Letter of Transmittal

October 17, 2014

Heather Sustersic
had132@psu.edu
Enclosed: Technical Report 3

Dear Professor Sustersic,

The following technical report was written to fulfill the requirements of AE 481W- Structural Technical Report 3.

This report includes a structural analysis of the existing floor system in the Corporate Headquarters. The analysis includes an evaluation of a typical floor bay's framing under gravity loads, as well as an evaluation of both an interior and exterior column under the same gravity loads.

Additionally, Technical Report 3 includes the design and analysis of three alternate framing options for the bay examined previously. The calculations were performed to size members as well as check strength and deflection. These systems will be considered as potential options for the structural redesign to be performed next semester.

Thank you for taking the time to review this report; I look forward to discussing it with you in the future.

Sincerely,

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CORPORATE HEADQUARTERS

Great Lakes Region, U.S.A.

TECHNICAL REPORT 3

M. JULIA HAVERTY
STRUCTURAL OPTION
ADVISOR: H. SUSTERSIC
17 OCTOBER 2014

Table of Contents

Executive Summary

The Corporate Headquarters, located in the Great Lakes Region of the United States, is a new 5 story office and retail space designed to serve as new home base for an established and successful US based company. The building's architecture was designed to mirror its surrounding buildings, namely, the newer retail area situated directly to the north of the building. In keeping with that building style, the Corporate Headquarters features a façade of glass and face brick, construction crews broke ground on the roughly 660,000 SF building in August 2014 and have a projected completion date of Spring 2016.

A challenge in the design of the Corporate Headquarters is the poor existing soil conditions on part of the site. To remedy this problem, aggregate piers will be pushed down below foundation level for the column spread footings and piers to sit upon. In addition to the spread footings, a large portion of the building features slab on grade and a few grade beams.

The floor system in floors 2-5 is a composite floor framing structure consisting of metal deck on top of steel wide flange members. Average bays are rectangular and have slight variation in size although average sizes are around 38' and beams typically span 40'. The primary lateral system of the building is HSS braced frames near the building's core.

The primary loading conditions considered in the design of this structure were live loads, dead loads, snow loads, wind loads, seismic loads, and soil loads. To consider these loading conditions, the 2011 Ohio Building Code was set as primary design criteria. 2011 Ohio Building Code adopts IBC 2009, which references ASCE 7-05.

The following report will provide further information on these topics.

Due to security reasons, location maps are not currently permitted. Further consultation with the owner to follow for subsequent technical reports.

Site Plan and Location

Building Location: Great Lakes Region, U.S.A.

-exact location map not permitted

Site Map

Documents Used to Create This Report

Ohio Building Code 2011

-incorporates IBC 2009

American Society of Civil Engineers

-ASCE 7-05: Minimum Design Loads for Buildings

Corporate Headquarters

-Construction Documents

-Technical Specifications

Vulcraft Deck Catalog

-product manual

Boise- Cascase

-Weight of Building Materials Technical Note

American Concrete Institute

-ACI-318-11: Building Code Requirements for Structural Concrete

Gravity Load Calculations

Gravity Spot Checks

Structural Redesign 2: One-Way Concrete Slab

System Comparisons

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Cost Comparisons

• all costs are approximate values calculated using section B of RS means, using the 2002 square foot costs I created a relative cost analysis

Existing System (Composite Steel)

W beams & Girders

~40'x40' Bays, ~75 psf super imposed load ≈ $15.00/SF

Non-Composite Steel

W beams & Girders

~40x40' Bays, ~$17.45/SF

One-Way Concrete Slab, Cast in Place

Slab Span = 38' → $13.00/SF

Two-Way Concrete Slab, cast in place

~40x40' bays, ~$15.60/SF

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System Weights

Composite Steel

Bay Size $38' \times 40' = 1520$ SF

- composite deck = 39 psf

$39 \times 1520 = 59.28^{k}$

- beams: (4) W24x55 = $4(55)(40') = 8.8^{k}$
- girders: (2) W24x68 = $(2)(68)(38') = 5.168^{k}$
- shear studs: $[(4 \times 26) + (2 \times 114)]$ 10 lb/stud = 3.32^{k}

Total weight = $59.28 + 8.8 + 5.168 + 3.32 = 76.568^{k}$

$\frac{76.568}{1520} \times 1000 = 50.37$ psf

50.37 psf

Non-Composite Steel

Bay Size: $38' \times 40' = 1520$ SF

- same composite deck = 39 psf $\rightarrow 59.28^{k}$

(4 beams)(68 plf)(40') = 10.88^{k}

- girders: $(38')(183) + (38)(149) = 12.62^{k}$

Total = $59.28 + 10.88 + 12.62 = 82.78^{k}$

$\frac{82.78}{1520} = 54.46$ psf

54.46 psf

M. Julia Haverty | System Comparison | Technical Report 3

2

One-Way Concrete Slab

38' × 40' Bay = 1520 SF

115 pcf concrete × 19" slab

$(38 \times 40) \times 115 \times 19/12 = 276\ k$

$\frac{276}{1520} = 182\ psf$

182 psf

Two Way Concrete Slab

38' × 40' Bay = 1520 SF

145 pcf concrete × 15" slab

$1520 \times 145 \times (15/12) = 275.5\ k$

$\frac{275.5}{1520} = 181.25\ psf$

181.25 psf

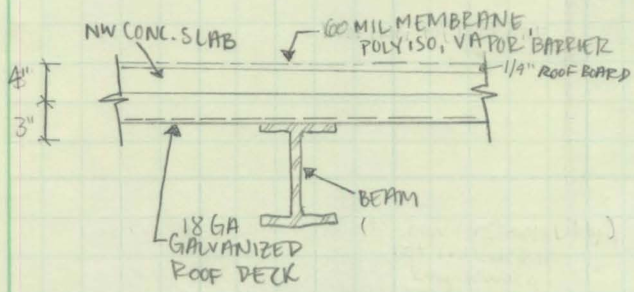
M. Julia Haverty				
Technical Report 3				
System Comparisons				
	Gravity Floor Systems			
	Composite Steel	**Non-Composite Steel**	**One-Way Concrete Slab**	**Two-Way Concrete Slab**
Architectural Considerations				
Maximum Depth	29"	45"	19"	15"
Fire Protection Needed	yes	yes	no	no
2 hr fire rating achieved?	yes	yes	yes	yes
System Statistics				
Comparative Cost to Existing	-	higher	lower	relatively similar
System Weight	50.37 PSF	54.46 PSF	182 PSF	181.25 PSF
durability	fair	fair	good	good
Future Design Considerations				
Concrete Shear Walls	-	yes	yes	yes
Steel Moment Frame	-	yes	no	no
Steel Braced Frame	yes	yes	no	no
Advantages	Max floor to ceiling height, light weight	relatively light, many lateral system choices	not very expensive, no fire protection needed	similar price to existing, no fire protection necessary
Disadvantages	requires fire protection, vibrations may be an issue	expensive, requires fire protection, very deep	Heavy system	Heavy system
Viable Option	N/A	Yes	No	Yes

GRAVITY LOAD CALCULATIONS

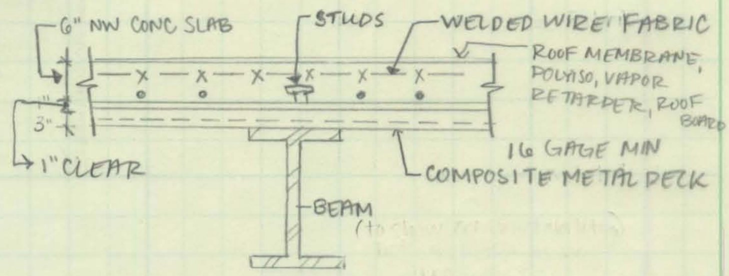
Georgia Pacific Product Spec

Typical Roof Bay Loading

Roof Construction Cross Sections



TYPE R-1 ROOF CONSTRUCTION



TYPE R-2 ROOF CONSTRUCTION

Roof R-1 Dead Load

adhered membrane = 1 PSF
1/4" DENSDECK ROOF BOARD = 1.2 PSF
polyisocyanurate roof insulation = 3 PSF
vapor retarder = 1 psf
4" Concrete slab = 150 × (4/12) = 50 psf
18 GA galvanized metal deck = 2.9 psf
Ceilings = 5 psf
MEP = 15 psf
Sprinklers = 3 psf
Framing = 10 psf

Roof R-1 Dead Load = 92.1 = 92 psf

Roof R-2 Dead Load

adhered membrane = 1 PSF
1/4" Densdeck Roof Board = 1.2 PSF
Polyisocyanurate roof insulation = 3 PSF
Vapor retarder = 1 psf
6" concrete slab = 150 pcf × (6/12) = 75 psf
16 GA composite metal deck = 2.2 psf
ceilings = 5 psf
MEP = 15 psf
sprinklers = 3 psf
Framing = 10 psf

Roof R-2 Dead Load = 116.4 psf

- though the majority of the roof is R-1 construction, enough typical bays are constructed in the R-2 style that I felt it should be included
  - approximately 1/3 of the roof is R-2

Roof Live Loading

ASCE 7-05 - Table 4-1

$L_r = 20$ psf

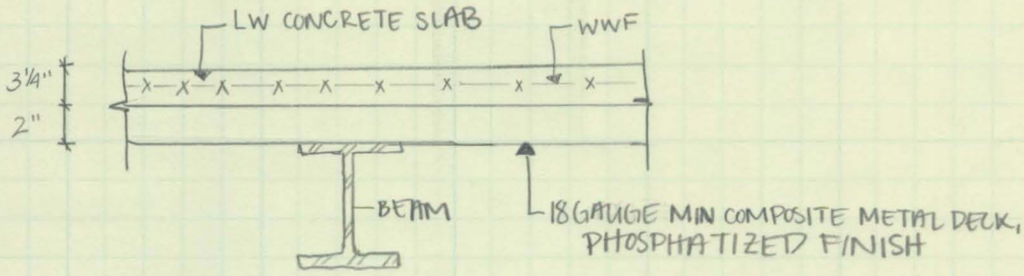
Structural Drawings - Sheet S-001

$L_r = 25$ psf

* I believe that the code value was increased due to the rooftop mechanical units and the frequency with which they may be serviced.

