load moments



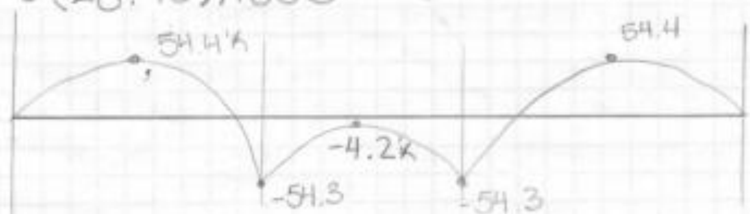
$W_{DL} = (105psf)(28.45)/1000 = 3.0$ KLF



live load moments



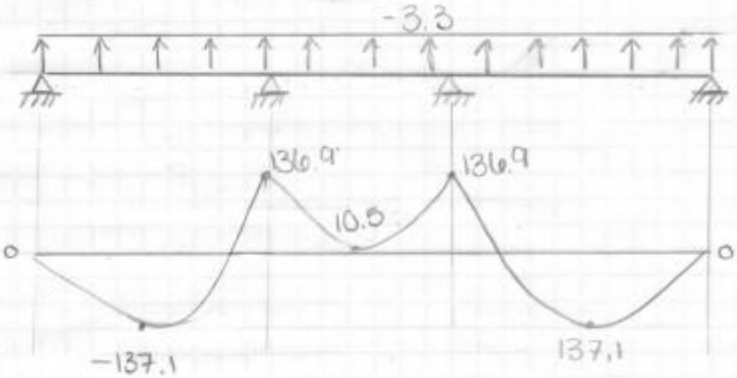
$W_L = 46(28.45)/1000 = 1.31$ KLF



Christopher Cioffi TECH 3 PT- 2way slab design

Total balancing moments Mbal

$W_0 = (2.13 - 2.06 + 5.7)/3 = -3.3$



Stage 1: Stresses immediately after jacking (DL+PT)
(ACI 318-11 § 18.4.1) No live load, self loading at 75%)

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

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

interior span:

$f_{TOP} = (10 - 10.5)(12)(1000)/3789 - 140.4$
$= -142\text{ psi}$ compression $< 0.65 f'c = 1800\text{psi}$
OKAY ✓

$f_{BTM} = (-10 + 10.5)(12)(1000)/3789 - 140.4$
$= -139\text{ psi}$ compression $< 0.65 f'c = 1800\text{psi}$
okay ✓

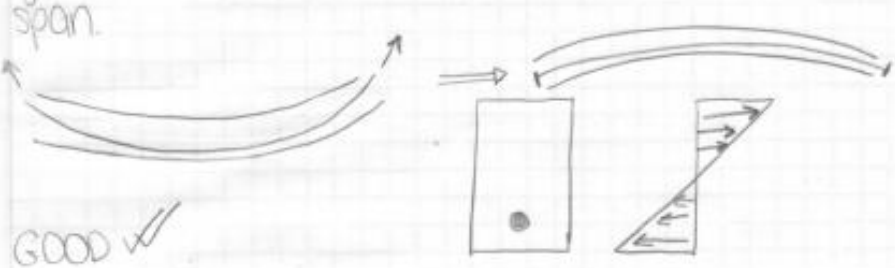
endspan:

$f_{TOP} = (-124.6 + 137.1)(12)(1000)/3789 - 140.4$
$= 101\text{ psi}$ compression $< 0.65 f'c = 1800\text{psi}$
okay ✓

$f_{BTM} = (124.6 - 137.1)(12)(1000)/3789 - 140.4$
$= -178\text{ psi}$ compression $< 0.65 f'c = 1800\text{psi}$
okay ✓

Christopher Cioffi TECH3 PT 2-way slab design

mental check:
- after jacking compression should be greater in btm of beam at middle of span.



GOOD ✓

Support stresses:

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

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

$f_{top} = (124 - 136.9)(12)(1000)/3789 - 140.4$
$= -181.3$ psi compression $< 0.65f'c$
okay ✓

$f_{BTM} = (-124 + 136.9)/3789 - 140.4$
$= -140.3$ psi compression $< 0.65f'c$
okay ✓

Stage 2 Stresses at service loads (dead, self and live). (ACI 318-11 §18.3.3 and §18.4.2)

midspan stresses:

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

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

interior span

$f_{top} = (+10 + 4.2 - 10.5)(12)(1000) / 3789 - 140.4$
$= -129$ psi compression $< 0.45f'c$
okay ✓

$f_{btm} = (-10 - 4.2 + 10.5)(12)(1000)/3789 - 140.4$
$= -152$ psi compression $< 0.65f'c$
OKAY ✓

Christopher Cioffi: TECH 3 PT 2-Way slab design

end span

$f_{top} = (-124.6 - 54.4 + 137.1)(12)(1000)/3789 - 140.4$

$= -273$ psi compression $< 0.45f'c$

okay ✓

$f_{BTM} = (124.6 + 54.4 - 137.1)(12)(1000)/3789 - 140.4$

$= -7.7$ psi compression $< 0.65f'c$

okay

Support Stresses

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

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

$f_{TOP} = (124 + 54.3 - 136.9)(12)(1000)/3789 - 140.4$

$= -9.3$ psi compression $< 0.65f'c$

okay

$f_{BTM} = (-124 - 54.3 + 136.9)(12)(1000)/3789 - 140.4$

$= -271.5$ psi compression $< 0.65f'c$

okay

all stresses are within permissible code limits.

