Jonathan Ebersole
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September 25, 2013
Dr. Linda Hanagan
Associate Professor
212 Engineering Unit A
University Park, Pa 16802

Dear Dr. Hanagan,

This document is developed to help guide you through the evaluation of a typical bay under gravity loads and the evaluation of three alternative framing systems for the Oklahoma University Children's Medical Office Building. The purpose of this assignment is to evaluate a typical bay under gravity loads and to analyze three different structural systems that I could use in my proposal. The document contains a site plan of the building along with a list of codes and documents used to determine the member sizes. The calculations contain the loading, existing member analysis, and three different alternative systems. The calculations are accompanied by sketches of existing and proposed bays. For each of the four systems, I analyzed the flexural strength, the shear strength, and the deflections of each member. A column evaluation was conducted for the existing system. For my three alternative systems, I chose to use non-composite steel, composite steel, and a one way slab with beams.

Sincerely,

Jonathan Ebersole

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OU Children's Medical Office Building

Jonathan Ebersole | Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2014/jme5193/index.html

Project Team

- Owner: University Hospitals Trust
- Construction Manager: Flintco, Inc.
- Project Architect: Miles Associates
- Design Architect: Hellmuth, Obata, and Kassabaun, Inc.
- Structural Engineer: Zahl-Ford
- MEP Engineer: ZRHD, P.C.
- Civil Engineer: Smith, Roberts, Baldischwiler, Inc.

General Information

- Location: 1200 North Children's Avenue, Oklahoma City, Oklahoma
- Occupancy: Office
- Size: 320,000 sq. ft.
- Height: 12 Stories for a total of 172 ft.
- Construction Dates: February 2007-Spring of 2009
- Building Cost: $59,760,000
- Delivery Method: Design-Bid-Build

Architecture

- Exterior Façade comprised of brick Veneer with large glass curtain wall on the front face of the building
- Supports Hospital with additional office space, exam rooms, and labs
- Membrane roof system with rigid insulation and light weight insulating concrete

Structural Design

- Reinforced concrete columns and beams
- 10" thick flat slab system with drop panels
- Concrete shear walls located in elevator shafts and stairwells
- Drilled pier foundation with a minimum bearing capacity of 45 KSF

Mechanical Design

- 7,500 CFM Air Handling unit occupies each floor
- Heat Exchanger is used to heat water before entering the heating coil
- 850 CFM fans are used to pressurize the stairwells

Lighting/Electrical Design

- Service voltage is 480/277 V, three phase, with 4 wires
- Voltage reduced to 120/208V, three phase, with 4 wires and supplied to each panel box
- Fluorescent lamps are used throughout the building to save energy costs

General Information

Executive Summary

OU Children's Medical Office Building is an office building located in Oklahoma. It is situated next to an existing hospital and parking garage. The building houses offices, examination rooms and labs for the expanding OU Children's Hospital. It is the largest free standing clinical office in the state and provides much needed medical services to the children of Oklahoma and their families.

The structure of the building is reinforced concrete. The building uses a flat slab system supported by columns and exterior beams. Drop panels are used at the column locations to provide extra shear and moment capacity to the slab. The columns are supported on piers that transfer the loads to bedrock underneath the building. The building also uses shear walls and moment frames to resist the lateral forces.

This building provides several unique challenges that a typical office building would not otherwise have. These include a parking garage located on the first floor, a future helicopter pad positioned on the roof, and impact loads on lower levels for vehicle collisions with the building. These design parameters will increase the difficulty of future design assignments as all load cases must be analyzed.