Typical Floor Bay Loading

Floor construction Cross Section



- Concrete slab appears to be an architectural feature and has no additional topping.

Floor Dead Load

3¼" Concrete Slab = $(115)\left(\frac{3.25}{12}\right) = 31.15$ psf ~
18 Gauge Composite metal deck = 2.8 ~ 3 psf
Ceiling = 5 psf
MEP = 10 psf
Sprinklers = 3 psf
Framing = 10 psf

Typical Floor Dead Load = 62 psf

Floor Live Load

From ASCE 7-05 Table 4-1 & § 4.2.2

Offices - 50 psf
Partitions - 15 psf
65 psf

Lobby "Public Area" - 100 psf

These live load values match those listed on S-001 of the structural drawings.

Typical Floor Bay Live Load = 65 psf

Courtyard Loading

From S-001,

	Courtyard Grass Area	Courtyard Tree Area
Dead	201 psf	441 psf
Live	100 psf	100 psf

Non-Typical Loading Conditions

Non-Typical Dead Load Values

- All Floor and Roof types have dead load values listed on sheet S-001 that match ASCE 7-05 typical values.

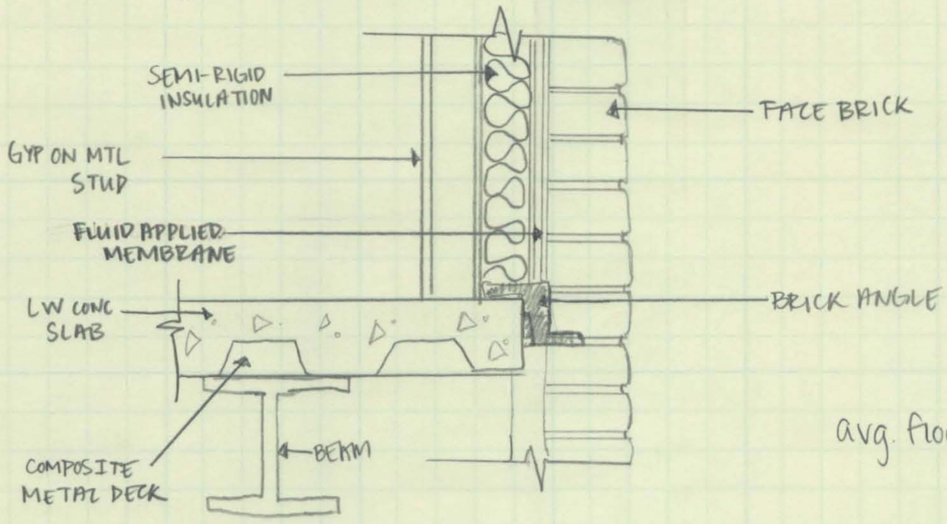
- Courtyard Tree area Soil dead load = 360 psf
  - the courtyard grass area lists a soil load of 120 psf on S-001 while the tree area lists 360 psf dead load
  - this value is large due to the trees taking root in the courtyard

Non-Typical Live Load Values

Load	Design Value	ASCE 7-05 value	Reasoning
Kitchen Area/Fridge	150 psf	125 psf	The higher load is due to heavy foot traffic in this area during lunch and the mobility of the equipment in the area, including future appliances
Typical Roof	25 psf	20 psf	As stated in roof live loading section, I believe this was done due to more frequent access to the RTU's.

Typical Exterior Wall Load

Typical Cavity Wall Section



avg. floor height ~ 16' 6"

Gravity Load Path – Wall System

The weight of the cavity wall system is supported by the brick relief angle and the concrete floor slab. The weight of the face brick is taken by the angle, which then sends it to the floor system. The floor system puts the weight of the brick and cavity wall on the edge beams, which send it to the transfer girders. Those girders eventually meet one of the braced frames, which in turn send the load down to the foundation system. and into the soil.

Cavity Wall Dead Load

Brick veneer: 39 psf × (16.5') = 643.5 plf
semi-rigid insulation:
5/8" gypboard on 3 5/8" metal stud } 12 psf × 16.5' = 198 plf
fluid applied membrane: 2 psf × 16.5' = 33 plf

Total Cavity Wall Dead Load = 847.5 = 848 plf

SNOW LOAD CALCULATIONS

Flat Roof Snow Load, p_f

$p_f = 0.7 C_e C_t I p_g$ (ASCE 7-05 Eq. 7-1)

From S-001:

$p_g = 20$ psf
occupancy category II
$I = 1.1$
$C_e = 1.0$
$C_t = 1.1$
exposure B

$p_f = 0.7(1.0)(1.1)(1.1)(20)$ $p_f = 16.94 \sim 17$ psf $\underline{p_f = 17 \text{ psf}}$

Snow Drift

- calculated for drift from mechanical penthouse roof
- windward snow drift

$h_b = \frac{p_s}{\gamma}$ in this case, $p_s = p_f = 17$ psf

$\gamma = 0.13 p_g + 14$ but cannot exceed 30 pcf
$= 0.13(20) + 14$
$\gamma = 16.6$

$h_b = \frac{17 \text{ psf}}{16.6 \text{ pcf}}$ $h_b = 1.02$ ft $h_c / h_b > 0.2 \rightarrow$ drift loads must be calculated

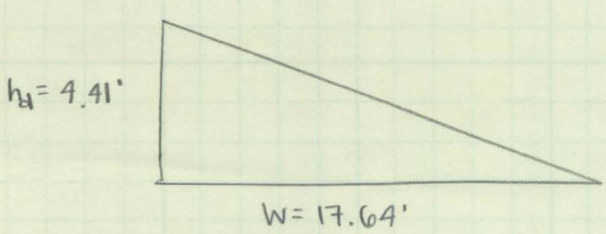
$h_c = 15'-1.5''$

$l_u = 394' \rightarrow$ from Figure 7-9, $h_d \sim 5.5$ ft

$h_d = 0.43 \sqrt[3]{l_u} \sqrt[4]{p_g + 10} - 1.5 = 0.43 \sqrt[3]{394} \sqrt[4]{20+10} - 1.5$
$h_d = 5.88'$
$3/4\, h_d = 4.41'$

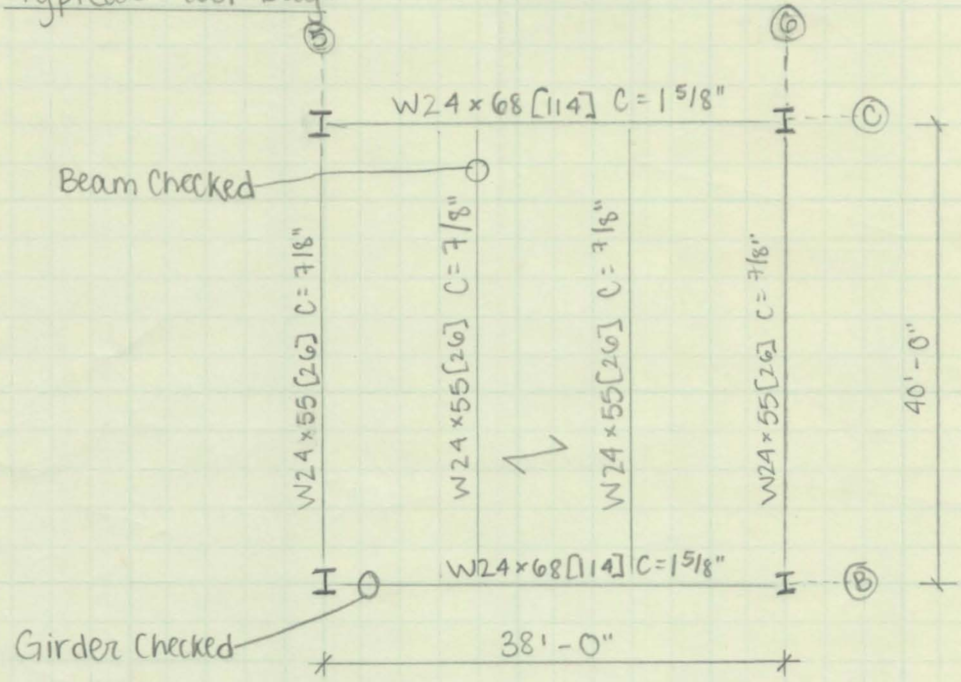
$h_d > h_b$ $h_d < h_c \rightarrow w = 4h_d$ $w = 4(4.41)$ $w = 17.64'$

Drift Density $p_d = h_d \gamma$ $p_d = 4.41(16.6)$ $\underline{p_d = 73.21 \text{ psf}}$



Gravity Spot Check:

Typical Floor Bay:



Note: This bay was chosen since bays of this shape and size, as well as members of these sizes, are found in the majority of the building

This particular bay is on the third floor of the building in segment A. As is customary on the upper floors of the building, the bay is in an open office area and will be subjected to changing use type and occupancy. Knowing what the client has envisioned for this free-flowing changeable space, I believe it would be appropriate to deem this space a public area instead of a typical office. Additionally, it is in an area of the building that would recieve heavier foot trafic than a bay against an exterior wall.

