James W. & Frances G. McGlothlin Medical Education Center
Virginia Commonwealth University
Richmond, VA

October 18, 2013

Professor Linda Hanagan
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Dear Professor Hanagan,

I am formally submitting Structural Technical Report #3 – Typical Member Spot Checks for Gravity Loads & Alternate Systems Typical Bay Design Study. As the name suggests, this report is a more thorough investigation of the gravity loads, found in Technical Report #2, that are applied to a typical bay in this building. Once the spot checks were completed, three alternate systems were considered and compared alongside with the existing structure. A table of contents and numbering of pages has been provided for ease of navigating this report. Calculations for the spot checks have been done by hand, and therefore have been scanned to be inserted in to this report. While investigating alternate systems, some approximation methods were used, but all pertinent information has been provided. I look forward to presenting my findings to you, other notable faculty, and my fellow classmates in the near future.

Sincerely,

Marissa Delozier

Enclosure: Report of Findings Related to Gravity Loads on Typical Members & Possible Alternate Systems for Typical Bay Design for the James W. & Frances G. McGlothlin Medical Education Center

General Information

The James W. & Frances G. McGlothlin Medical Education Center is a 13-story building that has both a basement and small sub-basement located below ground level, which is at an elevation of 153 feet. Since the building was constructed following the demolition of the A.D. Williams Building, the foundation system is designed to accommodate existing conditions. The superstructure of the building is composed of a braced moment frame system with concrete slabs on metal decking. Both the 13^{th} Floor and the rooftop are homes to mechanical equipment, requiring added strength. A bridge traveling over E. Marshall Street connects the new building on the 2^{nd} Floor with the existing Main Hospital 1^{st} Floor. Further information about the building and its location in downtown Richmond, Virginia can be found on the following pages.

NOTE: To decrease confusion and provide easier reading, from this point in the report and forward the James W. & Frances G. McGlothlin Medical Education Center will be referred to as VCU SOM project, short for Virginia Commonwealth University School of Medicine project.

Building Abstract

James W. & Frances G. McGlothlin Medical Education Center

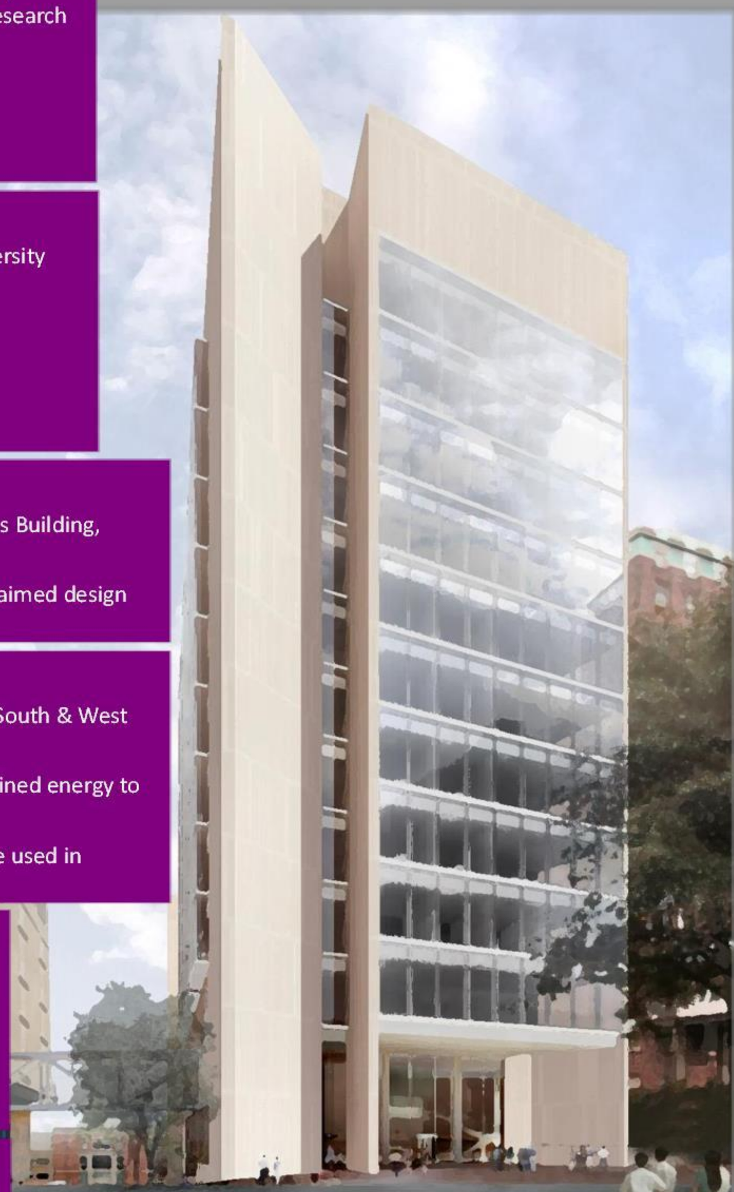
Virginia Commonwealth University – Richmond, VA

Project Information

Type of Building :	Multipurpose Education Facility
Functions :	Administrative/Classrooms/Research
Size :	220,000 GSF
Height :	13 stories
Time Frame :	Oct. 2009 – March 2013
Cost :	$159 million
Delivery :	Design–Assist–Build

Project Team

Owner :	Virginia Commonwealth University
CM :	Gilbane Building Company
Architect :	Ballinger
Structural + MEP :	Ballinger
Exterior Façade :	Pei Cobb Freed & Partners
Civil :	Draper Aden Associates
Geotechnical :	Geotech, Inc.



Architectural

– Erected following demolition of 8-story A.D. Williams Building, which previously housed VCU School of Medicine

– Exterior façade was designed by internationally acclaimed design firm Pei Cobb Freed & Partners

Sustainability

– Climate Wall System: double-layered glass walls on South & West facades trap & exhaust heated air

– Recovery Wheels: recover exhausted air & use contained energy to heat & cool building

– Storm Water Retention: collect water from roof to be used in toilets/urinals

Structural

– Drilled pier/slab-on-grade system works in conjunction with pre-existing caissons

– Structural steel braced moment frame system

– Bridge connects 2nd Floor of building to adjacent Main Hospital 1st Floor across E. Marshall Street

MEP

– 6 Air Handling Units serve the Lobby, Student Forum, Auditorium, and Chilled Beam system

– Cooling Tower on roof removes heat from 3 Chillers

– Use of Recovery Wheels saves 450 tons of cooling

– Daylighting sensors throughout building ensure energy is conserved

Marissa Delozier

Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2013/mnd5036/

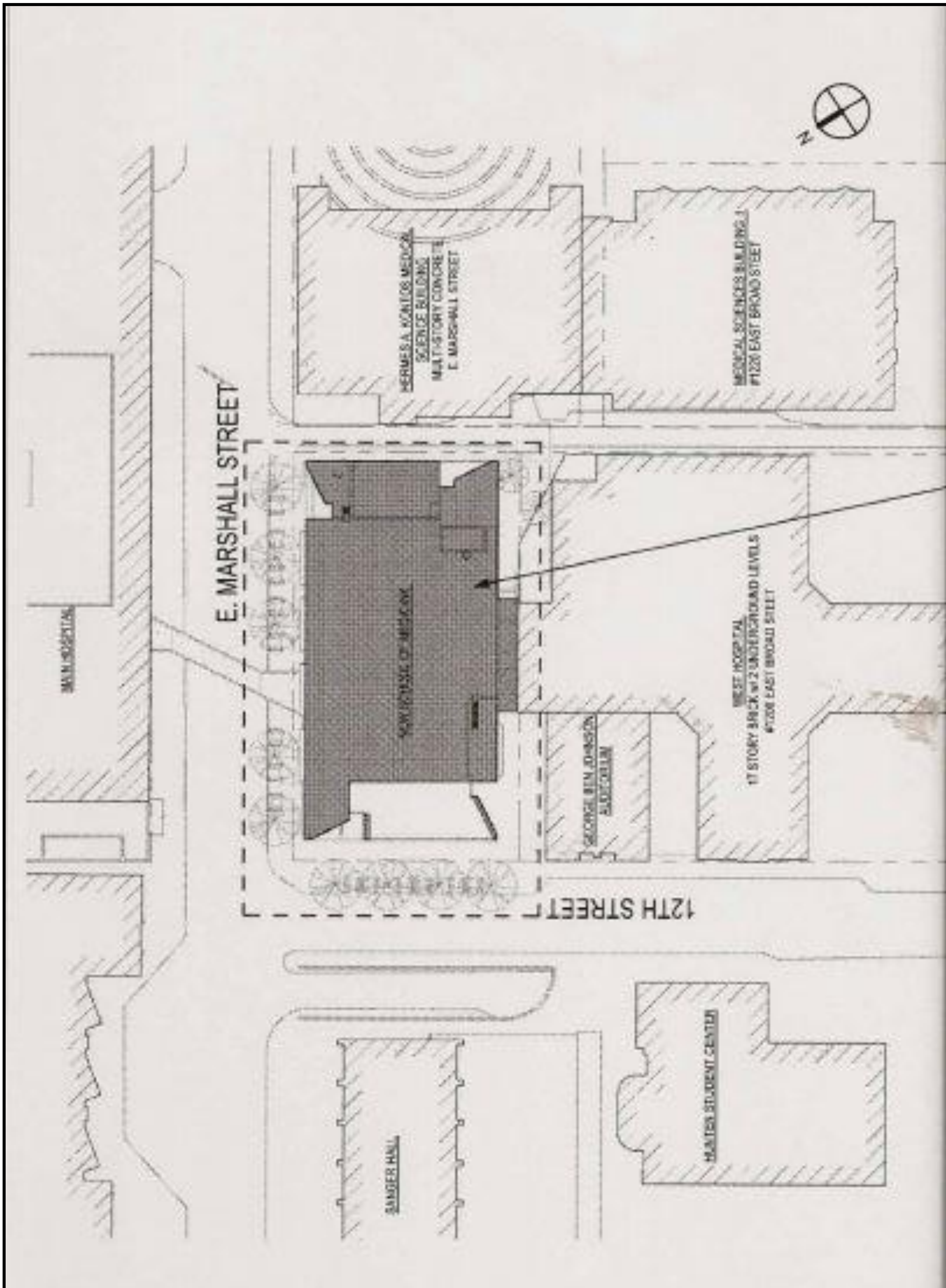
Executive Summary from Technical Report 1

The following technical report is a thorough overview of the existing conditions of the structural system found in the newly constructed James W. & Frances G. McGlothlin Medical Education Center. This report is composed of detailed descriptions of the drilled pier/slab-on-grade system, floor framing, braced moment frame system, roof scheme, bridge connecting to an adjacent structure, and all other components that contribute to the strength of the structure.