Site Plan

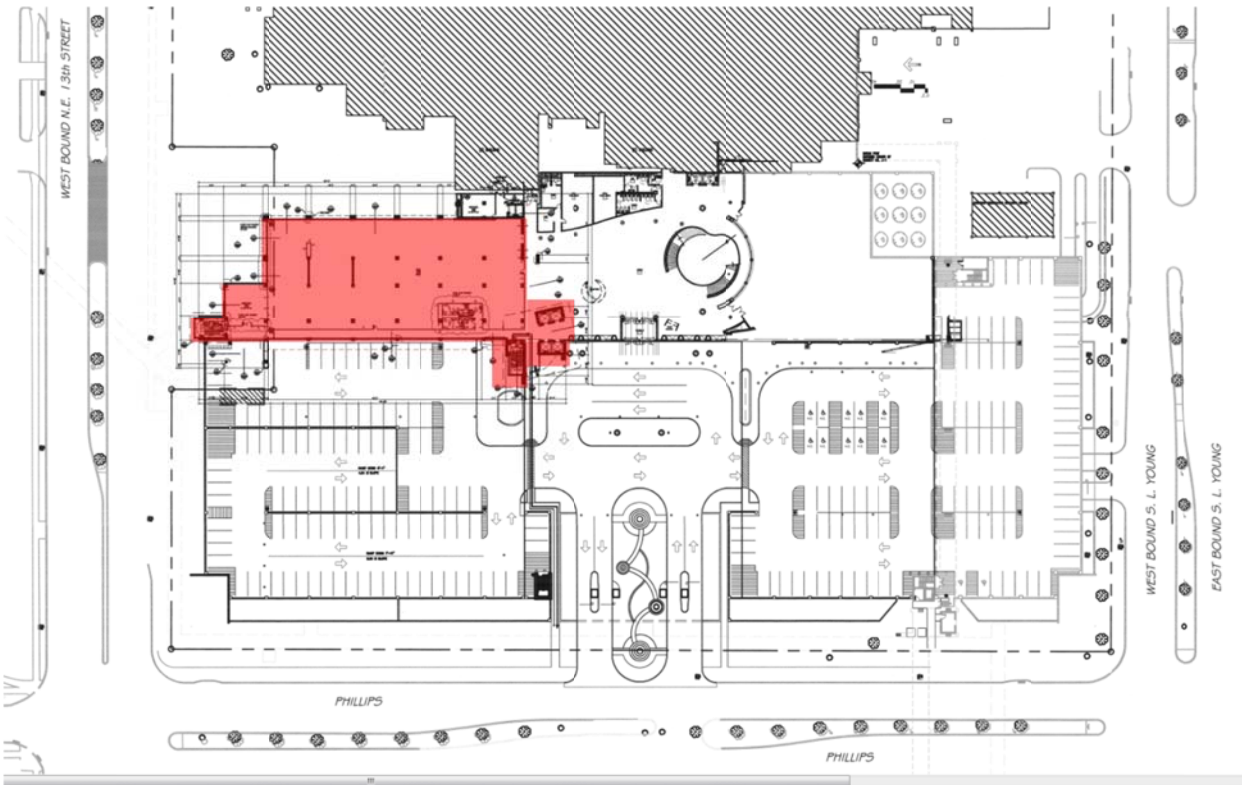


Figure A. Building outlined in red.

OU Children's Medical Office Building is located on 1200 N. Children's Avenue Oklahoma City, Oklahoma between Stanton L. Young Blvd and N.E. 13th Street. (Refer to figure A for site and building footprint). The building is twelve stories above grade and is approximately 180 feet tall. The building footprint is 22,820 square feet with a total area of 320,000 square feet. The building is positioned between an existing hospital and existing parking structure. A large atrium connects the hospital to the office building and parking structure but it is a future addition and not part of the original office construction. The building is located in an urbanized area which will later impact the design for the lateral loads.