From S-001,

	Office	public area
DL	61	61
LL	65	100
TL	126	161

Decking Check

3 1/4" LW concrete slab on 2" 18 guage composite metal deck w/ phosphatized finish

per Vulcraft Catalog: Total Slab Depth = 5 1/4"

→ using 1.5VL18 w/ t = 3.25, Vulcraft specifies a 4.75" total slab depth

Shoring Check

worst case 3 span condition: 12'-8", 12'-8", 12'-8"

3 Span Condition – Max unshored clear span

10'-5" < 12'-8" X

2 Span Condition – Max unshored Clear Span

10'-1" < 12'-8" X

1 Span Condition – Max unshored Clear Span

8'-2" < 12'-8" X

- Shoring required for all 3 span conditions
- From S-001 "provide shoring where required", so shoring would be used in this typical bay

Strength Check

Live Load = 100 psf

Superimposed Dead Load = 18 psf
Ceiling = 5 psf
MEP = 10 psf
Sprinklers = 3 psf

$W_{LL} + W_{misc\ DL} = 100 + 18 = 118$ PSF

Clear Span 12'-8"
From Vulcraf, max superimposed Live Load = 138 psf

138 > 118 → Deck sufficient to carry loads

Decking deemed adequate ✓

Beam Check

W24×55 [26] camber = 7/8" span = 40'-0", 12'-8" spacing

W_{DL} = 61 psf (calculated value = 62 psf, S-001 lists 61 psf)
W_{LL} = 100 psf

$W_u = 1.2D + 1.6L = 1.2(61) + 1.6(100)$

$W_u = 73.2 + 160 = 232.2\,(12.67')\,(1/1000) = 2.95$ klf

$M_u = \frac{wl^2}{8} = \frac{2.95(40^2)}{8}$ $M_u = 590.77$ k-ft ~591 k-ft

26 studs in 20'

40'-0"

26 studs | 26 studs

6 ribs with 2 studs
14 ribs with 1 stud

- deck perpendicular
- weak studs to be conservative
- 3/4" φ studs (S-001)
- f'c = 3500 psi (S-001)
- LW concrete

using table 3-21

interpolated value

$\Sigma Q_n = 6(14.6) + 14(17.15)$ $\Sigma Q_n = 327.7 \sim 328$ K

2 studs/rib   1 stud/rib

$Ts_{max} = A_s F_y$ F_y = 50 ksi (A992 – specified on S-001)
A_s = 16.2 in² (table 1.1)

$= 16.2(50)$

$Ts_{max} = 810$ K

Ccmax:

$Cc_{max} = 0.85 f'c\, b_{eff}\, t$ $= .85(3500)(120)(5)/1000$ $Cc_{max} = 1785^K$

$b_{eff} = 2 \times \min \begin{cases} span/8 = 40'/8 = 5' \leftarrow \text{controls} \\ 1/2 \text{ dist to adjacent beam} = \frac{1}{2}(12.667) = 6.33' \end{cases}$

$b_{eff} = 2 \times 5 = 10' = 120"$
t = ~5"
f'c = 3500 psi

$\Sigma Q_n = 328^k < C_{cmax} = 1785^k$

$\Sigma Q_n = 328^k < T_{smax} = 810^k$

Partially composite beams ✓
- matches structural dwgs

$a = \frac{\Sigma Q_n}{.85 f'c\, b_{eff}} = \frac{328}{.85(3.5)(120)}$ $a \doteq .92$

$y_2 = t - a/2 = 5 - (.92/2)$ $y_2 = 4.54"$

using table 3-19

W24×55, $\Sigma Q_n = 328^k$, $y_2 = 4.54"$, LRFD

need to interpolate:

	4.5	4.54	5
329	803	804.04	816
328		803.32	
203	713	713.56	720

$\phi M_n = 803$ k-ft

$\boxed{\phi M_n = 803 \text{ k-ft} > M_u = 591 \text{ k-ft}}$
Beam Strength Met ✓

Check unshored strength

total deck weight — beam weight

$W_u = [1.4(37)(12.67') + 1.4(55)]/1000$ $W_u = .733$ klf

OR

$W_u = [1.2[(37)(12.67')] + 1.6[\underbrace{(20)}_{\text{construction LL}}(12.67')]]/1000$ $W_u = 0.97$ klf ← controls

$M_u = \frac{W_u l^2}{8} = \frac{0.97(40)^2}{8} = 194$ k-ft

using table 3-19, for W24×55, $\phi M_p = 503$ K-ft

$M_u = 194$ k-ft $< \phi M_p = 503$ k-ft

- No shoring required for strength, however shoring is required due to deck span

Check Wet Concrete Deflections

$W_{wc} = [37(12.67) + 55]/1000 = .52$ klf

$\Delta_{wc} = \frac{5wl^4}{384EI} = \frac{5(0.52)(40)^4(1728)}{384(29000)(1350)} = .765$ $\Delta_{wc} = .765"$

$\Delta_{wc\,max} = \frac{l}{240} = \frac{40(12)}{240} = 2"$ $\Delta_{wc\,max} = 2"$

$\Delta_{wc} = .765" < \Delta_{wc\,max} = 2"$

→ wet concrete deflection ok, no camber needed
→ 7/8" camber provided on drawings

Check Live Load Deflection

$W_{LL} = [100(12.67)]/1000 = 1.267$ klf

using table 3-20 : W24x55, $\Sigma Q_n = 328^k$, $y_2 = 4.54"$
$I_{LB} \approx 2590$, value undersized from an interpolated value will provide a more conservative deflection

$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(1.267)(40)^4(1728)}{384(29000)(2670)} = .94"$ $\Delta_{LL} = .94"$

$\Delta_{LL\,max} = \frac{L}{360} = \frac{40'(12"/ft)}{360}$ $\Delta_{LL\,max} = 1.33"$

$\Delta_{LL} = .94" < \Delta_{LL\,max} = 1.33"$

→ live load deflections ok, 7/8" camber provided on drawings

Beam OK ✓

Girder Check   W24×68 interior girder
[114] , 1 5/8" camber ↳ carried W24×55

Determine Load Path on Girder due to Beam

$W_{DL} = 61\,psf \quad W_{LL} = 100\,psf$

$W_u = 1.2W_{DL} + 1.6W_{LL} = 1.2(61) + 1.6(100) = 233.2\ psf$

$233.2\ psf \times (12.67') / 1000 = 2.95\ klf + 1.2(55/1000) = 3.016\ klf$
↑beam trib

$P = wl = 3.016 \times 40' = 120\,K$

using table 3-23 2 equal loads symmetrically placed

$M_{max} = Pa, \quad a = 12.67'$

$M_{max} = 120(12.67') = 1520.4'^{K}$

114 studs in 38' → 3 studs/rib for 19 ribs
- LW conc.
- deck parallel to girder
  $w_r = 1.25"$ $\frac{w_r}{h_r} = \frac{1.25}{2} = .625 < 1.5$
  $h_r = 2"$
- 3/4" ϕ studs
- $f'c = 3500$

From Table 3-23, $Q_n = \frac{17.1 + 18.3}{2} = 17.7^K$

$\Sigma Q_n = 19(17.7) = 336.3 = 336^K$
↑3 studs/rib

$T_{s\,max} = A_sF_y = 20.1(50) \quad T_{s\,max} = 1005^K$

$C_{c\,max} = .85 f'c\, b_{eff}\, t$

$b_{eff} = 2 \times \min \begin{cases} span/8 = 4.75' = 57" \\ 1/2\ dist\ to\ adj\ beam = 20' \end{cases}$

$C_{c\,max} = .85(3500)(57)(5)/1000 = 847.875^K$

$\Sigma Q_n < C_{c\,max} \quad 336^K < 848^K$ ✓
$\Sigma Q_n < T_{s\,max} \quad 336^K < 1005^K$ ✓