Ultimate Strength:

determine factored moments
the primary PT moments, M_1, vary along the length of the span

$M_1 = P \times e$

*after jacking

$e = 0$ in at exterior support

$e = 3$ in at interior support
neutral axis to the center of tendon

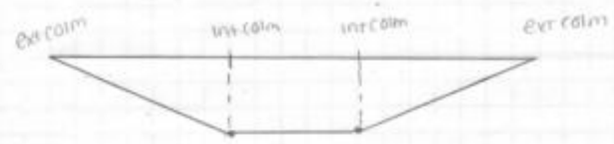
$M_1 = (412)(3.0)/12 = 103$'k

Christopher Cioffi TECH 3 PT 2-way slab design

The secondary post tensioning moments, M_{sec}, vary linearly between supports

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

$= 136.9 - 103 = 33.9$ 'k



The typical load combination for ultimate strength design is

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

at midspan

$= 1.2(124.6) + 1.6(54.4) + 1.0(33.9)$

$= 270.5$ 'k

at support

$= 1.2(-124.4) + 1.6(-54.3) + 1.0(33.9)$

$= -202.3$ 'k

Determine min bonded reinforcement.

negative moment regions:

$A_{s_{min}} = 0.00075 A_{cf}$ (ACI 318-11 §9.3.3)

interior supports:

$A_{cf} = \max.\ (8in)\left(\frac{22 + 17.5}{2}\right)(12)$

$A_{s_{min}} = 0.00075(1896) = 1.4 in^2$

$1.4/0.2 = 7$

7 # 4 bars TOP

Christopher Cioffi | TECH 3 | PT 2-way slab design

exterior supports:

$A_{cf} = (8)(22/2)(12)$

$= 0.00075(1056)$

$= 0.792 in^2$

$0.792/0.2 = 4$ #4 BARS ($A_s = 0.8 in^2$)

must span a min of 1/6 the clear span on each side of support (§18.9.4.2)
at least 4 bars required in each direction (§18.9.3.3)

place top bars within 1.5h away from face of the support on each side (§18.9.3.3)

$= 1.5(8") = 12"$

max bar spacing = 12" (ACI 318-11 §18.9.3.3)

Check min reinforcement if it is sufficient for ultimate strength:

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

d = effective depth

$A_{ps} = 0.153 in^2 (15 \text{ tendons}) = 2.30 in^2$

$f_{ps} = f_{se} + 10{,}000 + (f'_c bd)/300A_{ps}$ for slabs with $l/h > 35$ (ACI 18.7.2)

$= (174{,}000) + 10000 + (5000(28.45)d/(300(2.3)))$

$184000 + 206.2d$

$a = (A_s f_y + A_{ps} f_s)/0.85 f'_c b$

Christopher Cioffi | TECH 3 | PT 2-Way Slab design

at supports

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

$f_{ps} = 184000 + 206.2(7) = 185443.4 psi$

$a = [((1.4)(60) + (2.30)(185)) / 0.85(5)(28.45(12))]$

$= 0.35$

$\Phi M_n = 0.9[(1.4(60)) + 2.30(185)](7'' - ((0.35)/2)] / 12$

$= 260.8 ft\text{-}k$

202.3'k < 260.8 reinforcement for ultimate strength requirements govern

$As_{req} = 1.4 in^2$

interior supports	7 #4
exterior supports	4 #4

midspans

$As_{min} = 0.00075 A_{cf}$

$A_{cf,max} = 8(22)(12) = 2112$

$= 0.00075(2112)$

$= 1.5 in^2$

8 #4 ($A_s = 1.6 in^2$)