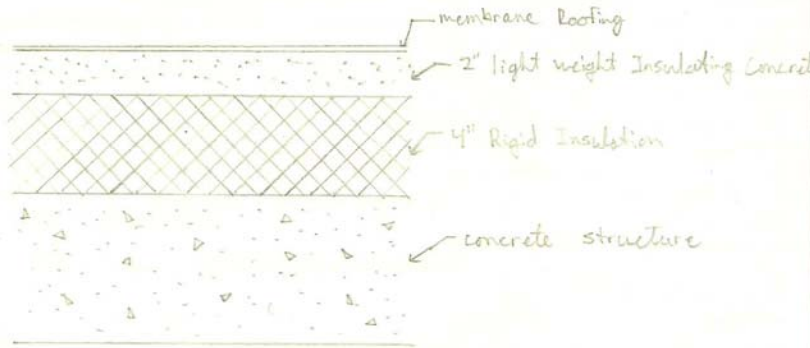
List of Documents

For this assignment, several documents where used in order to evaluate and design the required members. The ACI 318-02 code was used to analyze the existing structure, whereas, the ACI 318-11 was used to design the one way slab and beams in one of my alternative systems. I also used examples and design aids from the sixth edition of *Reinforced Concrete Mechanics and Design* written by James Wight and James MacGregor. To design the composite decking found in two of my alternative systems, I used the Vulcraft deck catalog. To determine the member sizes for these two systems, the fourteenth edition of the AISC Steel Construction Manual was used. Notes from AE 401, AE 402, AE 403 and AE 431 were also used.

2-1

1. Gravity Loads

1.a. Roof



cross section of Typical Roof Construction

Loads

Live Load - 20 psf (IBC 2003)

Dead Load

Materials

Membrane Roof	ASCE 7-02		~1 psf
Light Weight Insulating Concrete (2" thk)	ASCE 7-02	$115\text{pcf} \cdot \frac{2\text{in}}{12\text{in}}$	~19 psf
Rigid Insulation (4" thk)	AISC 14th ed.	$1.5\text{psf/in} \cdot 4\text{in}$	~6 psf
Superimposed Dead Load			~15 psf

Structure

slab (10" thk)	$150\text{pcf} \cdot \frac{10\text{in}}{12\text{in}}$	~125 psf
drop panels (4" thk)	$150\text{pcf} \cdot \frac{4\text{in}}{12\text{in}} \cdot \frac{10.67' \cdot 7.33'}{26' \cdot 32'}$	~5 psf
column (22"x22")	$150\text{pcf} \cdot \frac{(144\text{in} - 14\text{in})}{12\text{in}} \cdot \frac{22\text{in} \cdot 22\text{in}}{12^2 \cdot 26' \cdot 32'}$	~7 psf

2-2

Snow Loads

Flat Roof Snow Loads (ASCE 7-02)
slope = 1/4" per foot < 5° slope ⇒ fits criteria

$p_f = 0.7\, C_e\, C_t\, I\, p_g$

$C_e = 1.0$
$C_t = 1.1$
$I = 1.0$
$p_g = 10\text{ psf}$

$p_f = 0.7 \cdot 1.0 \cdot 1.1 \cdot 1.0 \cdot 10\text{ psf} = 7.7\text{ psf}$

but not less than:

p_g is 20 psf or less

$p_f = I(p_g) = 1.0 \cdot 10 = 10\text{ psf}$ → controls

Snow Drift Loads

Parapet

$l_u = 91.29\text{ ft}$
$h_d = 2.5\text{ ft}$
$h_c = 4.683\text{ ft}$
$\gamma = 0.13\, p_g + 14 = 0.13 \cdot 10 + 14 = 15.3\text{ pcf}$
$h_b = p_f / \gamma = 10 / 15.3 = 0.65\text{ ft}$
$w = 4 h_d = 4 \cdot 2.5 = 10\text{ ft}$

$h_c / h_b = 4.683 / 0.65 = 7.2 > 0.2 \therefore$ must be applied

$p_d = 3/4\, h_d\, \gamma = 3/4 \cdot 2.5 \cdot 15.3 = 28.69\text{ psf}$ (windward)