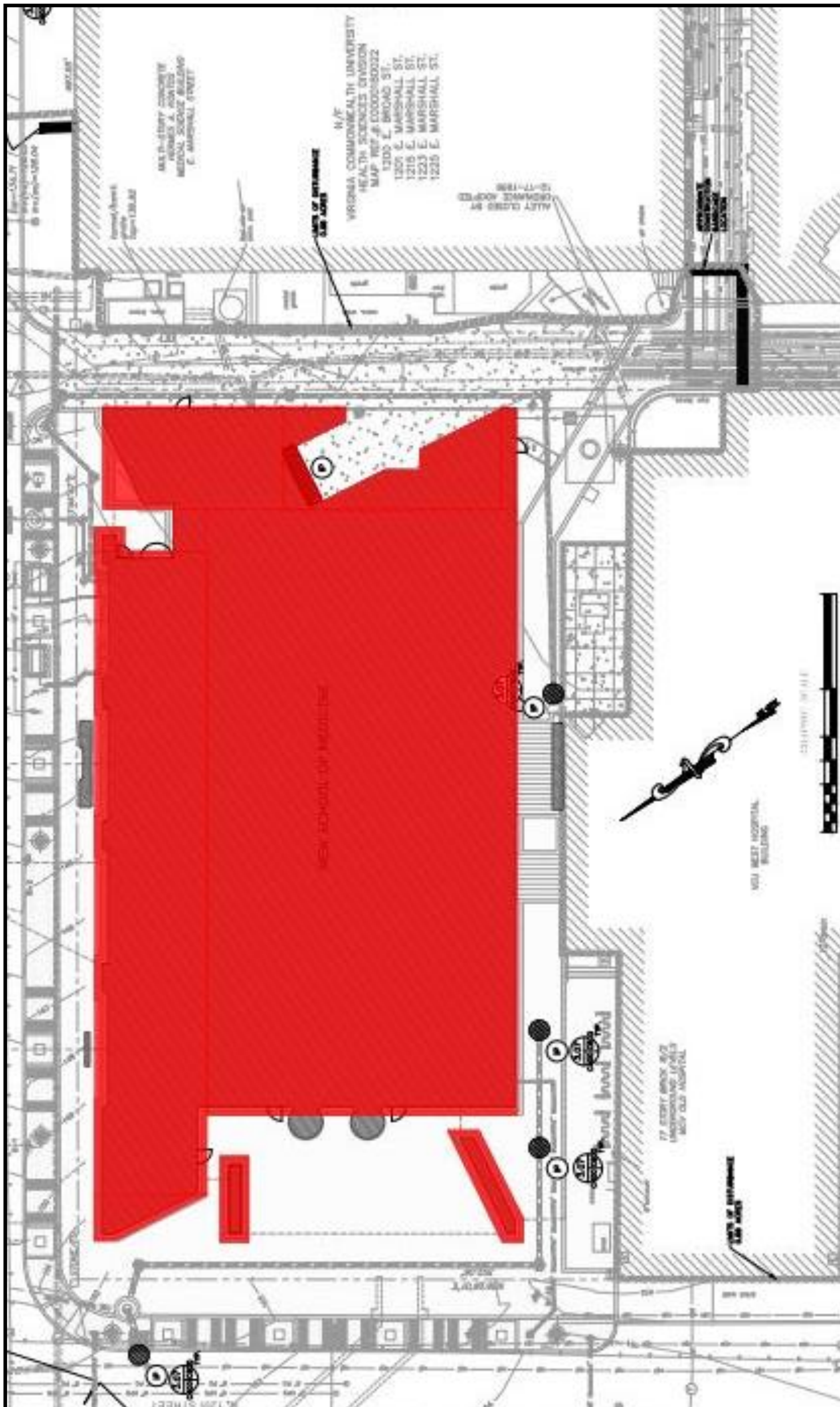
Though it is said that the sum is greater than its parts, the structural apparatuses that compose this project are diverse, complicated systems that must be thoroughly examined to fully appreciate the building. Many challenges exist surrounding the project: the site location, building size, intended function, connection to existing structures, and many more. This report is only the first investigation in to the structure of the James W. & Frances G. McGlothlin Medical Education Center – further analysis and study will be necessary to fully comprehend the magnitude of these systems.

In order to provide background information, floor plans, bays, columns, and other elements from the structure are referenced throughout the report and can be found in the appendices for further examination. State and national codes used in the design of the structure are also cited in the following report; these codes, more specifically loading values, will be utilized in further research and subsequent technical reports.

Location Plan



Site Plan



Reference Documents

In the preparation of the calculations found on the following pages, several documents outside the construction drawings and specifications were referenced. The main source of information was the American Society of Civil Engineers (ASCE) 7-05 code, specifically for both wind and seismic loads. All of the necessary variables, equations, and values needed to calculate the loadings and base shears were found from this document. A document utilized in the calculation of both roof and floor loadings was the Vulcraft Steel Roof and Floor Deck catalog. The American Institute of Steel Construction (AISC) 2005 code was also used for gravity loadings, to estimate size and weight.

Gravity Loads

Live Loads			
Floor/Area	Design (psf)	ASCE 7-05 (psf)	Typical Use
Sub-basement	250	150	Mechanical †
Loading Dock	350	—	—
Basement	100	100	Offices + Storage
1st	↑	100	Lobby
2nd		60	Assembly (fixed seat)
3rd		60	Assembly (fixed seat)
4th		80	Offices + Corridors
5th		80	Classrooms + Corridors
6th		↑	↑
7th			
8th			
9th		↓	↓
10th		80	Classrooms + Corridors
11th	↓	80	Offices + Corridors
12th	100	80	Offices + Corridors
13th	150	150	Mechanical †
Roof	45	20	Flat Roof

† This value was assumed.

Dead Loads°	
System	Assumed Loads (psf)
Decking	2
Insulation	2
Roofing	20
Misc. DL	10

°No known dead loads were referenced in the contract documents, so values were assumed based on common practice.

Snow Loads		
Area	Design (psf)	ASCE 7-05 (psf)
Ground	20	20
Roof	30 + drift	22*

* Value found with $P_f = 0.7 C_e C_t I P_g$

P_g = ground snow load
C_t = thermal factor
C_e = snow exposure factor
I = snow load importance factor

$P_f = 0.7(0.9)(1.0)(1.1)(20) = 14 \text{ psf}$

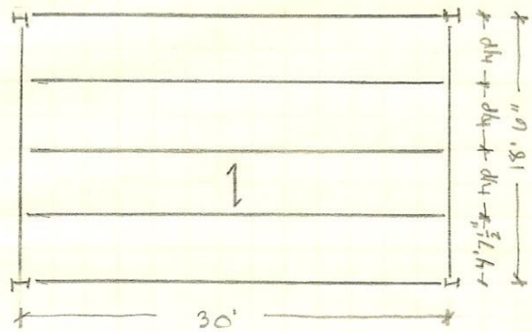
$14 \text{ psf} < 20 \text{ psf}$ ∴ NOT OK

$P_f = I P_g = 1.1(20) = 22 \text{ psf}$

Gravity Loads (cont.)

• Roof Construction

- Typical Roof Bay



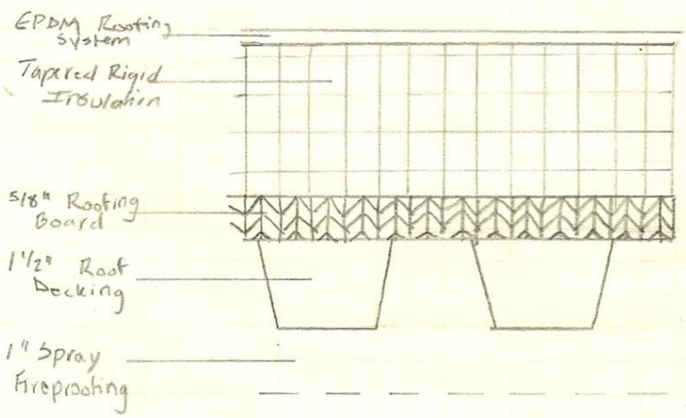
Design: 1½" wide rib galvanized steel deck

LL = 45 psf
DL = 20 (roofing) + 10 (misc. DL) + 2 (insulation) + 2 (decking) = 34 psf
SL = 30 + drift

$1.2DL + 1.6LL + 0.5SL = 1.2(34) + 1.6(45) + 0.5(30) = 128$ psf

clear span ~ 5'0" → using max const. span of 5'10" (> 5' ∴ ok ✓)
assume wide rib — allowable total load = 154 psf > 128 psf ∴ ok ✓
(excellent load carrying capacity)

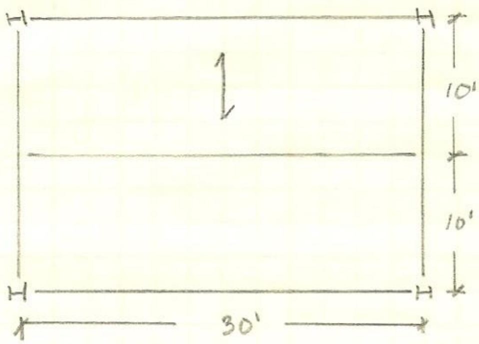
Design: 1½" wide rib ∴ ok ✓
1.46 psf < 2 psf assumed ∴ ok ✓



Gravity Loads (cont.)