Partially composite girder

$$a = \frac{\Sigma Q_n}{.85 f'c\, b_{eff}} = \frac{336}{.85(3.5)(57)} = 1.98"$$

$$y_2 = t - a/2 = 5 - 1.98/2 = 4.01 \sim 4"$$

Using table 3-19, $\Sigma Q_n = 336^k$, $y_2 = 4"$ W24 x 68

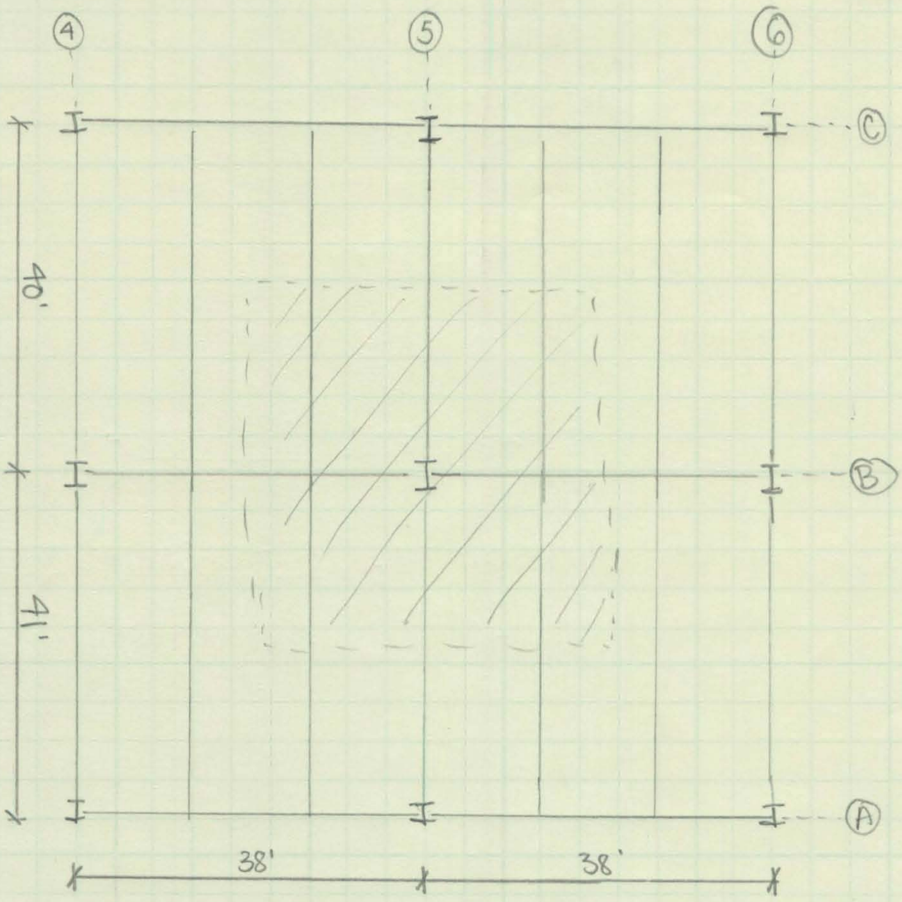
ΣQ_n	$Y_2 = 4"$
366	977
336	958 k-ft
251	904

$\phi M_n = 958$ k-ft

$M_u = 1520^{k\text{-}ft} < \phi M_n = 958$ k-ft

Girder not adequate for strength → shoring provided.

Interior Column : Column B5 (Level 1 to Roof)



Portion of Level 3 Framing Plan

Tributary Area: same for floors 1-5 & Roof

$A_T = \left(\frac{38}{2} + \frac{38}{2}\right) \times \left(\frac{41}{2} + \frac{40}{2}\right) = 1539 \text{ ft}^2$ / floor

Loads

Level	DL (psf)	LL (psf)	Space use
1	62	65	Retail
2	62	65	Retail
3	62	100	Office/public
4	62	100	office
5	62	100	office
Roof	92	25	Typical Roof

↳ Snow load = 17 psf

Loads

Controlling Load Combination: $1.2D + 1.6L + .5S$

Check Live Load Deflection

influence area = 1539 ft² > 400 ft²

Level 1 & 2 (same)

$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

From ASCE 7-05 Table 4-2, $K_{LL} = 4$ (int & ext col, no cantilever)

$L = 65\left(.25 + \frac{15}{\sqrt{4 \times 1539}}\right) = 28.68 \sim 29$ psf

$L \geq 0.40 L_o = 0.4(65) = 26$ psf

29 psf > 26 psf ✓ $\underline{L = 29 \text{ psf}}$

Levels 3, 4, 5 (same)

$L = 100\left(.25 + \frac{15}{\sqrt{4 \times 1539}}\right) = 44.12 \sim 45$ psf

$L \geq 0.40 L_o = .4(100) = 40$ psf

45 > 40 psf ✓ $\underline{L = 45 \text{ psf}}$

Roof Level

- Load un-reducable per ASCE 7-05 §4.8

Loading

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

Level 1: $[1.2(62) + 1.6(29)] \times 1539 / 1000 = 185.9 = 186^k$ (but since ground @ level 1 it does not contribute)

Level 2: same as level 1 $= 186^k$

Level 3: $[1.2(62) + 1.6(45)] \times 1539 / 1000 = 225^k$

Level 4: same as level 3 $= 225^k$

Level 5: same as level 3 $= 225^k$

Roof: $[1.2(92) + 1.6(25) + .5(17)] \times 1539 / 1000 = 244.5 = 245^k$

$\Sigma P_u = 186 + 3(225) + 245 = 1106^k$ $\underline{P_u = 1106^k}$

Column spliced between level 3 & 4

Level 3,4,5 : W14×74 : average KL value = 15.33'
Level 1,2 : W14×109 : average KL value = 18.67'

Levels 3-5: P_u = 225(2)+ 245 = 695^k, KL = 15.33 W14×74

KL	ϕP_n
15	667
15.33	655.78^k
16	633

$P_u = 695^k > 655.78^k = \phi P_n$
column not ok → but spliced so ok

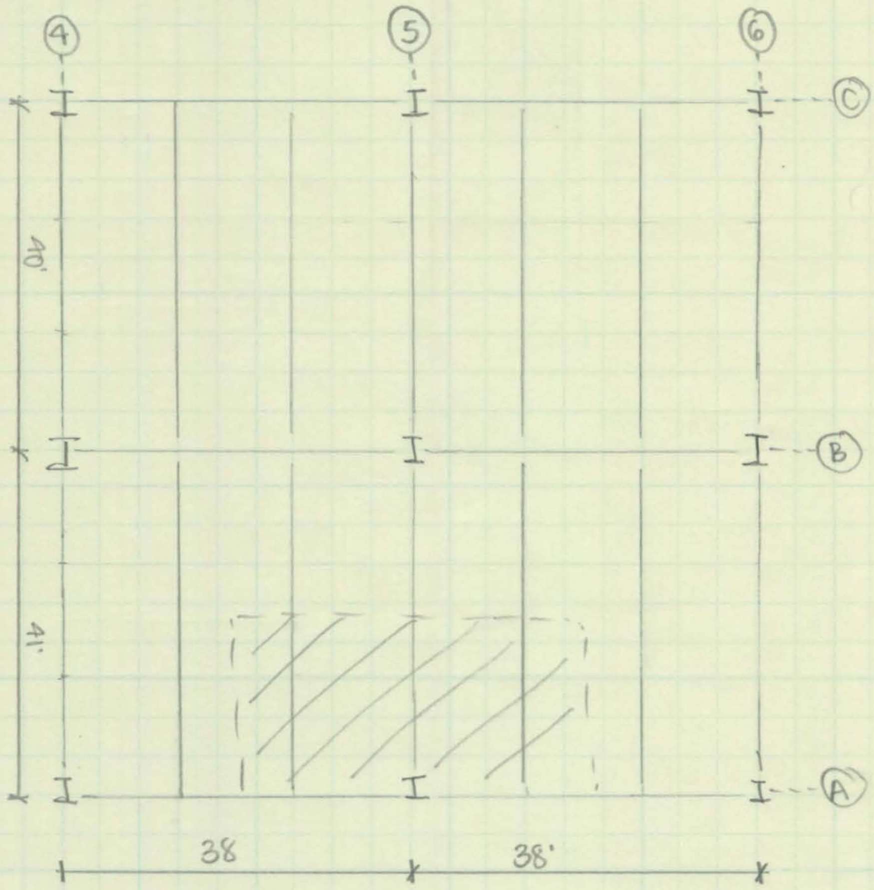
Levels 1-2: P_u = 1106^k , LK = 18.67 W14×109

KL	ϕP_n
18	1130
18.67	1110^k
19	1100

$P_u = 1106^k < 1100^k = \phi P_n$
Column OK

The columns are ok ✓

Exterior Column



Tributary Area : same for floors 1-5 & Roof

$A_T = \left(\frac{38}{2} + \frac{38}{2}\right) \times \left(\frac{41}{2}\right) = 779\ ft^2$ /floor

Loads

Level	DL (psf)	LL (psf)	
1	62	65	
2	62	65	
3	62	100	
4	62	100	
5	62	100	
Roof	92	25	17 psf snow load

Exterior Wall Load : 848plf $\times \left(\frac{38}{2} + \frac{38}{2}\right) / 1000 = 32.2^k$
↑ value from Tech 2

Columns spliced between level 3 & 4.

Levels 1-2 : W14×90   Levels 3-5 : W14×61

Live Load Deflection

influence area = 779 ft^2 > 400 ft^2 ✓

Level 1 & 2 (same)

$$L = 65\left(.25 + \frac{15}{\sqrt{4 \times 779}}\right) = 33.7 = 34 \text{ psf}$$

Level 3 - 5 (same)

$$L = 100\left(.25 + \frac{15}{\sqrt{4 \times 779}}\right) = 51.87 = 52 \text{ psf}$$

Roof: live load unreducable

Controlling Load Case = 1.2D + 1.6L + .5S

Level 1: $[1.2(62) + 1.6(34)]\, 779/1000 = 100.33 = 100^k$ (though it is on ground level & does not contribute)

Level 2: same as Level 1: 100^k

Level 3: $[1.2(62) + 1.6(52)]\, 779/1000 = 122.77 = 123^k$

Level 4: 123^k

Level 5: 123^k

Roof: $[1.2(92) + 1.6(25) + .5(17)]\, 779/1000 = 123.78 = 124^k$

$\Sigma P_u = 100 + 3(123) + 124 = 593^k$ $P_u = 593^k$

Using Table 4-1

Levels 3-5, $P_u = 123(3) + 124 = 493^k$ $K_L = 15.33'$ W14×61

KL	ϕP_n
15	543
15.33	533.34^k
16	514

493 < 533 → column OK ✓

Levels 1-2, $P_u = 593^k$ $KL = 18.67$ W14×90

KL	ϕP_n
18	929
18.67	911.66^k
19	903

$P_u = 593^k < 911.66^k = \phi P_n$

Column OK ✓

For my structural re-design I will continue to use the bay that was used in Gravity Load Spot Checks because it is a very typical bay that can be found throughout the majority of the building.