$d = 8'' - 1\tfrac{1}{2} - 1/4'' = 6\tfrac{1}{4}''$

$f_{ps} = 184000 + 206.2d$

$= 185288 psi$

$a = [(1.6)(60) + (2.3)(185)] / 0.85(28.45)(12)$

$a = 1.79$

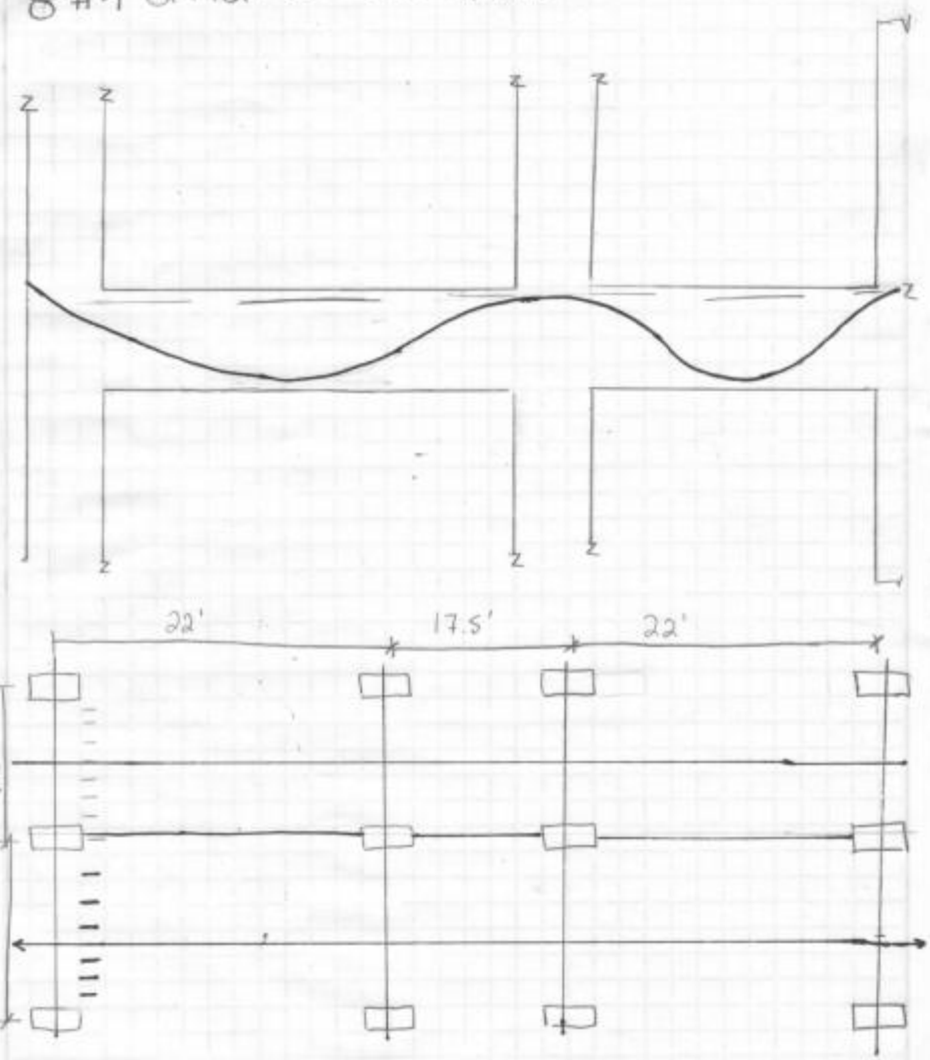
Christopher Cioffi TECH 3 PT 2-way slab design

$\phi M_n = 0.9[(1.6)(60) + (2.3)(185)](7" - (1.79/2))/12$

$= 238.7$ 'k

min reinforcement okay

8 #4 on top at end spans



Alternative System:
Composite Floor System

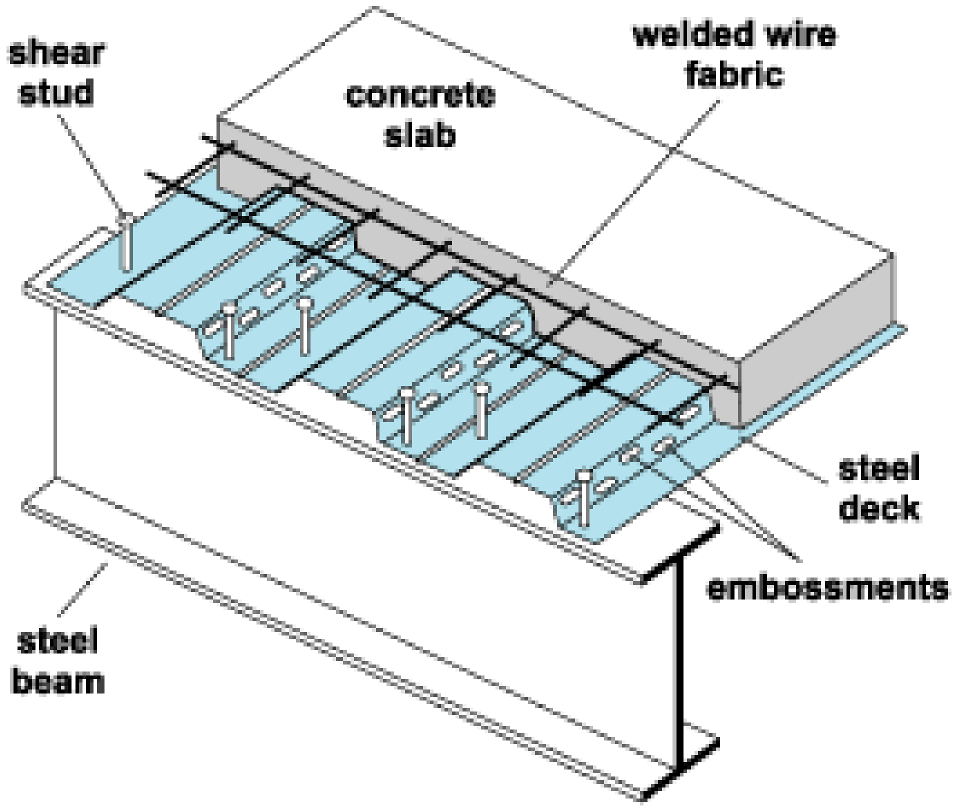
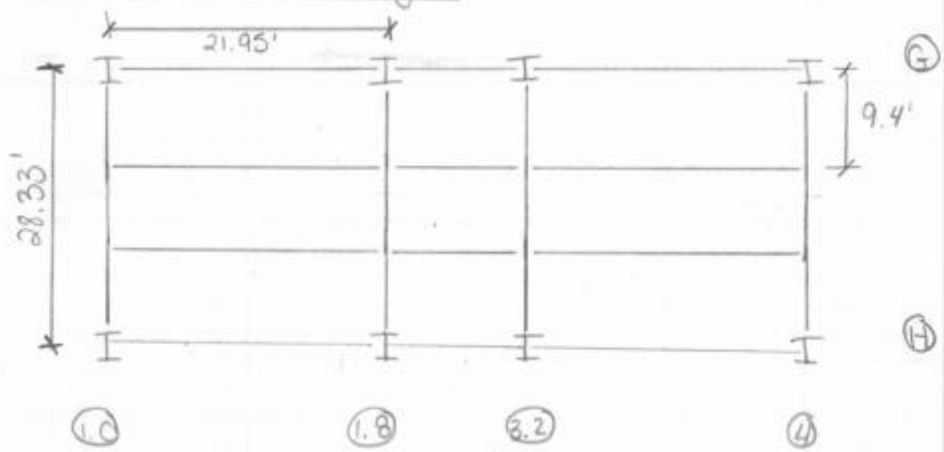