$p_d \cdot 0.5 \cdot w = 28.69 \cdot 0.5 \cdot 10 = 143.45\text{ plf}$

Load Combinations

$1.2D + 0.5L_r$ or S
$1.2D + 1.6L_r$ or S

Component

Slab

Live Load - 20 psf > S = 10 psf
Dead Load - 166 psf

2-3

$w_u = 1.2(166) + 1.6 \cdot (20) = \boxed{231.2 \text{ psf}}$

$w_u = 1.2(166) + 0.5(20) = 209.2$ psf

Column

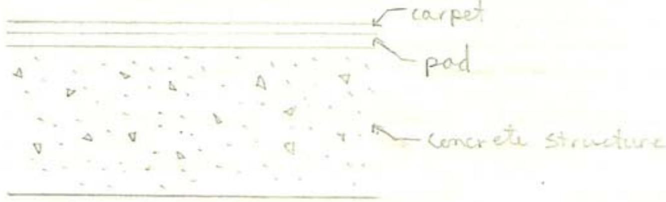
Live Load - 20 psf (unreducable
Dead Load - 178 psf

$w_u = 1.2(178) + 1.6(20) = \boxed{245.4 \text{ psf}}$

$w_u = 1.2(178) + 0.5(20) = 223.6$ psf

2-4

1. b. Floor



Cross Section of Typical Floor Construction

Loads

Live Loads
Office - 50 psf + 20 psf = 70 psf (IBC 2003)
Corridor - 80 psf (IBC 2003) → used to design for building flexibility

Dead Loads

Materials

Carpet with Pad (Boise Cascade - Weights of Materials) - 2 psf
Superimposed Dead Load - 15 psf

Structure

slab (10" thk) $150\,pcf \cdot \frac{10\,in}{12\,in}$ - 125 psf

drop panels (4" thk) $150\,pcf \cdot \frac{4\,in}{12\,in} \cdot \frac{10.67' \cdot 7.33'}{24' \cdot 32'}$ - 5 psf

column (30" x 30") $150\,pcf \cdot \frac{(168\,in - 14\,in)}{12\,in} \cdot \frac{30\,in \cdot 30\,in}{12\,in^2 \cdot 24' \cdot 32'}$ - 15 psf

Load Combinations

1.2D + 1.6L

Components

Slab

Live Load - 80 psf
Dead Load - 142 psf

2-5

$w_u = 1.2(142) + 1.6(80) = 298.4$ psf

column

Live Load - 80 psf

$$L = 80 \cdot \begin{cases} 0.4 \\ \max \; 0.25 + \dfrac{15}{\sqrt{\underbrace{4 \cdot 26 \cdot 32}_{\hookrightarrow 3328 > 400 \text{ ok } \checkmark}}} \end{cases} = 0.51 \cdot 80 = 40.8 \text{ psf}$$

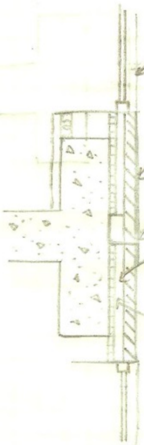
Dead Load - 162 psf

$w_u = 1.2 \cdot 162 + 1.6 \cdot 40.8 = 259.7$ psf

2-6

Exterior Wall

Brick Wall



Dead Load

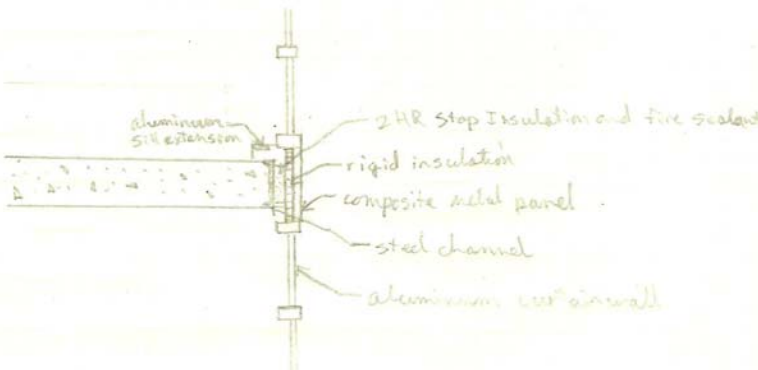
Materials
brick - 42 psf
rigid insulation (2" Thk.) - 3 psf