• **Floor Construction**

- Typical Floor Bay → assume 4th Floor



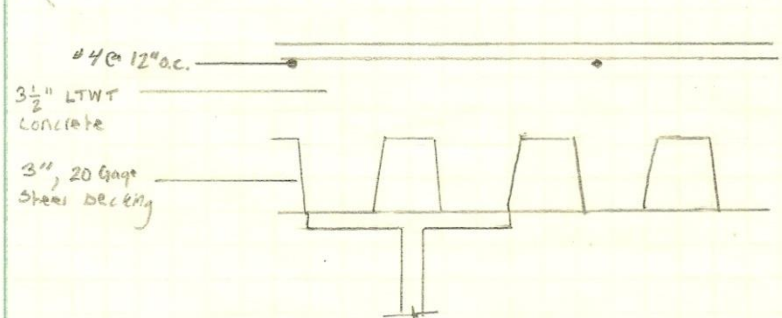
Design: 3", 20 gage decking

LL = 80 psf
DL = 15 (framing) + 10 (misc. DL) (assumed)
$1.2DL + 1.6LL = 1.2(15+10) + 1.6(80)$
$= 158$ psf

Assume: 2 hr spray fireproofing → ≥ 3 1/4" LTWT conc. topping

t = 3.25 → total slab depth = 6.25
clr span = 10', 2 spans
max span = 14' 2"
3VLI19 decking → 168 psf > 158 psf ∴ ok ✓

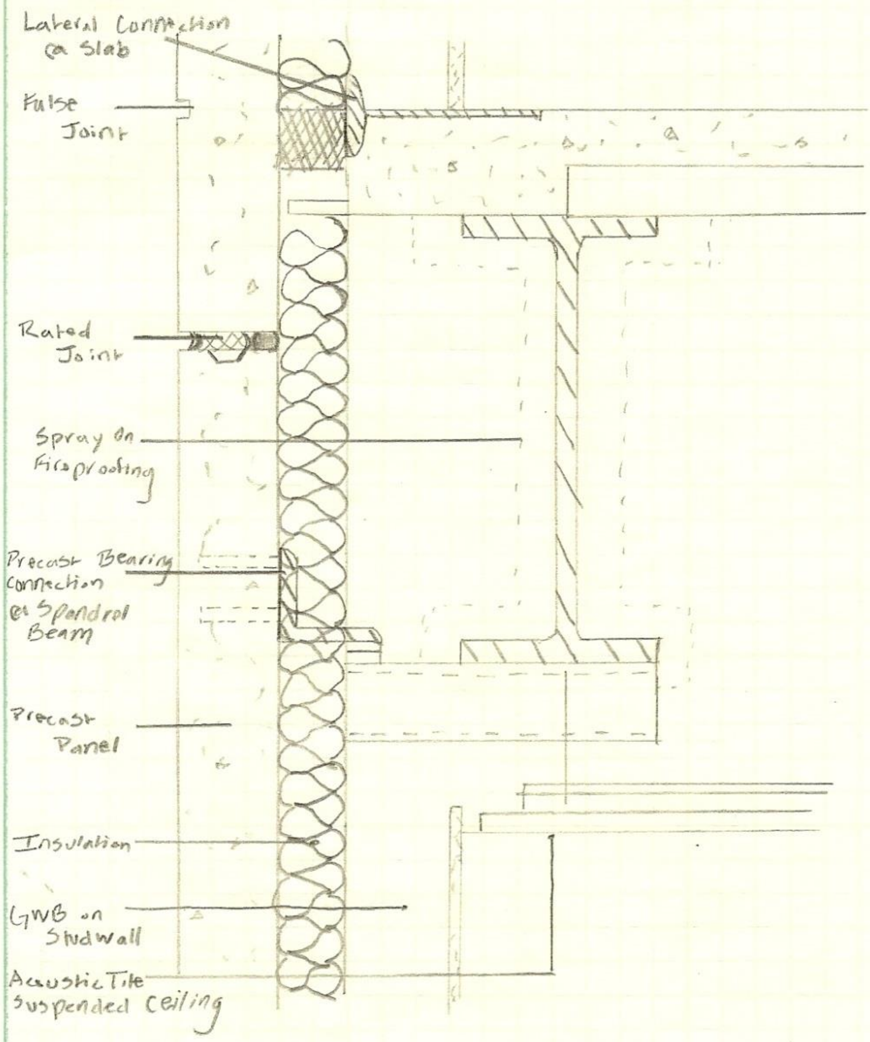
Design: 3", 20 gage + 3 1/2" LTWT conc ∴ ok ✓



Floor Construction Typical to Floors 4-12

Gravity Loads (cont.)

• Exterior Wall Construction - Pre-Cast Concrete Panels



Assumptions
- Precast panels → 130 pcf concrete
- Insulation → 4 pcf

Known
- Precast panels → 6" thick
- Insulation → 3 1/2" thick
- Panels cover roughly 60% of the building exterior

Exterior Wall Surface Areas
Side 1 = 34,800 sf
Side 2 = 17,400 sf
Side 3 = Side 1
Side 4 = Side 2 Total Surface Area = 104,400 sf

Weight = $[(130\text{pcf})(6''/12^{\text{in/ft}}) + (4\text{pcf})(3.5''/12^{\text{in/ft}})](104{,}400\text{sf})(0.6) = 4{,}145{,}00$

Total Weight of Panels for Entire Building = $\boxed{4{,}145\text{ k}}$

Gravity Loads (cont.)

· Exterior Wall Construction - (cont.)

- Glass

Assumptions
· Glass → 15psf for ~3/4" thick glass used
· Covers roughly 40% of building exterior

Weight = (15 psf)(104,400 sf)(0.4) = 626,400 lb
Total Weight of Glass for Entire Building = $\boxed{626k}$

Note: For later use in seismic calculations, weight of exterior system has been found by floor

Floor	Wall Surface Area (sf)	% Panels	% Glass	Weight (kips)
2nd	7,716	67	33	380
3rd	↑	67	33	380
4th		50	50	313
5th		↑	↑	↑
6th				
7th				
8th				
9th				
10th				
11th	↓	↓	↓	↓
12th	7,716	50	50	313
13th	10,520	100	0	696

Weight = {[(130 pcf)(0.5') + (4 pcf)(0.29')](SF)(% Panels) + (15 psf)(SF)(% Glass)} / 1000

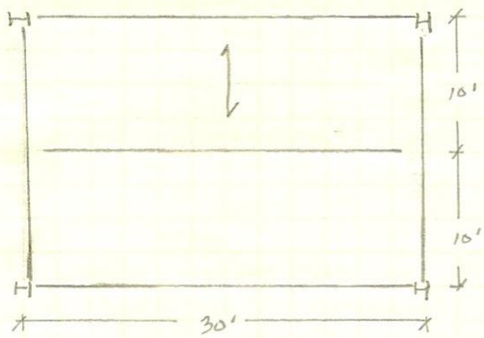
Gravity Loads (cont.)

· Non-Typical Loads

Floor/Area	Design/Assumption	Justification
Sub-basement	250/150 psf	- 150 psf was assumed based on the following: maintenance, vibration, movement, etc.
Loading Dock	350 psf	- This value was used for design. Due to the high delivery traffic and possibility of heavy point loads, it is better to be conservative.
13th Floor	150 psf	- Once again, 150 psf was assumed, but was also the design value.
Elevators @ Roof	75 psf	- Additional equipment and concrete on metal decking is required in this roughly 15' x 30' area. The value of 75 psf is an estimate based on live loads only, caused by some equipment and light maintenance.

Typical Member Spot Checks - Gravity Loads

As used in Structural Technical Report #2, the typical floor bay is sketched below. This floor bay is typical for Floors 4 thru 12 (except in areas discussed in the note below).



Note: There are slight variations in bays throughout the structure. However, the beams throughout the differing bays are in fact similar; the girders are the main difference. For this reason, two girder checks have been completed: one on the shorter typical bay (found above) and one on the much longer bay (found on subsequent pages).

- Floor Decking

Assumptions
- Vulcraft Steel Decking
  2 hr spray fireproofing → ≥ 3 1/4" LTWT Concrete topping, 3VLI
  LL = 80 psf
  DL = 15 (framing) + 10 (misc. DL) [psf]
  Total Load = 1.2DL + 1.6LL = 1.2(25) + 1.6(80) = 158 psf

total thickness = 3 1/4" + 3" = 6.25"
clear span = 10', 2 spans

3VLI19 → 168 > 158 psf ∴ ok ✓
Unshored clear span = 14' 2"

∴ Use 3VLI19 decking with 3 1/4" LTWT topping

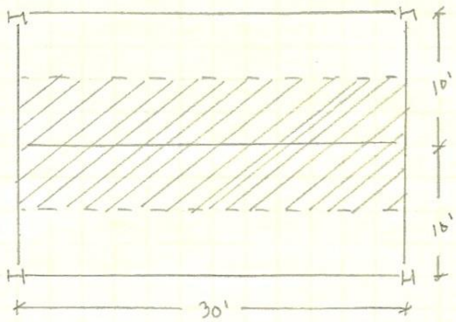
Design

3VLI20 w/ 3 1/2" LTWT topping → 157 psf ~ 158 psf ∴ ok ✓ (assumptions)

unshored clear span = 12'7" for 2 spans > 10' ∴ ok ✓
slab/deck wt = 2.14 psf + (110pcf)(3.5/12) ~ 34 psf

Typical Member Spot Checks (cont.)