Alternate System 1: Non-Composite Steel Framing.

Alternate system 2: One-way concrete slab

Alternate System 3: Two-way concrete slab

Alternate System 1: Non-Composite Steel Framing

Note: the beam & girder studied in the spot check will be the members used for re-design
- existing deck will remain.
- existing framing layout to remain

Beam sizing:

- gravity spot check: W24x55, $M_u = 591$ K-ft
  span = 40', 12.67' spacing

- size beam using table 3-2

Strength: need $\phi M_n > M_u$

select W21x68, $\phi M_n = 600$ K-ft

$600 > 591$ ✓

Deflections

$$\Delta_{max} = \frac{5wl^4}{384EI}$$

w = 62 psf DL × 12.67/1000 = .79 klf
I = 1480 in⁴

$$\Delta_{max} = \frac{5(.79)(40)^4 1728}{384(29000)(1480)} = 1.06"$$

$\Delta_{max} = 1.06"$

$$\frac{L}{360} = \frac{40(12)}{360} = 1.33 > \Delta_{max} \checkmark$$

Self-Wt check (10 psf given in allowance)

$$\frac{68 \text{ plf}}{12.67} = 5.37 \text{ psf}$$

$5.37 < 10$ ✓

Use W21x68 Typical Beam at 12.67' spacing

Girder Sizing – Interior Girder

DL = 62 + 2 psf self (from Gravity Check)
LL = 45 psf (from Gravity Spot Check)

$w_u = 1.2D + 1.6L \quad = 1.2(64) + 1.6(45) = 148.8 \text{ psf} \times 12.67/1000 = 1.89 \text{ klf}$

$P = w\ell = 1.89(38') = 71.6^k$

$M_{max} = Pa = 71.6(40') = 2865.65 = 2866 \text{ k-ft} = M_u$

Select W 40 × 183, $\phi M_n = 2900$ k-ft

$2900 > 2866$ ✓

Deflections: $\Delta_{max} = \dfrac{5w\ell^4}{384EI} = \dfrac{5(1.89)(38)^4 1728}{384(29000)(13200)} = .23''$

$\Delta = L/360 = \dfrac{38(12)}{360} = 1.27''$ $\quad 1.27 > .23$ ✓

Interior Girder W40 × 183

Exterior Girder

$a = 20'$

same w as interior $= 1.89$ klf $+ .848$ exterior wall load $= 2.74$ klf

$P = 2.74 \times 38' = 104.044^k$

$M_{max}: 104.044(20) = 2080.88 \text{ k-ft} = M_u$

Select W40 × 149, $\phi M_n = 2240^k$-ft

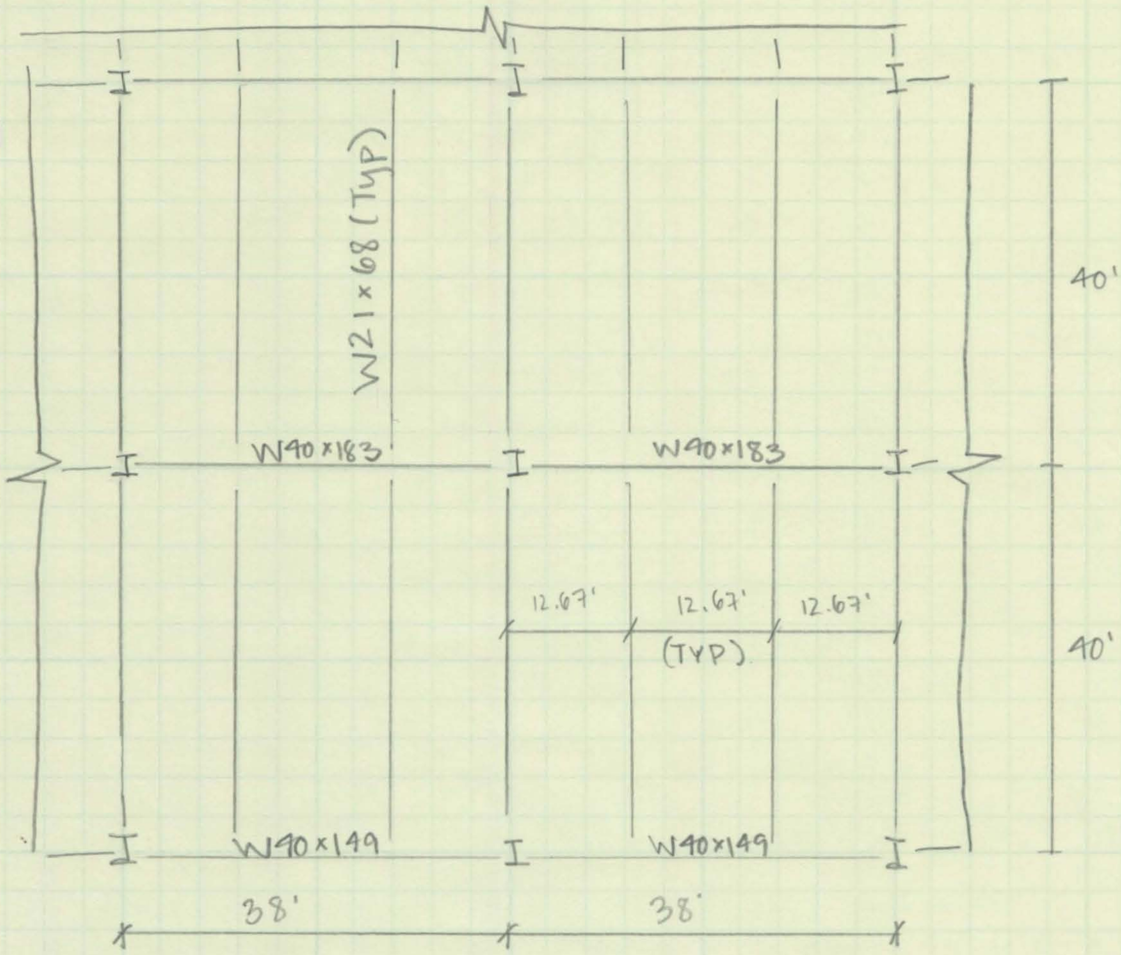
Deflection Check:

$$\Delta max = \frac{5(2.74)(38)^4(1728)}{384(29000)(9800)} = .45''$$

$\Delta_{allowable} = L/360 = 1.27''$

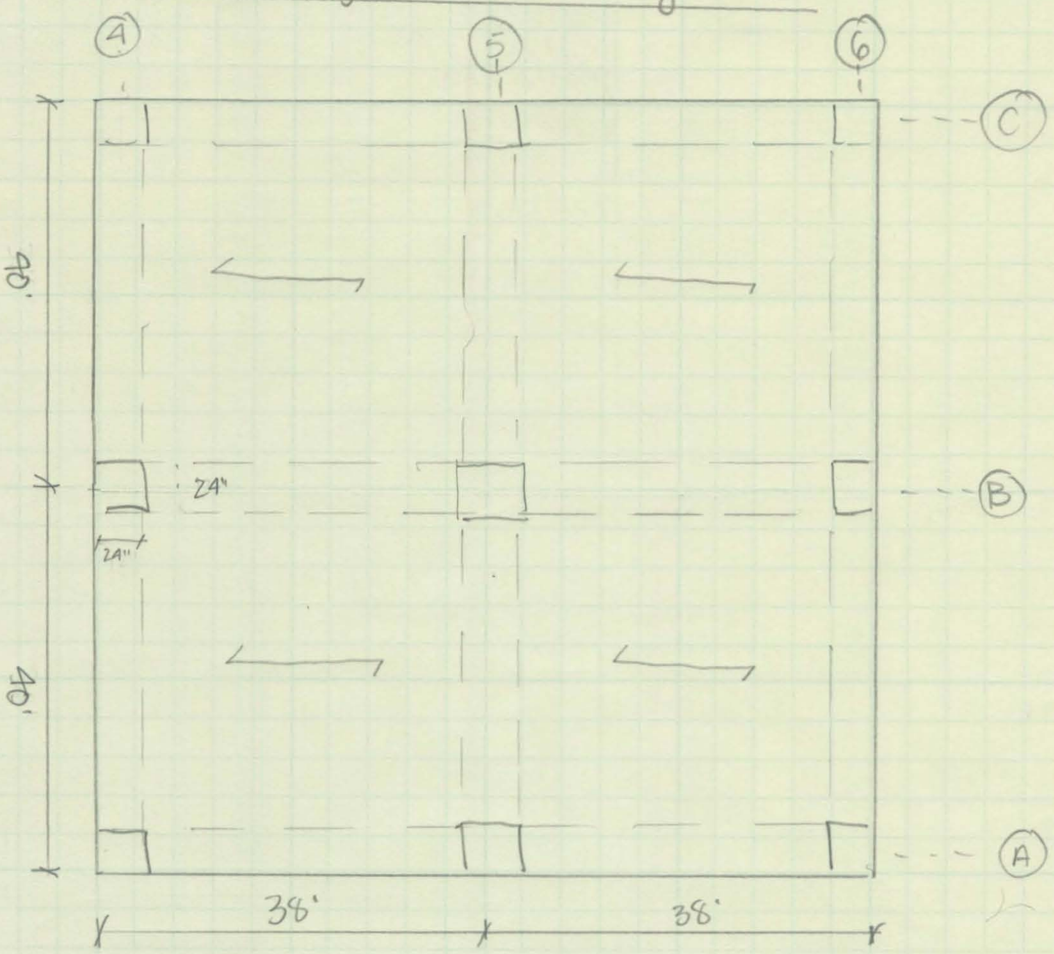
$1.27 > .45$

Exterior Girder = W40 × 149



Alternate System 1 Layout

Structural ReDesign 2 - One Way Slab



- $f'c = 3500$ psi   use LW concrete as specified in S-001
- deflection control
- slab min thickness
  endspan $= \ell/24$

$$\frac{\ell}{24} = \frac{38(12)}{24} = 19'' \rightarrow \text{use } 19'' \text{ thick slab}$$