Total Dead Load - 45 psf · 14 ft = 630 plf (for floors 1-3)
- 45 psf · 14.67 ft = 660 plf (for floor 4)
- 45 psf · 13.115 ft = 590.2 plf (for floor 5)
- 45 psf · 12 ft = 540 plf for floor 6 - Roof)

Load transfer
- brick → steel angle → structure

2-7

aluminum and Glass wall



Materials
aluminum ~4 psf
rigid insulation (2" thk) ~3 psf
glass (2 pane @ 1/4" Thk ea.) ~8 psf

Total dead load ~15 psf · 14 ft = 210 plf (for floors 1-3)
~15 psf · 14.67 ft = 220 plf (for floor 4)
~15 psf · 13.115 ft = 197 plf (for floor 5)
~15 psf · 12 ft = 180 plf (for floors 6-roof)

Load Transfer
- composite metal panel → steel channel → structure

Non-typical Loads

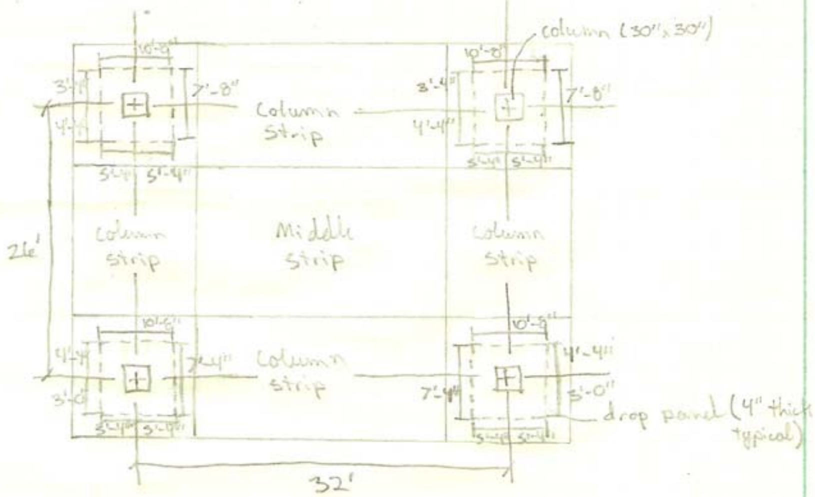
- vehicle impact load - 6 kips at 18" above finish floor
  - 6 kips is approximately the weight of a large vehicle

- helicopter pad load - 60 psf on roof between grid 6 and 7 and G and R.2
  - weight includes the weight of the helicopter and the weight of the pad.

- ambulance load - 480 plf of load lane - designed as HS15 from AASHTO - located at the first floor at joists

2-8

Typical Bay



Determine Minimum thickness of slab (with drop panels)

$\alpha_f = 0$ (The interior panel has no perimeter beams)

$\ell_n = 32 - \frac{30}{12} = 29.5\ ft$

From Table 9.5(c) ACI 318-02

Min $h = \frac{\ell_n}{36} = \frac{29.5}{36} = 0.819\ ft = 9.83\ in \approx 10\ in.$

Must not be less than 4 in, according to Section 9.5.3.2

2-9

Determin Shear Strength of Slab at Column Support

$f'c = 5000$ psi
$f_y = 60{,}000$ psi
Superimposed Dead Load. (Includes self weight of slab) - 298.4 psf

10'-8" x 7'-4" drop panel

Determine the Minimum Size of The Drop Panel based on ACI 318-02 Section <u>13.2.5</u>.