· Beams



Assumptions

$LL = 80\ psf$

$DL = 15\ (framing) + 10\ (misc.\ DL) + 5\ (self\ wt) + 34\ (slab/deck) = 64\ psf$

$K_{LL} A_T = 2(30')(10') = 600\ ft^2 > 400\ ft^2$

∴ live load may be reduced

$L = LL\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 80\left(0.25 + \frac{15}{\sqrt{600}}\right)$ > 0.5, ∴ ok ✓

$L = 69\ psf$

Total Load $= 1.2DL + 1.6L = 1.2(64) + 1.6(69)$
$= \underline{187\ psf}$

$W_u = 187\ psf\,(10') = 1870/1000 = 1.87\ klf$
$V_u = 1.87\ klf\,(30')/2 = 28.1^k$
$M_u = \frac{w_u l^2}{8} = \frac{1.87\ klf\,(30')^2}{8} = 210\ ft\cdot k$

Assumptions: $a = 1.0'' \rightarrow Y2 = (slab + conc.\ topping) - a/2$
$= (3'' + 3\tfrac{1}{2}'') - 1''/2 = 6''$

PNA = located below flange $f'_c = 3\ ksi$, LTWT conc.

Possibilities →		ϕM_n (ft·k)	ΣQ_n (k)
	W18 x 35	372	129
	W16 x 26	267	96
	W14 x 26	231	96.1

LTWT Concrete + weak stud positioning → $Q_n = 17.2^k$

		Total # of Studs	Weights* (lb)
	W18 x 35	(129/17.2)(2) = 16	1210
satisfies + economical →	W16 x 26	(96/17.2)(2) = 12	900
	W14 x 26	(96.1/17.2)(2) = 12	900

* Weight = (bm wt)(l) + studs (10)

- Check a → $b_{eff} = 2 \times \min \begin{cases} (30')(12)/8 = 45'' \\ \tfrac{1}{2}(10')(12) = 60'' \end{cases} = 90''$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{(17.2k)(12/2)}{0.85(3)(90)} = 0.45'' < 1.0''$ assumed ∴ ok ✓

Typical Member Spot Checks (cont.)

- Beams (cont.)

- Check Unshored Strength → W16x26

$$W_u = 1.2[(64-5)(10') + (26\ plf)] + 1.6(20)(10') = 1059/1000 = \underline{1.06\ klf}$$

or

$$W_u = 1.4(64-5)(10') + 1.4(26) = 862/1000 = 0.86\ klf \quad \text{max controls}$$

$$M_u = \frac{W_u l^2}{8} = \frac{1.06(30)^2}{8} = 120\ ft\cdot k < 267\ ft\cdot k = \phi M_n \text{ for W16x26} \therefore ok ✓$$

- Check Wet concrete Δ →

$$W_{wc} = (59)(10') + 26 = 616\ plf$$
$$\Delta_{wc} \le l/240 \le 30(12)/240 = 1.5''$$

$$\Delta_{wc} = \frac{5wl^4}{384EI} = \frac{5(0.616)(30)^4(1728)}{384(29000)I} \le 1.5'' \longrightarrow \therefore I \ge 258\ in^4$$

$$I \text{ for W16x26} = 301\ in^4 > 258\ in^4 \therefore ok ✓$$

- Check LL Δ →

$$W_{LL} = 69\ psf(10') = 690\ plf = 0.69\ klf$$
$$\Delta_{LL} \le L/360 \le 30(12)/360 = 1.0''$$
$$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(0.69)(30)^4(1728)}{384(29000)(301)} = 1.44'' > 1.0'' \therefore \text{NOT OK}$$

TRY W18x35 $\longrightarrow \frac{5(0.69)(30)^4(1728)}{384(29000)(510)} = 0.85'' < 1.0'' \therefore ok ✓$

- check a: $a = \frac{(17.2)(1612)}{0.85(3)(90)} = 0.6'' < 1''$ assumed $\therefore$ ok ✓

- check strength: $W_u = 1.2[(59)(10) + 35] + 1.6(20)(10) = \underline{1.07\ klf}$

$$W_u = 1.4(59)(10) + 1.4(35) = \underline{0.88\ klf}$$

$$M_u = \frac{1.07(30)^2}{8} = 120\ ft\cdot k < \phi M_n = 372\ ft\cdot k \therefore ok ✓$$

- check Δ_{wc}: $W_{wc} = 59(10) + 35 = 625\ plf$

$$\Delta_{wc} = \frac{5(0.625)(30)^4(1728)}{384(29000)I} \le 1.5 \longrightarrow I \ge 261\ in^4$$

$$I \text{ for W18x35} = 510\ in^4 \therefore ok ✓$$

Summary of Design → $\boxed{\text{use W18x35 w/ 16 studs/bm spaced evenly along length}}$

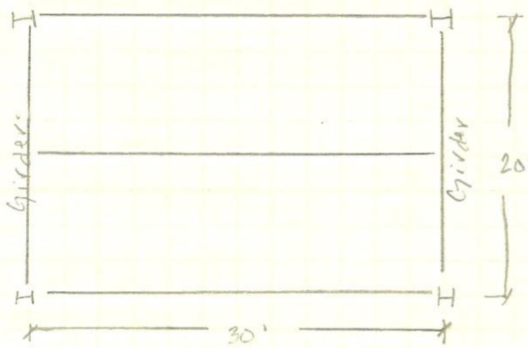
Typical Member Spot Checks (cont.)

· Beams (cont.)

Actual Design
W18x35 [16] for typical bay used (Floors 4 - 12 of building)
∴ ok✓

· Girders

Typical Bay #1 - Short



Assumptions
LL = 80 psf
DL = 15 + 10 + 5 + 34 = 64 psf
(framing) (misc.) (self) (slab/deck)
$K_{LL} A_T = 2(20')(30') = 1200 ft^2 > 400 ft^2$
∴ live load may be reduced

$L = LL\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 80\left(0.25 + \frac{15}{\sqrt{1200}}\right)$ > 0.5, ∴ ok✓

L = 55 psf

$P = [1.2(64)(30)(10) + 1.6(55)(30)(10)] = 49440/1000 = 49.4^k$
$M_u = 49.4(10) = 494$ ft·k

Assumptions: a = 1.0" ⟶ $y_2 = 6"$
PNA = located below flange, $f'_c = 3$ ksi, LTWT conc.

		ϕM_n (ft·k)	ΣQ_n (k)
Possibilities ⟶	W21x44	530	163
	W18x60	664	226
	W18x50	550	184
	W18x46	500	169

LTWT concrete + $f'_c = 3$ ksi ⟶ $Q_n = 17.2^k$

		Total # of Studs	Weight (lb)
satisfies + economical ⟶	W21x44	(163/17.2)(2) = 20	1080
	W18x60	(226/17.2)(2) = 26	1460
	W18x50	(184/17.2)(2) = 22	1220
	W18x46	(169/17.2)(2) = 20	1120

- Check a ⟶ $b_{eff} = 2 \times \min \begin{cases} (20')(12)/8 = 30" \\ 1/2(30')(12) = 180" \end{cases} = 60"$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{17.2(20/2)}{0.85(3)(60)} = 1.12" > 1.0"$ assumed ∴ NOT OK

Typical Member Spot checks (cont.)

· Girders (cont.)

- Assume $a = 2.0'' \rightarrow Y_2 = 5.5''$

	ϕM_n (ft·k)	ΣQ_n (k)	# studs	Weight (lb)
→ W21x44	524	163	20	1080
W18x60	656	220	26	1460
W18x50	543	184	22	1220
W18x46	494	169	20	1120

$a = 1.12'' < 2.0''$ ∴ ok ✓

- Check Unshored Strength → W21x44

$P_u = [(1.2)(59psf)(10') + (1.2)(44) + 1.6(20)(10)](30') = 32.4^k$
$M_u = 324$ ft·k $< \phi M_n = 524$ ft·k ∴ ok ✓

- Check Wet Concrete Δ →

$P_{wc} = (59 + 5)(30)(10) = 19.2^k$ $\quad \Delta_{wc} \doteq L/240 \doteq 20(12)/240 = 1''$

$\Delta_{wc} = \frac{PL^3}{28EI} = \frac{(19.2)(20)^3(1728)}{28(29000)(843)} = 0.4'' < 1''$ ∴ ok ✓

- Check LL Δ →

$P_{LL} = (55)(30)(10)/1000 = 16.5^k$
$\Delta_{LL} \doteq L/360 \doteq 20(12)/360 = 0.67''$

$\Delta_{LL} = \frac{PL^3}{28EI} = \frac{(16.5)(20)^3(1728)}{28(29000)(843)} = 0.33'' < 0.67''$ ∴ ok ✓

Summary of Design → $\boxed{\text{use W21x44 w/ 20 studs/girder}}$

Actual Design → W18 x 65 w/ 6 studs/girder

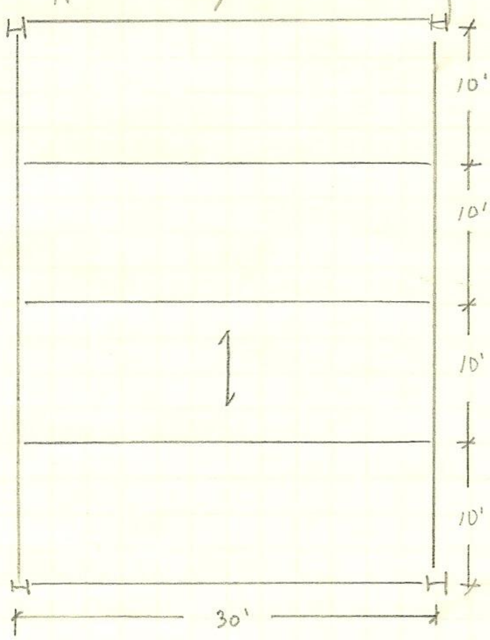
$I = 1070 > 843$ used
$\phi M_n = 503 > 324$ ft·k needed ∴ ok ✓

Possible Reasoning → wanted to keep system height continuous with beams on floors where only this "short" bay is used

Typical Member Spot Checks (cont.)