Panel Design

$w_{DL} = 10\text{psf} + 115\left(\frac{19}{12}\right) = 192.08$ psf

$w_{LL} = 100$ psf for "public space"

Controlling Load Case

$1.2D + 1.6L = 1.2(192.08) + 1.6(100) = 390.5$ psf

Try #6 bars

$d = h - C_c - d_b/2 = 19 - 1.5 - \frac{.75}{2}$ $d = 17.125''$

Beam Design

$w = (390.5)(38') / 1000 = 14.84$ klf

$M_u = \frac{wl^2}{8} = \frac{14.84\,(40'-2')^2}{8} = 2678.62$ k-ft

self weight

$2678.62 \times 1.1 = 2946.482$ k-ft

$b = \frac{4}{5}d \qquad bd^2 = 20M_u$

$\frac{4}{5}d^3 = 20(2946.5)$

$d = 41.9'' = 42''$

$b = \frac{4}{5}(42) = 33.5''$

$h = d + 2.5 = 44.5$

$w_{self} = bd = \frac{44.5(33.5)}{144} \times 115 = 1.19$ klf

$W_u = 14.84 + 1.2(1.19) = 16.27$ klf

$M_u = \frac{16.27\,(40 - \frac{44.5}{12})^2}{8} = 2959.32$ k-ft

$bd^2 = 20\,M_u$

$(33.5)(42)^2 = 20(2959.32)$

$59094 < 59186.38 \therefore$ good

Required Steel

$A_s = \frac{M_u}{4d} = \frac{2959}{4(42)} = 17.61\ in^2$

Try 14 #10 bars $= 14(1.27) = 17.78\ in^2$

$d = 44.5 - 1.5 - 1.27/2 = 42.365''$

Nominal Moment

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{17.78(50)}{.85(3.5)(33.5)} = 8.92''$

$c = a/B_1 = 8.92/0.8 = 11.15$

$\varepsilon_s = 0.003\left[\frac{d-c}{c}\right] = 0.003\left[\frac{42.365-11.15}{11.15}\right] = .0084 > .005 \rightarrow$ not ok

need more rebar

Try 18 #9 bars = 18(1.00) = 18 in^2

$d = 44.5 - 1.5 - \frac{1.128}{2} = 42.44"$

try new grade

$a = \frac{18(60)}{.85(3.5)(33.5)} = 10.84$

$c = 10.84/.8 = 13.55$

$\varepsilon_s = 0.003\left[\frac{42.44-13.55}{13.55}\right] = .0064 > 0.005 \rightarrow$ still not ok

more reinforcing

Try 17 #10 bars = 17(1.27) = 21.59 in^2

$d = 44.5 - 1.5 - \frac{1.27}{2} = 42.365"$

$a = \frac{21.59(60)}{.85(3.5)(33.5)} = 12.997 = 13$

$c = 13/.8 = 16.25$

$\varepsilon_s = 0.003\left[\frac{42.365-16.25}{16.25}\right] = .0048 < .005 \rightarrow$ ok

use $\phi = 0.9$

$\phi M_n = 0.9(A_s f_y)(d - a/2)$
$= [0.9(21.59)(60)(42.365 - 13/2)]/12 = 3510.7$ ft-k

$\phi M_n = 3510.7$

$\phi M_n > M_u$ ✓

$M_u = 2959.32$

min area of steel

$A_{s\,min} = \frac{200}{f_y} bd = \frac{200}{60{,}000}(33.5)(42.365) = 4.76$ in^2

$21.59 \gg 4.76$ ✓

Check max reinforcing ratio

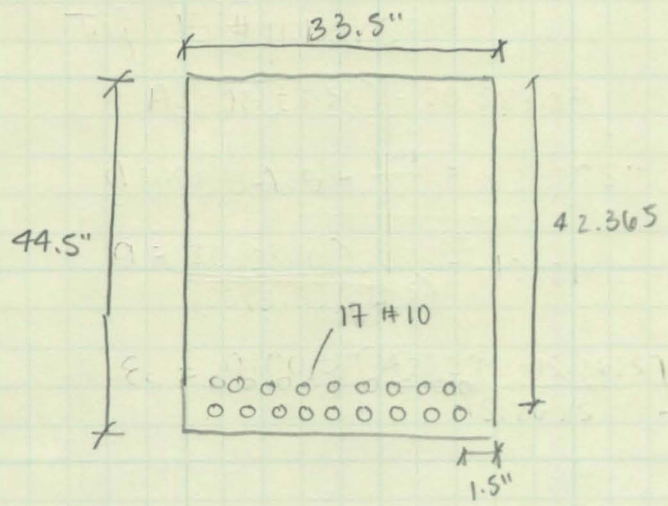
$\rho_{max} = 0.85\beta_1 (f'c/F_y)(\varepsilon_u/\varepsilon_y + \varepsilon_y)$

$= 0.85(.8)(3.5/50)(0.003/0.003+0.005)$

$\rho_{max} = 0.01785$

$\rho = A_s/bd = 21.59/(33.5)(42.365) = 0.0152$

$\rho < \rho_{max} \rightarrow ok$



use a 33.5" × 44.5" beam w/ 17 #10 rebar

Slab Design

$t = 19"$ try #6 bars $(A = .44 in^2)$

using the unit strip method (1' strip)

$A_{s\,min} = 0.0018(12")(19") = .4104$ per ACI 318-11 § 7.12.2.1

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

$.4104 = .44(12"/spacing)$
spacing = 12.87"

$S_{max} = \min\begin{cases} 3t = 3(19) = 57 \\ 18" \leftarrow \text{controls} \end{cases}$

$12.87 < 18$ ✓ per ACI 318-11 §10.5.4

spacing crack control

$S = 15\left(\frac{40{,}000}{f_s}\right) - 2.5\,C_c$ or $S = 12\left(\frac{40{,}000}{f_s}\right)$

$= 15\left(\frac{40{,}000}{60{,}000}\right) - 2.5(.75)$ $= 12\left(\frac{40}{60}\right)$

$S = 8'' \rightarrow$ controls

$S = 8.125''$

Try #6 rebar @ 8" O.C. $d_b = .75''$

$A_s = .44$

$\frac{8}{12} \times .44 = .293$

$e = A_s/bd = .293/(12'')(19'') = .00129$

$\phi = 0.9$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.44(60)}{.85(3.5)(12)} = .739$

$d = 19'' - .75 - .75/2 = 17.875''$

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

$= [.75(.44)(60)(17.875 - \frac{.739}{2})]/12$

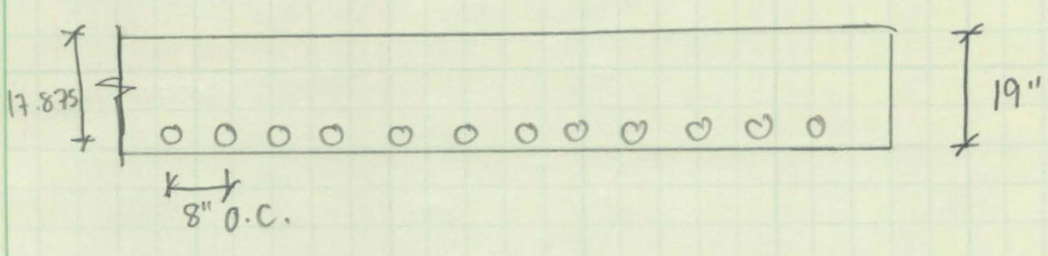
$\phi M_n = 28.88$ k-ft

$w = 1.2\left[\left(\frac{19}{12}\right)(115)(1') + 10 \text{ psf}\right] + 1.6(100]$ $w = .3905$ klf

$M_u = \frac{wl^2}{8} = \frac{.3905(38')^2}{8} = 70.49'^k$

$\phi M_n = 28.88 < 70.49 = M_u$ ✓

use a 19" slab w/ #6 rebar @ 8" O.C.