Photo Credit: answers.com

Christopher Cioffi | TECH 3 | Design: Composite floor System

Composite floor design:



28.33/3 = 9.4

W_{LL} = 40 + 10 = 50psf

W_{DL} = 10psf

from vulcraft Manual: TRY

use normal weight concrete

-3span condition w/ 9'5"

-2VLI Chart

max sheet length = 42' > 21.95' ✓okay

use 2VLI 22

3span 9'9" > 9'5" ✓okay

(max construction span).

Super imposed LL = [9'6"] = 94psf

Check:

LL + DL + SW = 50 + 10 + 39 = 99psf ✗ NOT GOOD

TRY 2VLI 20

-3span condition: 11'2" good✓

max sheet length 42' > 21.95' ✓good

Super Imposed LL = 106 at 9'6" ✓okay

99psf < 106psf ✓

Christopher Cioffi | TECH 3 | Composite floor System

Beam design:

$W_u = [1.2(10 + 39) + 1.6(50)]\,9.4' = 1.3\text{ KIP}$

(39 = decking)

$V_u = \frac{(1.3)(21.95)}{2} = 14.27^k$

$M_u = \frac{w\ell^2}{8} = \frac{(1.3)(21.95^2)}{8} = 78.3'k$

TRY a W10 x19 → $\phi M_{px} = 81.0'k$

W10 x19 information:

$A_s = 5.62\text{ in}^2$ $\quad I_x = 96.3\text{ in}^4$

$d = 10.2''$ $\quad S_x = 18.8\text{ in}^3$

$t_w = 0.3\text{ in}$ $\quad Z_x = 21.6\text{ in}^3$

$b_f = 5.77\text{ in}$

$b_{eff} = \min \begin{cases} \frac{(21.95)(12)}{8} = 33'' \rightarrow \text{controls} \\ \frac{(9.4)(12)}{2} = 57'' \end{cases}$