- Girders (cont.)

Typical Bay #2 - Long



Assumptions
LL = 80 psf
DL = 30 + 10 + 10 + 34 = 84 psf
increased framing + self wt

$K_{LL} A_T = 2(40')(30') = 2400\ ft^2$

$L = LL\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 80\left(0.25 + \frac{15}{\sqrt{2400}}\right)$ > 0.5, ∴ ok ✓

L = 45 psf

$P = [1.2(84)(30)(10) + 1.6(45)(30)(10)]$
$= 51.8^k$
$M_u = 518^{ft\cdot k}$

$a = 2.0'' \longrightarrow Y_2 = 5.5''$
$f'_c = 3\ ksi$, LTWT conc.

- Check LL Δ 1st → (will most likely control)

$P_{LL} = (45)(30)(10) = 13.5^k$
$\Delta_{LL} \leq l/360 \leq (40)(12)/360 = 1.33''$

$\Delta_{LL} = \frac{Pl^3}{28EI} \leq 1.33''$ $\qquad I \geq \frac{(13.5)(40)^3(1728)}{28(29000)(1.33)} = 1382\ in^4$

		ϕM_n (ft·k)	ΣQ_n (k)	# Studs	Weight (lb)
Possibilities →	W30 x 90	1470	329	40	4000
	W27 x 84	1270	309	36	3720
	W24 x 76	1050	280	34	3380
	W21 x 68	844	250	30	3020
satisfies + economical →	W24 x 62	824	228	28	2760

- Check a → $b_{eff} = 2 \times \min \begin{cases} (40')(12)/8 = 60'' \\ 1/2(30')(12) = 180'' \end{cases} = 120''$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{17.2(28/2)}{0.85(3)(120)} = 0.79'' < 2.0''$ assumed ∴ OK ✓

- Check Unshored Strength →

$P_u = [(1.2)(74)(10) + (1.2)(62) + 1.6(20)(10)](30) = 38.5^k$

Typical Member Spot Checks (cont.)

· Girders (cont.)

$P_u = 38.5^k$
$M_u = 385^k < 824^k \quad \therefore$ ok ✓

- Check Wet Concrete $\Delta \rightarrow$

$P_{wc} = (74 + 5)(30)(10) = 23.7^k$
$\Delta_{wc} \leq l/240 \leq (40)(12)/240 = 2''$ $\qquad I$ for $W24 \times 62 = 1550 \text{ in}^4$

$\Delta_{wc} = \dfrac{PL^3}{28EI} = \dfrac{(23.7)(40)^3(1728)}{28(29000)(1550)} = 2.1'' > 2'' \quad \therefore$ NOT OK

TRY W24 x 76 $\rightarrow I = 2100 \quad \therefore \Delta_{wc} = 1.5'' < 2'' \quad \therefore$ ok ✓

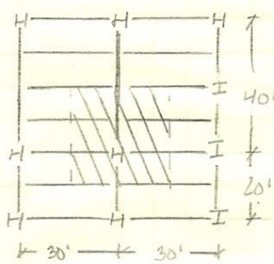
$\Delta_{LL} = 0.88'' < 1.33'' \quad \therefore$ ok ✓
$a = 0.95'' < 2'' \quad \therefore$ ok ✓
$M_u = 385^k < 1050^k \quad \therefore$ ok ✓

Summary of Design $\rightarrow$ $\boxed{\text{use W24 x 76 w/ 34 studs/girder}}$

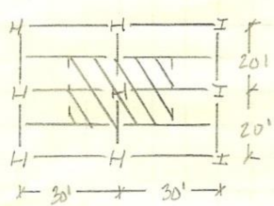
Actual Design $\rightarrow$ W18 x 211 [36]

Both I and ϕM_n greatly exceed values found, $\therefore$ ok ✓

· Interior Column



for Floors 6 - Roof



for Floors 4-5

Located on 4th Floor at Column Lines (5) and (B); lowest typically framed floor (floors below contain auditorium/lobby space)

Assumptions

LL = 80 psf (Floors 4-12)
150 psf (13th)
~~20 psf~~ (Roof) < SL

Snow Load = 30 psf
DL = 15 (framing) + 10 (misc.) + 5 (self wt) + 34 (slab/deck) = 64 psf
Roof DL = 20 (roofing) + 2 (decking) + 2 (insulation) + 10 (misc.) = 34 psf

· Typical Member Spot Checks (cont.)

· Interior Column (cont.)

$$L_{4-5} = L_o + \Big|_{max}^{0.4}\; 0.25 + \frac{15}{\sqrt{4(2)(30)(20)}} = L_o(0.47) = 80(0.47) = 37.3\text{ psf}$$

$$L_{6-12} = L_o + \Big|_{max}^{0.4}\; 0.25 + \frac{15}{\sqrt{4(7)(30)(30)}} = L_o(0.4) = 80(0.4) = 32\text{ psf}$$

$$L_{13} = L_o + \Big|_{max}^{0.4}\; 0.25 + \frac{15}{\sqrt{4(30)(30)}} = L_o(0.5) = 150(0.5) = 75\text{ psf}$$

$$P_u = [1.2DL + 1.6LL + 0.5SL]A_T$$

$$P_{u\,4-5} = \{1.2(64)(2) + 1.6(37.3)(2)\}(30\times 20) = 164^k$$

$$P_{u\,6\text{-Roof}} = \{1.2[(64)(8) + 34] + 1.6[(32)(7) + 75] + 0.5(30)\}(30\times 30) = 1034^k$$

$P_{u\,TOTAL} = 1198^k$, braced at 20' lengths $\rightarrow$ $\boxed{\text{use W14×132}}$

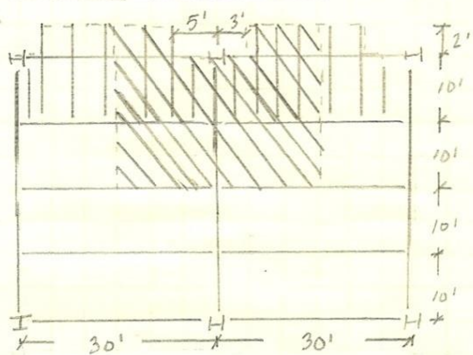
Summary of Design $\rightarrow$ $\boxed{\text{W14×132 braced at 20' lengths}}$

Actual Design $\rightarrow$ W14×145 braced at 28' lengths
$\phi P_n = 1140^k \sim P_{TOTAL} = 1198^k$ found w/ assumptions

∴ OK ✓

· Exterior Column

Floors 6-Roof

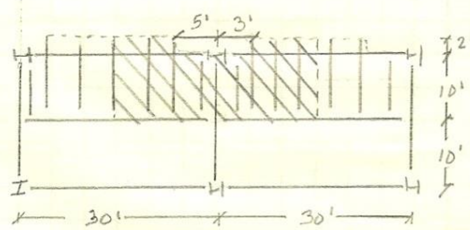


Extends to Basement Level, located at column lines (A) and (4)

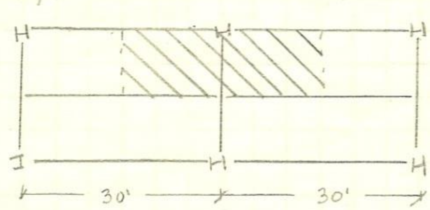
Assumptions
- LL = 100 psf (1st)
  - 60 " (2-3)
  - 80 " (4-12)
  - 150 " (13)
- SL = 30 psf
- DL = 64 psf + 8 psf (additional cantilever beams)
- Roof DL = 34 psf

Ext. Wall:
Panels = [(130)(0.5) + (4)(0.29)] = 66.2 psf
Glass = 15 psf

Floors 4-5



Floors 1-3



Typical Member Spot Checks (cont.)

• Exterior Column (cont.)

$$L_1 = L_o \left|\begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(30)(10)}} \end{array}\right._{max} = 100(0.683) = 68.3\text{psf}$$

$$L_{2-3} = L_o \times \left|\begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(2)(30)(10)}} \end{array}\right._{max} = 60(0.56) = 33.4\text{psf}$$

$$L_{4-5} = L_o \times \left|\begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(2)(300+24+20)}} \end{array}\right._{max} = 80(0.54) = 42.9\text{psf}$$

$$L_{6-12} = L_o \times \left|\begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(7)(600+24+20)}} \end{array}\right._{max} = 80(0.4) = 32.0\text{psf}$$

$$L_{13} = L_o \times \left|\begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(600+24+20)}} \end{array}\right._{max} = 150(0.55) = 81.8\text{psf}$$

$$P_{u_1} = \{1.2(64) + 1.6(68.3)\}(30)(10) + 1.2(15)(30-8)(14.67) + 1.2(66.2)(8)(14.67) = \underline{71^k}$$

$$P_{u_{2-3}} = \{1.2(64)(2) + 1.6(33.4)(2)\}(30)(10) + 1.2(66.2)(30)(14.67)(2) = \underline{148^k}$$

$$P_{u_{4-5}} = \{1.2(72)(2) + 1.6(42.9)(2)\}(300+24+20) + 1.2(15)(22+4)(14.67)(2) + 1.2(66.2)(8)(14.67)(2) = \underline{139^k}$$

$$P_{u_{6-Roof}} = \{1.2[(72)(8) + 34] + 1.6[(32)(7) + 81.8] + 0.5(30)\}(600+24+20) + 1.2(15)(22+4)(14.67)(7) + 1.2(66.2)(8)(14.67)(7) + 1.2(66.2)(30)(24) = \underline{967^k}$$

$P_{u_{TOTAL}} = 1325^k$, braced at 40' lengths $\longrightarrow$ W14 x 283

Summary of Design $\longrightarrow$ $\boxed{\text{W14 x 283 braced at 40' lengths}}$