Minimum thickness : $\frac{10\,in}{4} = 2.5\,in < 4\,in$

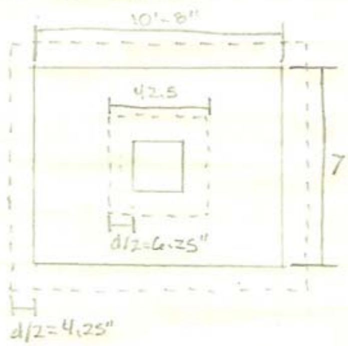
Minimum $l/6$: $\frac{32}{6}$ = 5'-4" = 5'-4"

Minimum $l/6$: $\frac{26}{6}$ = 4'-4" = 4'-4"

Minimum $l/6$: $\frac{20}{6}$ = 3'-4" = 3'-4"

Minimum $l/6$: $\frac{13.75}{6}$ = 2'-3½" < 3'-0"

Critical Section



Investigate shear strength at critical section bo located d/2 from column perimeter

$q_{Du} = 1.2 \cdot \frac{4}{12} \cdot 150 = 60$ psf

$V_u = 0.2984\,ksf\,(32 \cdot 26 - 3.54^2) + 0.06\,(10.67 \cdot 7.33 - 3.54^2)$
$= 248.47$ kips

2-10

$b_o = 4 \cdot 42.5 = 170$ in

$2+\frac{4}{\beta} = 2+\frac{4}{1.23} = 5.25$ $\qquad \beta = \frac{32}{26} = 1.23$

$\frac{a_s \cdot d}{b_o} + 2 = \frac{40 \cdot 12.5}{170} + 2 = 4.94$

min 4

$\phi V_c = \phi \cdot 4 \cdot \lambda \sqrt{f'_c}\, b_o\, d = 0.75 \cdot 4 \cdot 1.0 \cdot \sqrt{5000} \cdot 170 \cdot 12.5 = 450.8$ kips

$\phi V_c > V_u$
450.8 kips > 248.47 kips ok

Investigate shear strength at critical section b_o located at d/2 from the edge of the drop panel

$V_u = 0.2984\,(32 \cdot 26 - 11.375 \cdot 8.04) = 220.98$ kips

$b_o = 2 \cdot (128+8.5) + 2 \cdot (88+8.5) = 466$ in

$2+\frac{4}{\beta} = 2+\frac{4}{1.23} = 5.25$

$\frac{a_s \cdot d}{b_o} + 2 = \frac{40 \cdot 8.5}{466} + 2 = 2.73$

min 4

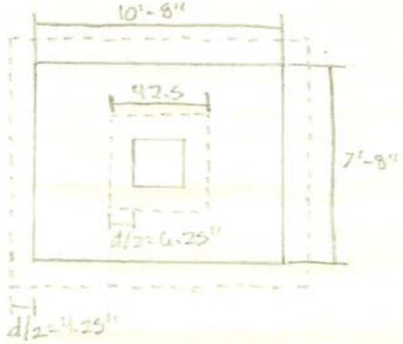
$\phi V_c = \phi \cdot \left(\frac{a_s \cdot d}{b_o} + 2\right) \lambda \sqrt{f'_c}\, b_o\, d = 0.75 \cdot 2.73 \cdot 1.0 \cdot \sqrt{5000} \cdot 466 \cdot 8.5$
$= 573.5$ kips

$\phi V_c > V_u$
573.5 kips > 220.98 kips Ok

2-11

10'-8" x 7'-8" drop panel

Critical Section



Investigate shear strength at critical section bo located d/2 from column perimeter

$q_{Du} = 1.2 \cdot \frac{4}{12} \cdot 150 = 60\ psf$

$V_u = 0.2984(32 \cdot 26 - 3.54^2) + 0.06(10.67 \cdot 7.67 - 3.54^2)$
$= 248.7$ kips

$b_o = 4 \cdot 42.5 = 170$ in

$2 + \frac{4}{\beta} = 2 + \frac{4}{\beta} = 5.25$ $\qquad \beta = \frac{32}{26} = 1.23$

$\frac{\alpha_s \cdot d}{b_o} + 2 = \frac{40 \cdot 12.5}{170} + 2 = 4.94$

min 4

$\phi V_c = \phi \cdot 4 \cdot \lambda \cdot \sqrt{f'c} \cdot b_o \cdot d = 0.75 \cdot 4 \cdot 1.0 \cdot \sqrt{5000} \cdot 170 \cdot 12.5 =$
$= 450.8$ kips

$\phi V_c > V_u$
450.8kips > 248.7 kips OK

2-12

Investigate shear strength at critical section bo located at d/2 from the edge of the drop panel.