$b_{eff} = 66''$

$V'_c = 0.85(5)(66)(2'') = 561K$

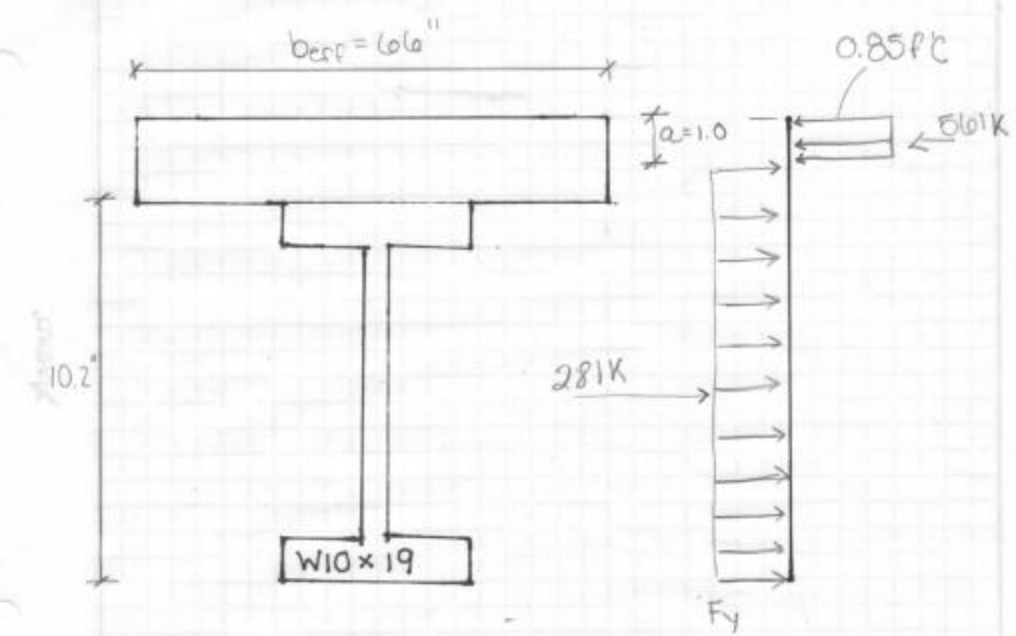
$V'_s = (5.62)(50) = 281k$

$V'_c > V'_s$ PNA is in the slab *

$a = \frac{(5.62)(50)}{(0.85)(5)(66)} = 1.0''$

$a_{actual} < a_{assumed}$

CHRISTOPHER CIOFFI | TECH 3 | Composite floor system



<u>Check wet concrete deflection:</u>

$W_{wc} = 39psf(9.4) + 19plf = 0.386 klf$

Beam weight

$\Delta_{wc} = \frac{5wL^4}{384EI} = \frac{5(0.386)(21.95^4)(1728)}{384(29000)(96.3)} = 0.722''$

$\Delta_{wc\,max} = \frac{\ell}{240} = \frac{21.95(12)}{240} = 1.09''$

$\Delta_{wc\,max} = 1.09'' > \Delta_{wc} = 0.722''$ good ✓

<u>Check unshored strength</u>

$W_{LL} = 50(9.4) = 0.47 klf$

$W_{DL} = (39)(9.4) + 19 = 0.386 klf$

Christopher Goffi | TECH 3 | Composite floor system

$W_u = 1.2(0.386) + 1.6(0.47) = 1.22$

$M_u = \frac{(1.22)(21.95^2)}{8} = 73.5'k < \Phi M_p$

$73.5'k < 81'k$ good ✓✓

(use (4) W 10×12)

$\Phi M_n = \Phi(A_s F_y)\left[d/2 + t - \frac{a}{2}\right]$

$= 0.9(5.62)(50)\left[10.2/2 - 4 - \frac{1.0}{2}\right]$

$= 151.74$

$81'k < 151.74'k$ good ✓✓

$\Phi V_n = (0.9)(0.6) F_y d t_w$

$= 0.9(0.6)(50)(10.2)(0.25)$

$= 68.85k$

$68.85K > 14.27K$ good ✓✓

design of shear studs:

1 stud/rib from Q_n table = 17.2K/stud

⊥ deck

$V's = 281K$ ← use min to gain full strength

$V'_c = 561K$

$n = \frac{\Sigma Q_n}{Q_n} = \frac{281}{17.2} = 16.34 = 17$ studs

Check live load deflection:

$W_{LL} = 50(9.4) = .470$ $\qquad I_{LB(y=3.5)} = 304$

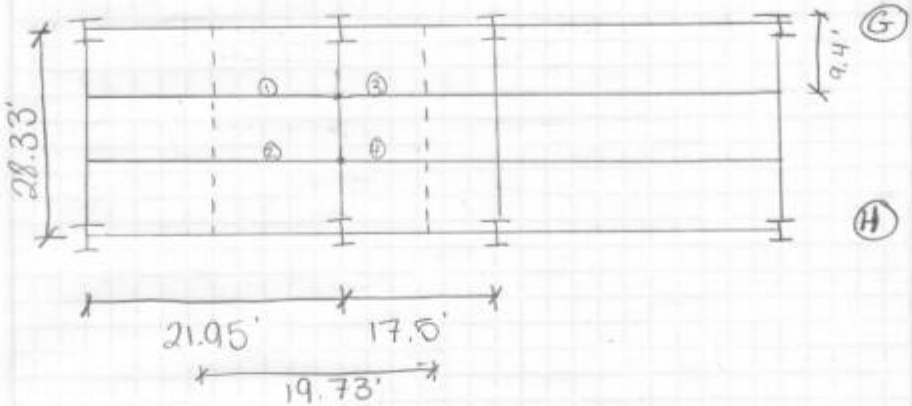
$\Delta_{LL} = \frac{5(0.470)(21.95^4)(1728)}{(384)(29000)(304)} = 0.278''$

Christopher Cioffi TECH 3 Composite Floor System

$$\frac{\ell}{360} = \frac{21.95(12)}{360} = 0.731$$

$0.278 < 0.731$ good ✓

GIRDER DESIGN:



LOADS:

$W_{DL} = (10 + 39)(9.4) = 0.461\ k/ft$

$W_{LL} = (50)(9.4) = .47\ k/ft$

① and ②
$(0.461)(1.2)(10.98) + (0.47)(1.6)(10.98) = 14.33k$

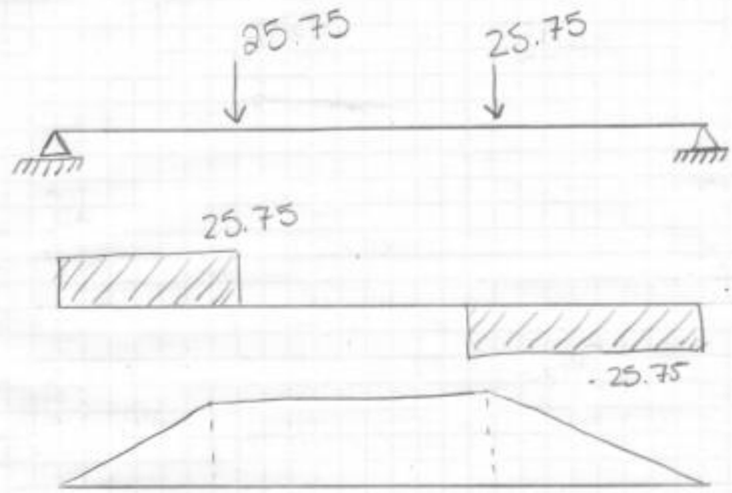
③ and ④
$(0.461)(1.2)(8.75) + (0.47)(1.6)(8.75) = 11.42k$

$P_u = 37k.$

$b_{eff} = \left| \begin{array}{l} \frac{28.33(12)}{8} = 42" \longrightarrow \text{controls} \\ \frac{19.73(12)}{2} = 118" \end{array} \right.$

$b_{eff} = 2(42) = 84"$

Christopher Cioffi | TECH 3 | Composite floor system



$M = 25.7k(9.4) = 242'k$ assume $a = 2$

TRY W21 x 48

$V'_c = 0.85(5)(84)(2) = 714k.$

$V'_s = (50)(14.1) = 705$

$V'_c > V'_s$ PNA is in slab

$a = \frac{(14.1)(50)}{0.85(5)(84)} = 1.97''$ actual < assumed good ✓

$\phi M_n = (0.9)(14.1)(50)\left[\frac{20.6}{2} + 4 - \frac{1.97}{2}\right] / 12$

$= 704\ 'k$

$\phi M_n > M_n$ good ✓

$y = 3.5$

$I_{lb} = 2210$

W_{LL} deflection

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{11.42(28.33)^3(1728)}{28(29000)(2210)} = 0.25''$

$\frac{\ell}{360} = \frac{28.33 \times 12}{360} = 0.9 > 0.25$ good ✓✓

Christopher Cioffi | TECH 3 | Composite floor System

Check unshored:

$W_{DL} = (48)(9.4)(19.73) = 8.9$

$W_{LL} = (50)(9.4)(19.73) = 9.3$

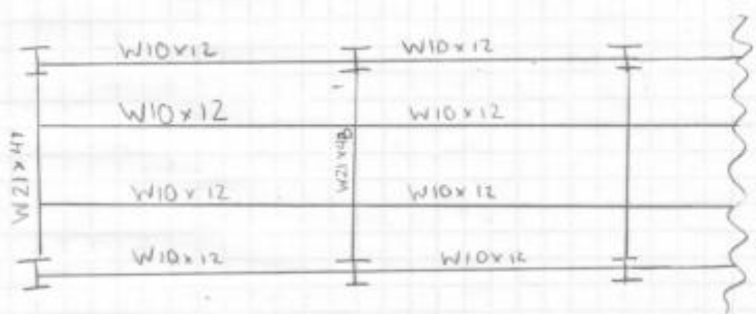
$P = 18.2$

$\frac{PL}{4} = \frac{(18.2)(28.33)}{4} = 128'k < 274'k$ good

Shear studs:

$n = \frac{705}{18.3} = 38 \times 2 = 76$ studs/girder

use W21×48



Cost Analysis and Comparison of the Existing and New Design

Comparison Conditions	Two-Way Slab System with Shear Caps	One-Way Slab with Reinforced Beams	Post-Tension Concrete Two-Way Slab	Composite Decking System with Steel Framing
Slab Self Weight (psf)	100	138	60	39
Slab Depth (Inches)	8	11	8	4
Bay Area (sft)	3500	3500	3500	3500
Weight Per Typ Bay (Kips)	354	484	210	147
Fire rating	2 Hours	2 Hours	2 Hours	2 Hours
Cost Per Sf	$17.05	$19.65	$17.65	$14.43
Constructability	N/A	Moderate	Moderate	Easy

Christopher Cioffi | TECH 3 | COST Analysis

COMPARISON OF SYSTEMS Calculations

PT 2-Way

$$\frac{100 \text{ lbs}}{8'' \times 8''} = \frac{x}{12'' \times 12''}$$

$$\frac{100}{0.4'} = \frac{x}{1'}$$

40 lbs/ft + tendons self weight

$40 + 20 \approx 60$ psf

Weight analysis: [per 1 Typical Bay]