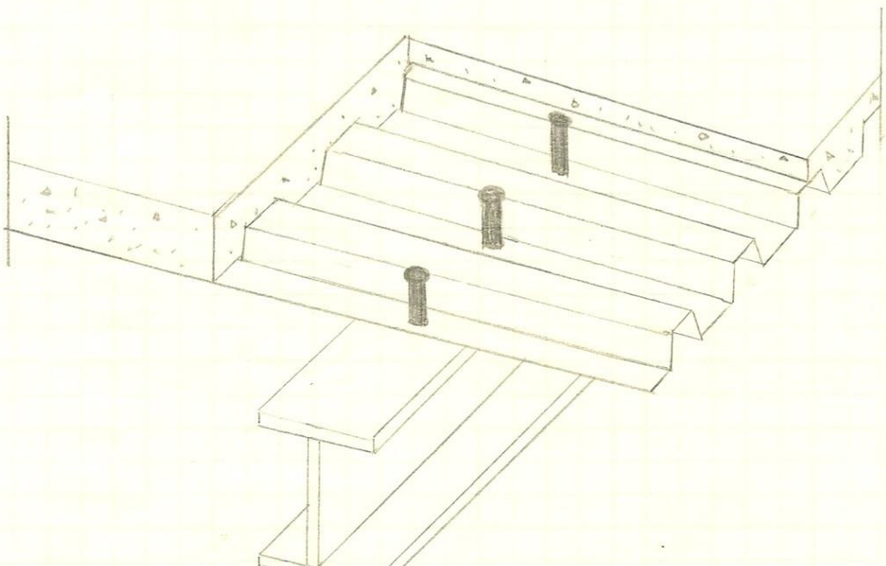
Actual Design $\longrightarrow$ W14 x 398 braced at 20' lengths

However, adjacent column is in fact a W14 x 283. Misjudgement could be caused by assumption of loads, especially for Dead Loads from exterior wall system.

Alternate Systems Study

I. Metal Deck Composite Beam System
II. Non-Composite Steel System with Bar Joists
III. Pre-Cast Concrete on Structural Steel
IV One-Way Reinforced Slab with Beams/Girders

I. Metal Deck Composite Beam System



System Description

As analyzed in the previous section, the VCU SOM project utilizes a Metal Deck Composite Beam/Girder System (a simplified sketch is shown above). There are two differently sized bays that constitute the framing for Floors 4 through 13. Summaries for both bays (referred to as Short and Long to decrease confusion) are found below.

"Short" Bay Summary
Deck – 3", 20 gage (3VLI20)
Topping – 3½" LTWT Concrete
Beams – W18x35 with 16 studs
Girders – W18x65 with 6 studs

"Long" Bay Summary
Deck – 3", 20 gage (3VLI20)
Topping – 3½" LTWT Concrete
Beams – W18x35 with 16 studs
Girders – W18x211 with 36 studs

I. Metal Deck Composite Beam System (cont.)

Advantages
- topping provides fireproofing for deck
- designed for unshored construction → less money
- less time to erect steel
- steel can span larger areas, which provides more open space
- composite action increases strength

Disadvantages
- installation of studs requires additional skilled labor
- requires more inspections due to complexity
- beams/girders still require fireproofing

Cost → Moderate (typically between 16 and 19 $/SF)

Size and Weight

Total Depth = 3" + 3 1/2" + 20 5/8" = 27 1/8"

Weight = 52 psf

II. Non-Composite Steel System with Bar Joists

System Description

Since the current system in-place utilizes steel, it seemed necessary to consider other structural steel systems as well. The Non-Composite Steel System examined was designed to accomodate the larger classrooms, meaning it was designed to be a 30' x 40' bay. To maintain the necessary 2 hr fire rating, $2\frac{1}{2}$" concrete topping was used on 1.0C24 decking. The bar joists were placed at 4', creating 10 spans within a bay. The girder chosen was designed to be economical while also meeting deflection constraints.

Summary of Design

Slab/Deck: 1.0C24 w/ $2\frac{1}{2}$" topping (depth = 3½")
Bar Joists: 24K9 @ 4'
Girder: W24 x 146

Advantages

- is capable of clearly spanning longer areas
- can be lighter than composite steel construction
- allows for room in ceiling for navigating MEP equipment

Disadvantages

- still require additional fireproofing for steel members
- longer members achieved by design can become pricey
- longer spans increase possibility of vibrations in floor

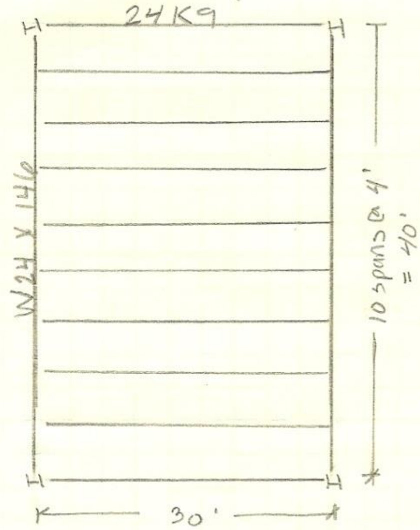
Cost → Moderate (typically between 16 and 20 $/SF)

Size and Weight

Total Depth = 24.7" + 3.5" = 28.2"

Weight = 53.9 psf

II. Non-Composite Steel System with Bar Joists (cont.)



Assumptions
to achieve 2 hr fire rating → $2\frac{1}{2}$" NW CONC topping
(will still require spray fireproofing on bar joists + girders)

LL = 80 psf
DL = 10 psf (misc., does not include slab/deck or self wt)
Total Load = 1.2(10) + 1.6(80) = 140 psf

clr span = 4', 2 spans

1.0 C 24 → 153 psf > 140 psf ∴ ok ✓
clr span = 5' 9" > 4' ∴ ok ✓
Weight = 37 psf

Total Load = 1.2(37) + 140 = 184 psf

W_{t+L} = (184)(4) = 736 lb/ft + joist wt W_{+L} = (37 + 10 + 80)(4) = 508 lb/ft + joist wt

Possibilities →	20K10	799	12.2	336
	24K9	816	12	419
	26K8	816	12.1	457
	30K7	825	12.3	543

TRY 20K10 → 736 + (12.2)(1.2) = 751 < 799 ∴ ok ✓
w for $L/240$ = 336(1.5) = 504 lb/ft < 508 ∴ NO GOOD

TRY 24K9 → 736 + 12(1.2) = 750 < 799 ∴ ok ✓
w for $L/240$ = 419(1.5) = 629 > 508 ∴ ok ✓

$\boxed{\therefore \text{ use 24K9 bar joist at 4'}}$

LL = 80 psf
DL = 47 psf

$W_u = [1.2(47) + 1.6(80)](40) = 7.38$ klf
$M_u = w_u L^2/8 = (7.38)(40)^2/8 = 1476$ ft·k

		ϕM_n	I
Possibilities →	W33 x 180	1560	5900
	W30 x 124	1530	5360
	W18 x 175	1490	3450
	W24 x 146	1570	4580

$\Delta = L/240 = (40)(12)/240 = 2"$ $\Delta_{LL} = L/360 = (40)(12)/360 = 1.33"$

TRY W24 x 146 → $\Delta = \frac{5 w_u L^4}{384 EI} = \frac{5(\frac{80}{1000})(30)(40)^4(1728)}{384(29000)(4580)} = 1.04" > 1.33"$ ∴ ok ✓

$\boxed{\therefore \text{ use W24 x 146 girders}}$

III. Pre-Cast Concrete on Structural Steel

System Description

Continuing with using structural steel, this system introduces pre-cast hollowcore concrete planks into the investigation. Using Nitterhouse concrete products, 8" x 4'-0" planks were selected to span 20' lengths. The beam and girder layout remains the same as the original "long" system. To account for the added weight of the planks, both beam and girder sizes were increased.

Summary of Design

Slab: 8" x 4' Hollowcore Pre-Stressed Concrete Planks w/ additional 2" topping

Beams: W21 x 44

Girders: W33 x 130

Advantages

- easier to construct → no pouring, welding
- provides extremely good deflection control

Disadvantages

- requires increased coordination
- is much heavier than decking + topping
- will still require significant depth

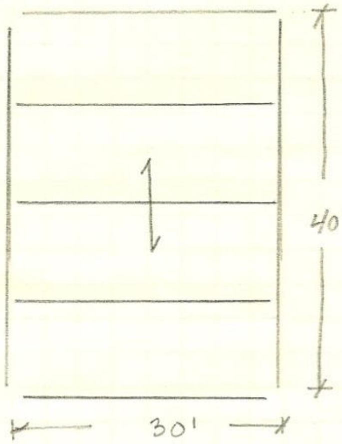
Cost → High (is typically greater than 22 $/SF)

Size and Weight

Total Depth = 43"

Weight = 64.5 psf

III. Pre-Cast Concrete on Structural Steel (cont.)



Assumptions
- 2 hr fireproofing needed
- NW conc in precast hollowcores
- will span in direction shown
- planks come in 4' widths

DL = 15 (framing) + 10 (misc.) = 25 psf
LL = 80 psf

Total Load = 1.2 (25) + 1.6 (80) = 158 psf

TRY 8" x 4' Plank → @ 20' span
4 - 1/2" ⌀ → 185 psf > 158 psf
∴ OK ✓

Check Δ: $\Delta_{LL} = \frac{5 W_u L^4}{384 E I} = \frac{5\left[1.6\left(\frac{80}{1000}\right)4\right](20)^4(1728)}{384(4415)(3134)} = 0.133''$

$\Delta_{max} = L/360 = (20)(12)/360 = 0.67'' > 0.133''$ ∴ OK ✓

$\boxed{\therefore \text{ use } 8'' \times 4' \text{ Hollowcore Planks @ } 4\text{-}\tfrac{1}{2}''\,⌀}$