Deflection check

Beam:

$$I = \frac{bh^3}{12} = \frac{33.5(44.5)^3}{12} = 246004.8073 \qquad E = 115^{1.5}\sqrt{3.5} = 2307$$

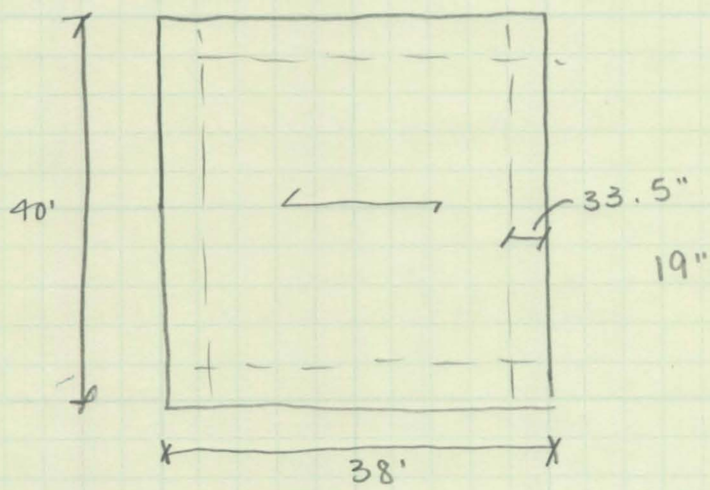
$$w = 10 + \left(\tfrac{19}{12}\right)(115)(38') + \frac{33.5(44.5)}{144}(115) = 8.12 \text{ klf}$$

$$\Delta_{TL} = \frac{5wl^4}{384EI} = \frac{5(8.12)(40)^4}{384(2307)(246004.8)}(1728) = .82''$$

$$\frac{l}{480} = \frac{40(12'')}{480} = 1''$$

$$\frac{l}{360} = \frac{40(12)}{360} = 1.33$$

$$.82 < 1'' \checkmark$$

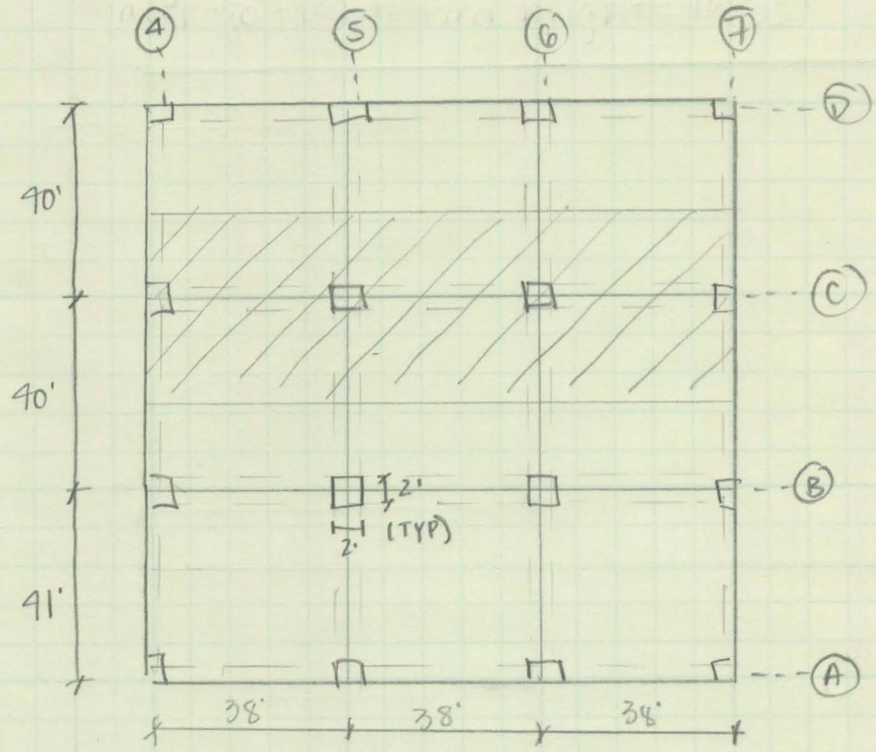


F'c = 3500 psi
LW conc (to keep w/ S-001)
f_y = 60 ksi

19" slab w/ #6 rebar @ 8" o.c.

Two-Way Slab (flat plate)

- valid because bays are nearly square: $(L/S) < 2.0$
- NW concrete
- $f'c = 4000$ psi
- 24 × 24 concrete columns assumed at current column locations
- E = 29000
- $f_y = 60$ ksi



interior beams, edge beams. Per ACI 318-11 Table 9.5(c)

min thickness of slab = $\ell_n / 33$ = $40(12)/33 = 14.54"$

↳ no edge beams

try 15" slab

Try using Direct Design Method

- min 3 continuous spans → ok
- span lengths should differ by less than 1/3 ℓ_n
  (all same span → ok)
- no columns offset from grid ✓
- unfactored live load cannot exceed 2× unfactored dead load

$$= \frac{L}{D} = \frac{100}{62} \leq 2.0 \quad ✓$$

→ OK to use DDM

Load Determination

LL = 100 psf
DL = (15/12) 150 + 3 psf allowance = 190.5 psf
Misc DL = 15 psf

Load Combination = $1.2D + 1.6L = 1.2(190.5+15) + 1.6(100) =$

$q_u = 406.6$ psf

- since the slab was sized using ACI 318-11 Table 9.5(c), deflection may be neglected.

Check shear capacity

Check 2 way shear - column C-5 (see plan on previous page)

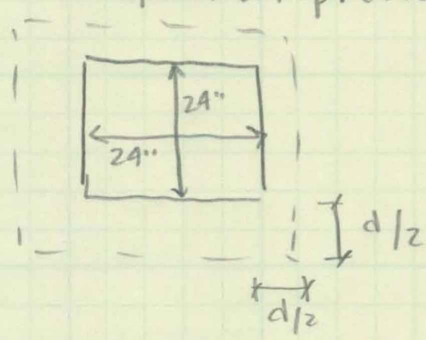
$q_u = 406.6$ psf

• .75" ℄ assumed

$d = 15 - .75 - \frac{3(.625)}{2}$

$d = 13.3125"$

$d/2 = 6.656"$



$b_o = 4(24 + 6.656) = 122.624$ in²

$\beta = 24"/24" = 1.0 \quad \alpha_s = 40$

$$V_{cmin} = \begin{cases} 2 + 4/\beta = 6" \\ \frac{\alpha_s d}{b_o} = \frac{40(13.3125)}{122.624} = 4.34" \\ 4" \leftarrow \text{controls} \end{cases}$$

$V_c = V_{cmin}\sqrt{f'c}\,(b_o)(d)$
$= 4\sqrt{4000}\,(122.624)(13.3125)$

$\phi = .75$ for shear

$V_c = 412.97^k \qquad \phi V_c = 309.73^k$

$V_u = .4066\left[(19' \times 20') - \left(\frac{122.624}{144}\right)\right]$

$V_u = 154.16^k \qquad \phi V_c > V_u$ ✓

Slab passes for punching shear

Check one way shear:

$$V_u = 0.4066\left(19 - \frac{24}{2}\right)(20 - 24/2)$$

$$V_u = 124.42^k$$

$$V_c = 2(1.0)\sqrt{4000}\,(24 \times 12)(13.3125)/1000$$

$$V_c = 484.97 \qquad \phi V_c = 363.73$$

$$\phi V_c > V_u \checkmark$$

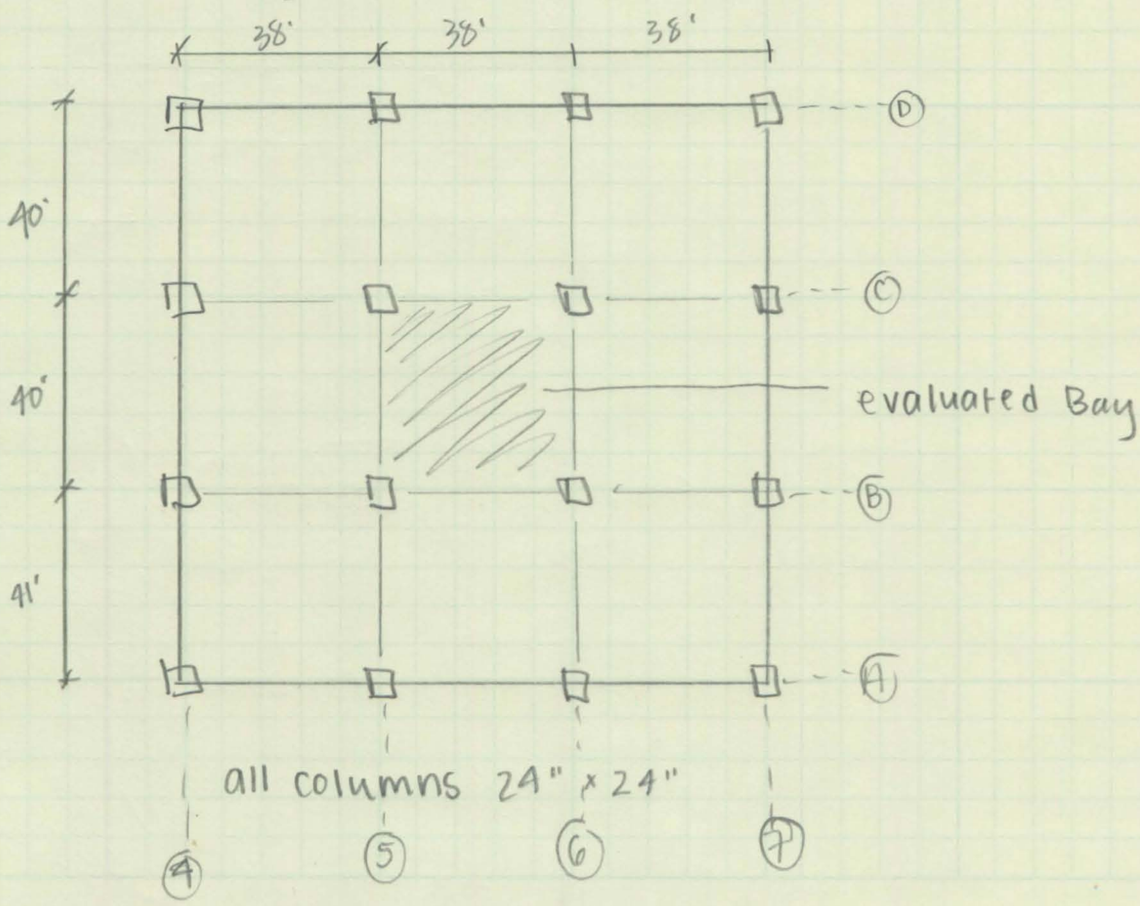
Slab passes shear check ✓

move to flexural direct design method

Direct Design Method

$q_u = 406.6$ psf

$$M_0 = \frac{q_u \ell_2 \ell_n^2}{8}$$



Slab Moment Determination

Long Direction: $M_o = [406.6\,(38)\,(40 - \frac{24}{12})^2]/8 = 2789^{\prime k}$