2-way slab →

$\left(\frac{8}{12}\right)(150) = 100$ psf

Shear caps →

$\left(\frac{4}{12}\right)(150) = 50$ psf

Total Bay area $= (27.3 + 29.6)(22 + 17.5 + 22)$

$= 3500$ sf

Shear cap area $= (10.5\text{sf} \times 2) + \frac{1}{2}(10.5 \times 10)$

$= 73.5$ sf

$W = 3500(100) + 50(73.5)$

$= 353.7$ Kips

Christopher Ciofti | TECH 3 | Cost Analysis

1 way slab →

$slab = \left(\frac{11}{12}\right)(150) = 137.5 psf = 138 psf$

$beam = \left(\frac{27}{12}\right)(150) = 337.5 psf$

$3500(138) = 483 Kips$

$beams = 4\left(\frac{19''}{12}\right)(27.3) = 0.17$

$+ 4\left(\frac{19}{12}\right)(27.3) = 0.18$

PT 2way slab:

$3500 \times 60 = 210 Kips$

composite deck:

$deck = (39)(3500) = 136.5K$

$Beams = (8)(19)(21.95) = 3.3K$

$(4)(19)(17.5) = 1.3K$

$(4)(48)(28.33) = 5.5K$

$= 147 Kips.$

Christopher Cioffi | TECH 3 | Cost Analysis

Comparison of cost:

Two way slab w/ drop panels:
RS MEAN ITEM 5200
Bay size ≈ 25'×30'
COST = 17.05 /sft

---

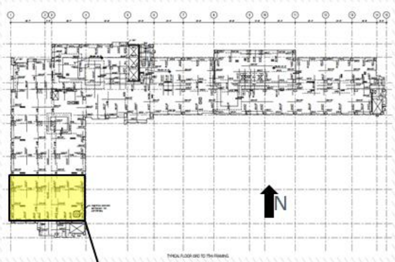
one way slab:
RS MEAN ITEM 7600
Bay size ≈ 30×35
COST = $ 19.65 /sft

Composite deck:
RS MEAN ITEM 3400
Bay size ≈ 25'×30'
cost = 17.65 /sft

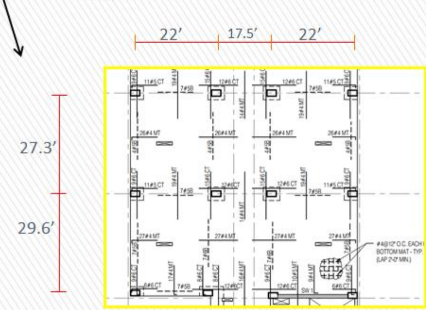
PT 2-way slab
RS MEAN
BAY size ≈ 25'×30
COST = $14.43

POWER POINT PRESENTATION

TYPICAL BAY STUDIED:

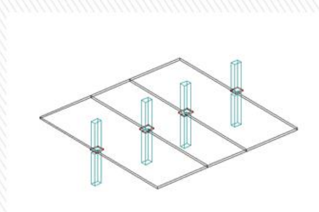


- 3rd to 7th Floors
- Apartments
- Egress



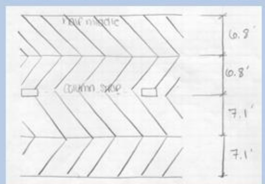
Existing System:

- 2 way flat plate slab with shear caps
- 8” Slab
- 4” shear caps around columns extending 6” on each side
- #4 Rebar Bottom Mat spaced 21” o.c. both directions

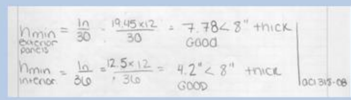


Two-Way Slab Spot Check:

Distribution of moments between column and middle strips:

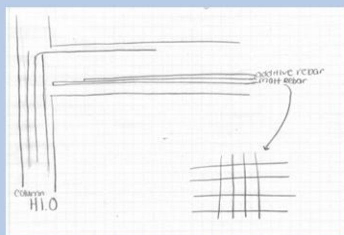


Checked Minimum Slab Thickness ACI 318-08



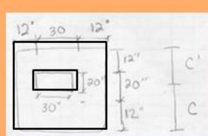
Distribution of total negative and positive moments:

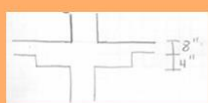
Location	Column Stip Moments	Middle Strip Moments
Interior	-174.13'K	-74.54'K
Middle	110.73'K	73.82'K
Exterior	-92.27'K	0'K



Punching Shear Checks on Shear Caps

Dimension #1:



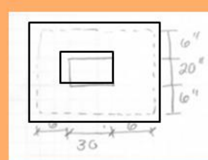


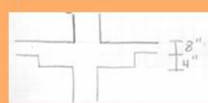
Vc verses Φ Vc

262.2k > 135.4k

Slab is okay for punching shear

Dimension # 2:





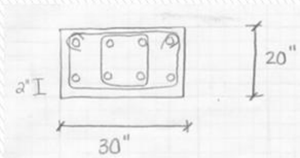
Vc verses Φ Vc

227.62k > 137.4k

Slab is okay for punching shear

Interior Column and Exterior Column Gravity Check:

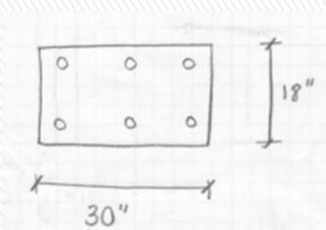
Interior (H1.8)