DL = 25 psf + 61.25 psf = 86.3 psf
← from plank info

LL = 80 psf
$L = 80\left(0.25 + \frac{15}{\sqrt{600}}\right)$
L = 69 psf

$W_u = [(1.2)(86.3) + (1.6)(69)](10) = 2140$ lb/ft = 2.14 klf

$M_u = \frac{W_u L^2}{8} = (2.14)(30)^2/8 = 241$ ft·k

		ϕM_n	I
Possibilities →	W18 x 35	249	510
	W16 x 40	274	518
	W14 x 43	261	428

$\Delta \le L/360 \le (30)(12)/360 = 1.0''$

$\Delta_{LL} = \frac{5 w L^4}{384 E I} = \frac{5[1.6(69/1{,}000)(10)](30)^4(1728)}{384(29000)\,I} \le 1.0''$

∴ I ≥ 693, so NO to beams above

		ϕM_n	I
Possibilities →	W21x44	358	843
	W18x50	379	800
	W16x57	394	758

$\boxed{\therefore \text{ use W21x44 for beams}}$

$W_u = 2140$ lb/ft + (1320 #)(3)/(40') = 2.24 klf

$M_u = \frac{W_u L^2}{8} = (2.24)(40)^2/8 = 448$ ft·k

III. Pre-Cast Concrete on Structural Steel (cont.)

$\Delta \le l/360 \le (40)(12)/360 = 1.33''$

$\Delta_{LL} = \frac{5 w l^4}{384 E I} \le 1.33''$ $\therefore I \ge 6578 \text{ in}^4$ ← will control

Possibilities →

	ϕM_n	I
W33 x 130	1750	6710
W30 x 148	1880	6680
W36 x 135	1910	7800

$\boxed{\therefore \text{ use W33 x 130 for girders}}$

IV One-Way Reinforced Slab with Beams/Girders

System Description

The One-Way Slab w/ Beams/Girders System is a more divergent option from the original scheme, making it that much more important to consider. All calculations can be found on the following pages. Unlike the current design of the structure, all concrete was assumed to be normal weight. Other assumptions can be found with the calculations. The resulting system is summarized below and the advantages/disadvantages are listed.

One-Way Slab w/ Beams Summary

Slab - 5" thick with #4 bars
Beams - 12" x 21" with #8 bars (top + bottom)
Girders - 12" x 14" with #6 bars (top + bottom)

Advantages

- No additional fireproofing is required
- System components are smaller, resulting in an increased floor-to-ceiling height
- typically less expensive than steel

Disadvantages

- Requires formwork and more labor → increase in time needed and money spent
- While beam/girder size may decrease, system will require much larger columns than steel construction
- bay size used (to decrease floor-to-floor ht) increases # of columns

Cost → Moderate (typically between 15 and 18 $/SF)

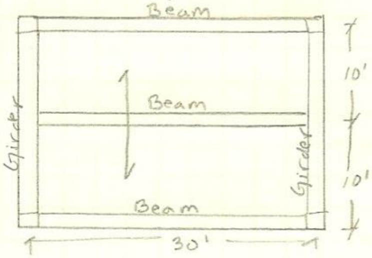
Size

Slab → t = 5" w/ #4 bars @ 12 o.c. (btm & top)
Beams → 12" x 21" w/ (4) #8 bars (top) + (3) #8 bars (btm)
Girders → 12" x 14" w/ (4) #6 bars (top) + (3) #6 bars (btm)

Total Depth = 26" = 2' 2"

Weight ~ 110 psf

IV. One-Way Reinforced Slab with Beams/Girders (cont.)



Assumptions

NW Concrete
$f'_c = 4000$ psi
$f_y = 60$ ksi
2hr fire rating

LL = 80 psf
DL = 10 psf (misc.)

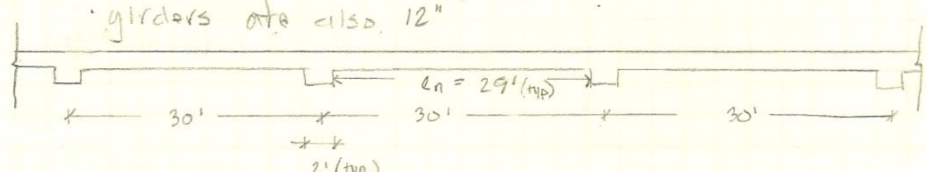
- Minimum Thickness of Slab
  - both ends are continuous → $\ell/28 = (10')(12''/1')/28 = 4.29''$
  - min. thickness for 2 hr rating → 4.6" min. , ∴ use $h_f = 5''$
- Loads

  $5''/12 \times 150$ pcf (NW CONC) $= 62.5$ psf

  Total Load $= 1.2[10 + 62.5] + 1.6[80] = 215$ psf

  $W_u = (215\text{ psf})(10') = 2150$ lb/ft $= 2.15$ klf

- Moments
  - Assumptions
    - columns are 12" x 12"
    - more than two spans, continuous interior supports
    - addressing interior spans
    - girders are also 12"

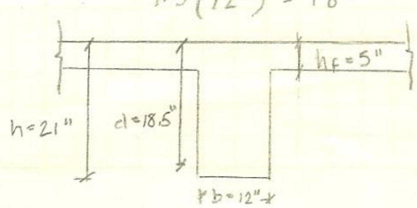


$M_u = w_u \ell_n^2/11 = (2.15\text{ klf})(29')^2/11 = 164$ ft·k {ACI 318-11 §8.3.3}

- Estimate Beam Size
  - Assumptions
    1.5b : d

  $b(1.5b)^2 = 20(164) \rightarrow b \sim 11.3'' \rightarrow$ TRY $b = 12''$

  $1.5(12'') = 18''$ $h = 18'' + \underset{\text{cover}}{1.5''} + \underset{\text{stirrups}}{1.0''} = 20.5'' \rightarrow$ use 21"



d = 18.5"
b = 12"

IV. One-Way Reinforced Slab with Beams/Girders (cont.)

- Calculate self wt.

$$\frac{(12'')(21'')}{144} \times \underset{(NW\ conc)}{150\ pcf} = 263\ lb/ft$$

new $w_u = 1.2(263) + 2150 = 2466\ lb/ft = 2.47\ klf$

$M_u^- = w_u l_n^2/11 = (2.57\ klf)(29')^2/11 = 196\ ft\cdot k$

$M_u^+ = w_u l_n^2/16 = (2.57\ klf)(29')^2/16 = 135\ ft\cdot k$

- Reinforcement
  - Bottom Reinforcement

$$A_s = \frac{M_u^+}{4d} = \frac{(135\ ft\cdot k)}{4(18.5'')} = 1.82\ in^2 \rightarrow \text{TRY } (3)\#8 \rightarrow A_s = 2.37\ in^2$$

· check d: $h = 21'' \rightarrow d = 21'' - 1.5'' - 0.375'' - (1.0/2) = 18.6''$

· check b: $b_{min} = 2(1.5'') + 2(0.375'') + 3(1.0'') + 2(1.0'') = 8.8'' < 12''$ ∴ ok ✓

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(2.37 in^2)(60000\ psi)}{0.85(4000\ psi)(12)} = 3.49'' \qquad c = \frac{a}{\beta_1} = \frac{3.49}{0.85} = 4.1''$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{4.1}(18.6 - 4.1) = 0.011 > \varepsilon_y \quad \therefore \text{ok ✓}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (2.37)(60)[18.6 - (3.49/2)]/12 = 200\ ft\cdot k > 135\ ft\cdot k \quad \therefore \text{ok ✓}$$

$$A_{s\,min} \geq \begin{cases} \frac{3\sqrt{f'_c}}{f_y} bd = \frac{3\sqrt{4000}}{60000}(12)(18.6) = 0.71\ in^2 \\ \frac{200}{f_y} bd = \frac{200}{60000}(12)(18.6) = 0.74\ in^2 \end{cases} << 2.37\ in^2 \quad \therefore \text{ok ✓}$$

$\rightarrow \boxed{\therefore \text{ use } (3)\#8 \text{ bars}}$

- Top Reinforcement

$$A_s = \frac{M_u^-}{4d} = \frac{(196\ ft\cdot k)}{4(18.5'')} = 2.64\ in^2 \rightarrow \text{TRY } (4)\#8 \rightarrow A_s = 3.16\ in^2$$

· check d: $h = 21'' \rightarrow d = 21'' - 1.5'' - 0.375'' - (1/2) = 18.6''$

· check b: $b_{min} = 2(1.5'') + 2(0.375'') + 4(1.'') + 3(1.'') = 10.8'' < 12''$ ∴ ok ✓

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(3.16 in^2)(60000\ psi)}{0.85(4000\ psi)(12'')} = 4.65'' \qquad c = \frac{a}{\beta_1} = \frac{4.65}{0.85} = 5.47''$$

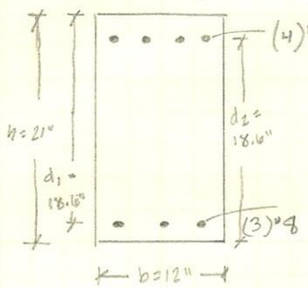
$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{5.47}(18.6 - 5.47) = 0.007 > \varepsilon_y \quad \therefore \text{ok ✓}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (3.16)(60)[18.6 - (4.65/2)]/12 = 257\ ft\cdot k > 196\ ft\cdot k \quad \therefore \text{ok ✓}$$

$$A_{s\,min} \geq \begin{cases} \frac{3\sqrt{f'_c}}{f_y} bd = \frac{3\sqrt{4000}}{60000}(12)(18.6) = 0.71 \\ \frac{200}{f_y} bd = \frac{200}{60000}(12)(18.6) = 0.74 \end{cases} << 3.16 \quad \therefore \text{ok ✓}$$

$\rightarrow \boxed{\therefore \text{ use } (4)\#8 \text{ bars}}$

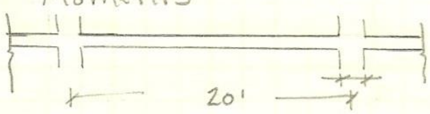
IV. One-Way Reinforced Slab with Beams/Girders (cont.)