Short Direction: $M_o = [406.6\,(40)\,(38 - \frac{24}{2})^2]/8 = 2635^{\prime k}$

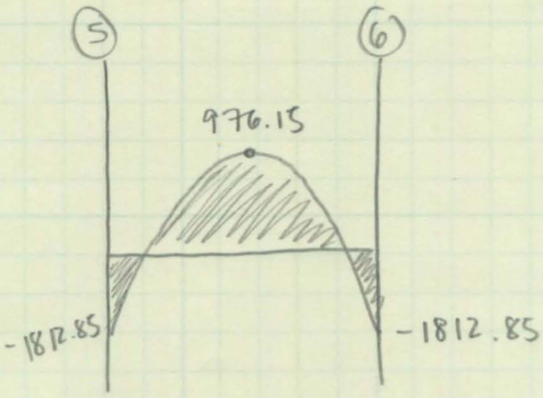
Coefficients for featured moments

Interior spans:
- midspan: $.35\,M_o$
- Column: $.65\,M_o$

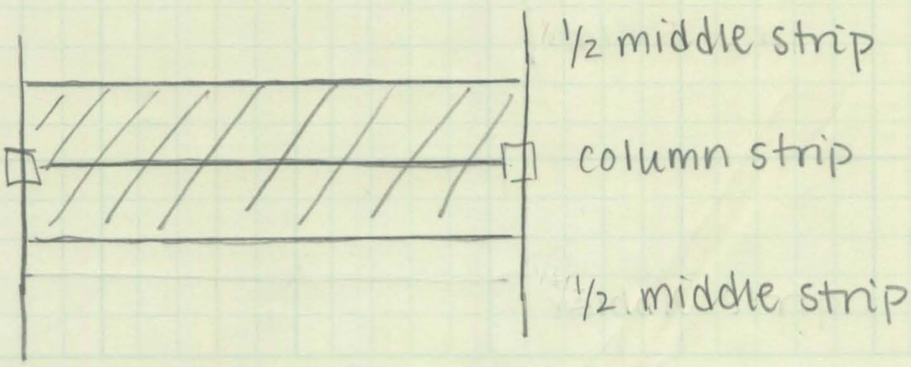
Exterior Span
- midspan: $.33\,M_o$
- column: $.67\,M_o$

	M −	M +
Long Direction	$.65\,M_o = 1812.85^{\prime k}$	$.35\,M_o = 976.15^{\prime k}$
Short Direction	$.67\,M_o = 1765.45^{\prime k}$	$.33\,M_o = 869.55^{\prime k}$

For long Direction



$\alpha = 0$ b/c no interior beams
$\beta = 0$

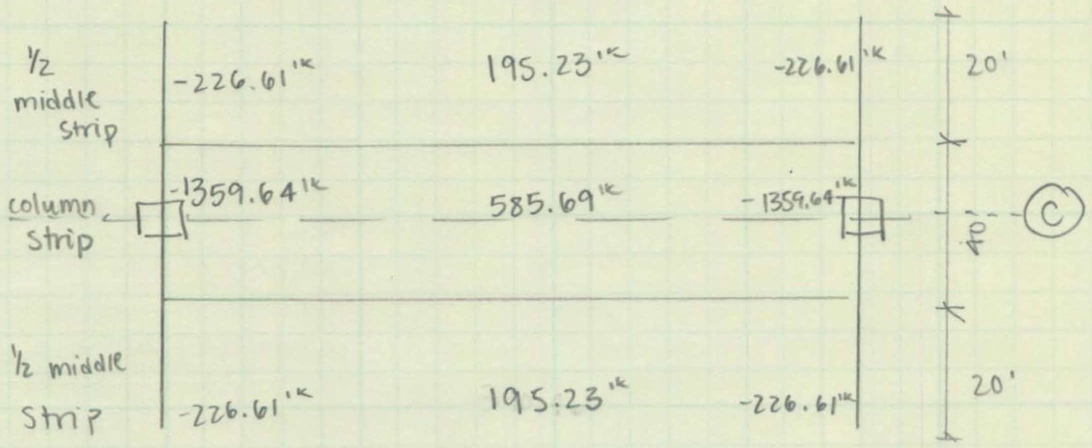


% negative moment determination § 13.6.4.1

$$\alpha_f = 0: \quad \rightarrow .75\ Cs = .75(-1812.85) = -1359.64^{'k}$$
$$\rightarrow .25\ Ms = .25(-1812.85) = -453.21^{'k}$$

% positive moment § 13.6.4.1

$$\alpha_f = 0 \quad \rightarrow .60 \text{ to } Cs = .6(976.15) = 585.69^{'k}$$
$$\rightarrow .40 \text{ to } Ms = .4(976.15) = 390.46^{'k}$$



Design Reinforcement for Bending

$$A_{s\,req} = \frac{M_u}{\phi f_y j d} = \frac{-1359.64(12)}{.9(60{,}000)(.95)(13.3125)(1/1000)} = 23.89\ in^2$$

§ 13.3.2 - spacing

$$S_{max} \le 2h = 2(15) = 30''$$

Minimum Reinforcement

$$A_{s\,min} \ge 0.0018bh$$
$$\ge 0.0018(40)(15)(12)$$

$$A_{s\,min} \ge 12.96\ in^2$$

$$A_{s\,req} > A_{s\,min} \checkmark$$

Try 19 #10 bars @ 12" spacing

$A_s = 19(1.27) = 24.13 > 23.89\ in^2$

$$a = \frac{(24.13)(60,000)}{.85(4000)(15\times12)} = 2.37$$

$$\phi M_n = .9(24.13)(13.3125 - \frac{2.37}{2})(60)(\frac{1}{12})$$

$\phi M_n = 1424.09 > M_u = 1359.64$ ✓

<u>Middle Strip, M^-</u> : $M_u = 453.21$

$$A_{s\,req} = \frac{-453.21\times12}{.9(60)(.95)(13.3125)} = 7.96\ in^2$$

since this is less than 12.96 in², use A_s min instead

Try <u>13 #9 bars</u> $\rightarrow A_s = 13(1.0) = 13\ in^2$

$$a = \frac{13(60000)}{.85(4000)(15\times12)} = 1.27$$

$$\phi M_n = .9(13)(13.3125 - \frac{1.27}{2})(60)(\frac{1}{12})$$

$\phi M_n = 741.63 > M_u$ ✓

<u>Column Strip M^+</u> $M_u = 585.69$

$$A_{s\,req} = \frac{585.69\times12}{.9(60)(.95)(13.3125)} = 10.29\ in^2$$

- same condition as middle strip above, $A_s = 12.96 in^2$,

  use 13 #9 bars @ 12"

<u>Middle Strip M^+</u> : $195.23^{'k}$

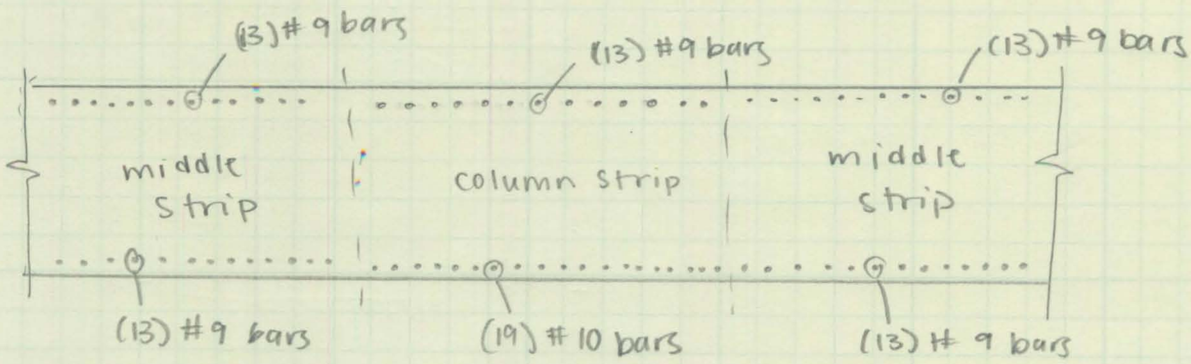
$$A_{s\,req} = \frac{195.23^{'k}\times12}{.9(60)(.95)(13.3125)} = 3.43\ in^2$$

- same condition as above, $A_{s\,min} = 12.96\ in^2$, use 13#9 bars

Slab Reinforcement Summary

Column Strip
Top: (13) #9 bars @ 12"
Bottom: (19) #10 bars @ 12"

Middle Strip
Top: (13) #9 bars @ 12"
Bottom: (13) #9 bars @ 12"



Cost Comparisons

- all costs are approximate values calculated using section B of RS means, using the 2002 square foot costs I created a relative cost analysis

Existing System (Composite Steel)

W beams & Girders

~40'x40' Bays, ~75 psf super imposed load ≈ $15.00/SF

Non-Composite Steel

W beams & Girders

~40x40' Bays, ~$17.45/SF

One-Way Concrete slab, Cast in Place

Slab span = 38' → $13.00/SF

Two-Way concrete slab, cast in place

~40x40' bays, ~$15.60/SF