Location	Pn(K)	ΦPn(K)	Pass/Fail
4th Floor	839.2	1296.3	Pass
2nd Floor	1244.7	1557.92	Pass
1st Floor	1606	2531.7	Pass
Ground	1901.3	2531.7	Pass

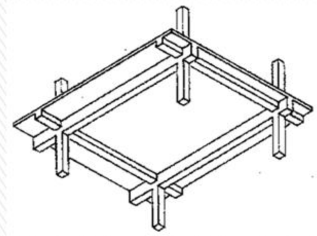
Interior Column and Exterior Column Gravity Check:

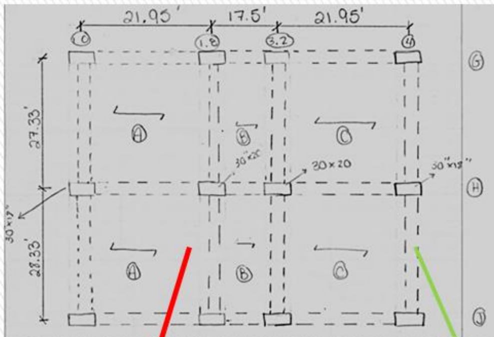
Exterior Column:

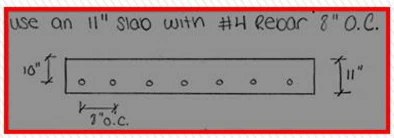


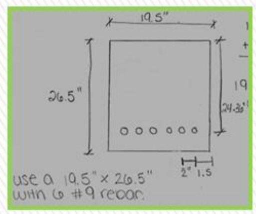
Location	Pn(K)	ΦPn(K)	Pass/Fail
4th Floor	589	815.5	Pass
2nd Floor	888	1069.1	Pass
1st Floor	1105	1069	Pass
Ground	1478	1069.1	Fail

One Way Slab System With Reinforced Beams:

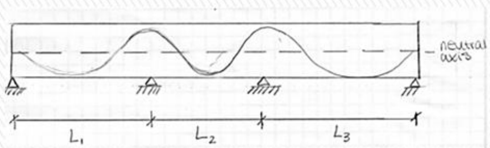








Post Tensioned Two Way Slab:



15 Tendons Per Bay

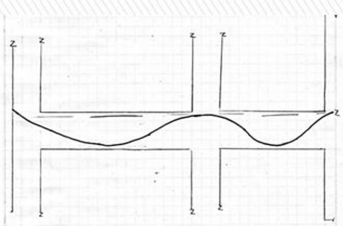
Interior Supports: 7#4 bars top

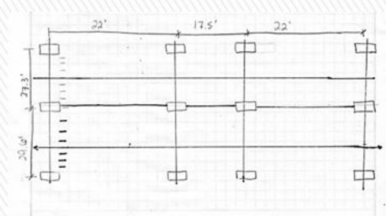
Exterior Supports:

4#4 bars top

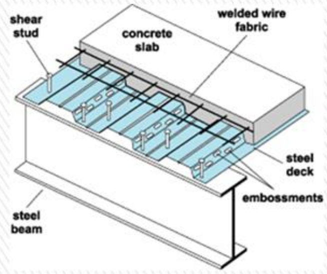
Middle:

8#4 Bars top

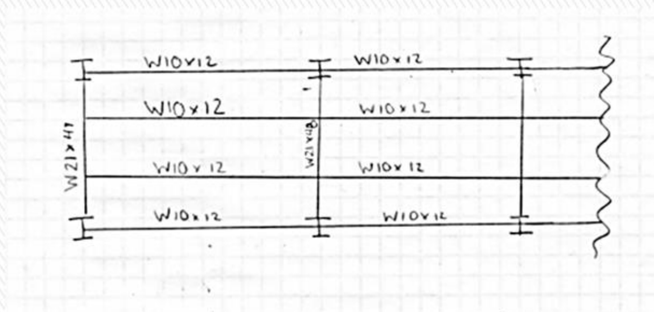




Composite Floor System:



Decking: Vulcraft 2VLI
20
3 Span 11'2"
SI LL= 106psf



Cost Analysis:

Comparison Conditions	Two-Way Slab System with Shear Caps	One-Way Slab with Reinforced Beams	Post-Tension Concrete Two-Way Slab	Composite Decking System with Steel Framing
Slab Self Weight (psf)	100	138	60	39
Slab Depth (Inches)	8	11	8	4
Bay Area (sft)	3500	3500	3500	3500
Weight Per Typ Bay (Kips)	354	484	210	147
Fire rating	2 Hours	2 Hours	2 Hours	2 Hours
Cost Per Sf	$17.05	$19.65	$17.65	$14.43
Constructability	N/A	Moderate	Moderate	Easy

Most Feasible Design:

Composite Decking System with Steel Framing

- *Maximizes floor to floor height*
- *Minimum cost per square foot*
- *Easy to construct*
- *Takes the least amount of time to construct (time management)*
- *Weight less- less steel used, smaller column sizes*