· Check Deflections → $\ell/28 = (30)(12)/28 = 13"$
$13" << 21"$ ∴ OK ✓
{ACI 318-11 §9.5}

Summary of Beams
12" x 21"
with (4) #8 top
(3) #8 btm

- Moments



Beam → $2.47\ \text{klf}(30'/2) = 37^k$

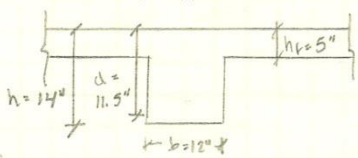
$37^k/(20' - 1') = 1.95\ \text{klf}$

$M_u^- = w_u \ell_n^2/11 = (1.95\ \text{klf})(19')^2/11 = 64\ \text{ft}\cdot\text{k}$
$M_u^+ = w_u \ell_n^2/16 = (1.95\ \text{klf})(17')^2/16 = 44\ \text{ft}\cdot\text{k}$

- Estimate Girder Size
Assumptions
girders have b = 12"

$d^2 = \frac{20(64)}{12} = 10.3 \longrightarrow$ TRY d = 11"

h = 11" + 1.5" + 1.0" = 13.5" → use 14", d = 11.5"
b = 12"



- Calculate self wt

$\frac{(12")(14")}{144} \times 150\ \text{pcf}$ (NW CONC) $= 175\ \text{lb/ft}$

new $W_u = 1.2(175) + 1950 = 2160\ \text{lb/ft} = 2.16\ \text{klf}$

$M_u^- = w_u \ell_n^2/11 = (2.16\ \text{klf})(19')^2/11 = 71\ \text{ftk}$
$M_u^+ = w_u \ell_n^2/16 = (2.16\ \text{klf})(19')^2/16 = 49\ \text{ftk}$

- Reinforcement
- Bottom

$A_s = \frac{49}{4(11.5)} = 1.06\ \text{in}^2 \longrightarrow$ TRY (3) #6 → $A_s = 1.32\ \text{in}^2$

h = 14" → d = 14" − 1.5" − 0.375" − (0.75"/2) = 11.75"
$b_{min} = 2(1.5") + 2(0.375") + 3(0.75") + 2(1.0") = 8" < 12"$ ∴ OK ✓

IV. One-Way Reinforced Slab with Beams/Girders (cont.)

$a = \frac{(1.32)(60000)}{0.85(4000)(12)} = 1.94"$ $\quad c = \frac{a}{\beta_1} = \frac{1.94}{0.85} = 2.28"$

$\varepsilon_s = \frac{0.003}{2.28}(11.75 - 2.28) = 0.012 > \varepsilon_y \therefore$ ok ✓

$\phi M_n = (1.32)(60)(11.75 - 1.94/2)/12 = 71$ ft·k > 49 ft·k $\therefore$ ok ✓

$A_{s,min} \geq \begin{cases} \frac{3\sqrt{4000}}{60000}(12)(11.75) = 0.45\ in^2 \\ \frac{200}{60000}(12)(11.75) = 0.47\ in^2 \end{cases} << 1.32\ in^2 \therefore$ ok ✓

$\longrightarrow \boxed{\therefore \text{ use (3) \#6 bars}}$

- Top

$A_s = \frac{71}{4(11.5)} = 1.54\ in^2 \longrightarrow$ TRY (4) #6 $\rightarrow A_s = 1.76\ in^2$

$h = 14" \longrightarrow d = 11.75"$

$b_{min} = 2(1.5") + 2(0.375") + 4(0.75") + 3(1.0") = 9.8" < 12" \therefore$ ok ✓

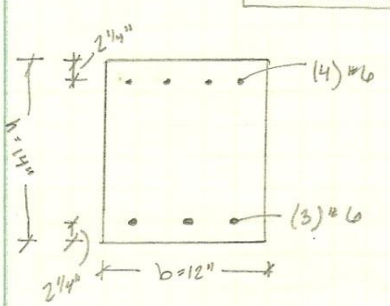
$a = \frac{(1.76)(60000)}{0.85(4000)(12)} = 2.6"$ $\quad c = \frac{2.6}{0.85} = 3.0"$

$\varepsilon_s = \frac{0.003}{3}(11.75 - 3) = 0.009 > \varepsilon_y \therefore$ ok ✓

$\phi M_n = (1.76)(60)(11.75 - 2.6/2)/12 = 92$ ft·k > 71 ft·k $\therefore$ ok ✓

$A_{s,min} \geq \begin{cases} 0.45\ in^2 \\ 0.47\ in^2 \end{cases} << 1.32\ in^2 \therefore$ ok ✓

$\longrightarrow \boxed{\therefore \text{ use (4) \#6 bars}}$



· Check deflections $\rightarrow l/28 = (20)(12)/28 = 8.6" < 14" \therefore$ ok ✓

{ACI 318-11 §9.5}

<u>Summary of Girders</u>
12" x 14"
with (4) #6 top
(3) #6 btm

- Moments

$t_{slab} = 5"$ $\quad$ Total Load = 215 psf

$M_u^- = (215\ psf)(1')(9')^2/11 = 1.58\ k\ (\frac{ft}{ft})$

$M_u^+ = (215\ psf)(1')(9')^2/16 = 1.1\ k\ (\frac{ft}{ft})$

IV. One-Way Reinforced slab with Beams/Girders (cont.)

-Bottom

$d \sim 5" - 1.0" - 0.5" = 3.5"$

$A_s = \frac{1.1}{4(3.5)} = 0.079 \text{ in}^2$ (per foot)

$A_{s_{min}} \geq \begin{cases} \frac{3\sqrt{4000}}{60000}(12)(3.5) = 0.133 \text{ in}^2 \\ \frac{200}{60000}(12)(3.5) = 0.14 \text{ in}^2 \text{ (per foot)} \end{cases} \longrightarrow$ TRY #4 bars @ 12" o.c. $\longrightarrow A_s = 0.2 \text{ in}^2$ (per foot)

$d = 5" - 1.0" - (0.5"/2) = 3.75"$

$a = \frac{(0.2)(60000)}{0.85(4000)(12)} = 0.29"$ $\quad c = \frac{0.29}{0.85} = 0.35"$

$\varepsilon_s = \frac{0.003}{0.35}(3.75 - 0.35) = 0.03 > \varepsilon_y \therefore$ ok ✓

$\phi M_n = (0.2)(60)[3.75 - (0.29/2)]/12 = 3.6 \text{ k}\left(\frac{ft}{ft}\right) > 1.1 \therefore$ ok ✓

$\longrightarrow \boxed{\therefore \text{ use \#4 bars @ 12" o.c.}}$

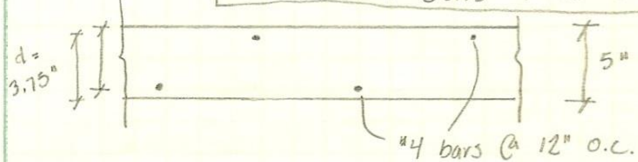
-Top

$A_s = \frac{1.58}{4(3.5)} = 0.113 \text{ in}^2$ (per foot)

$A_{s_{min}} \geq 0.14 \text{ in}^2 \longrightarrow$ TRY #4 bars @ 12" o.c.

$\phi M_n = 3.6 > 1.58 \therefore$ ok ✓

$\longrightarrow \boxed{\therefore \text{ use \#4 bars @ 12" o.c.}}$



-Shear $\longrightarrow$ for slab: $V_u = \frac{w_u l_n}{2} = \frac{(215 \text{ psf})(9')(1')}{2} = 0.97 \text{ k per 1 ft}$

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{4000}(12)(3.75) = 5.7 \text{ k per 1 ft}$

$0.75(5.7) = 4.3 \text{k per 1 ft} > 0.97 \therefore$ ok ✓

for beams: $V_u = (2.47)(29)/2 = 35.8^k$

$V_c = 2(1)\sqrt{4000}(12)(18.6) = 28.2$

$0.75(28.2) = 21^k < 35.8^k \therefore$ NOT OK, will need shear stirrups

for girders: $V_u = (2.16)(19)/2 = 20.5^k$

$V_c = 2(1)\sqrt{4000}(12)(11.75) = 17.8^k$

$0.75(17.8) = 13.3^k < 20.5^k \therefore$ NOT OK, will need shear stirrups

Marissa Delozier | AE Senior Thesis | Tech Report #3

Alternate System Comparison

Considerations		Composite Deck + Beam/Girder	Non-Composite Deck + Bar Joists/Girder	Pre-Cast Concrete on Structural Steel	One-Way Slab with Beams/Girders
		Systems			
General	Weight	52 psf	53.9 psf	44.5 psf	110 psf
General	Depth	27 1/8"	28 1/5"	43"	26"
General	Cost	Moderate	Moderate	High	Moderate
Architectural	Bay Size	30' x 40'	30' x 40'	30' x 40'	30' x 20'
Architectural	Floor-to-Floor Ht.	——	Decreased	Decreased	Increased
Architectural	Fire Rating	2 hr	2 hr	2 hr	2 hr
Structural	Lateral System Impact	——	None	None	Changed to shear walls
Structural	Foundation Impact	——	None	None	Will need increased
Construction	Schedule	——	Increased	Increased	Increased
Construction	Constructability	Moderate	Easy	Easy	Moderate
Serviceability	Max Deflection	0.85"	1.04"	0.98"	Permissable by ACI 318-11 Table 9.5(a)
Serviceability	Vibration Control	Fair	Fair	Moderate	Best
Possibility	Possible or Not Likely	Possible	Possible	Not Likely	Not Likely