$V_u = 0.2984(32 \cdot 26 - 11.375 \cdot 8.375) = 219.8$ kips

$b_o = 2 \cdot (128 + 8.5) + 2 \cdot (92 + 8.5) = 474$

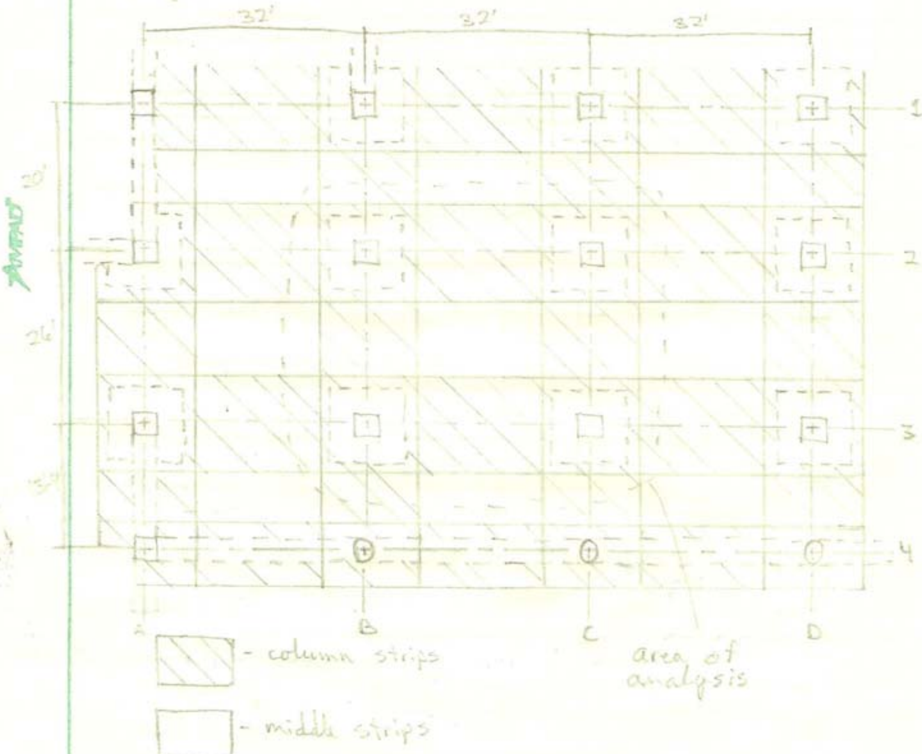
$$\min \begin{cases} 2 + \frac{4}{\beta} = 2 + \frac{4}{1.23} = 5.25 \\ \frac{\alpha_s \cdot d}{b_o} + 2 = \frac{40 \cdot 8.5}{474} + 2 = 2.72 \\ 4 \end{cases}$$

$\phi V_c = \phi\left(\frac{\alpha_s \cdot d}{b_o} + 2\right) \lambda \sqrt{f'_c}\, b_o d = 0.75 \cdot 2.72 \cdot 1 \cdot \sqrt{5000} \cdot 474 \cdot 8.5$
$= 581.2$ kips

$\phi V_c > V_u$
581.2 kips > 219.8 kips Ok

2-13

Determine the positive and negative moments for the column strips and Middle strips using the Direct Design Method.



Check if Direct Design Method can be used.

- Minimum of 3 spans in each direction ✓

- Ratio of $L_\ell / L_s \leq 2$

$\frac{32}{20} = 1.6$ ✓ $\quad \frac{32}{26} = 1.23$ ✓ $\quad \frac{32}{13.75} = 2.33$ X

- Shall not differ by $\frac{1}{3} L_\ell$

$32' = 32' = 32'$ ✓ $\quad \frac{1}{3}\,26 = 8.67 \quad 26 - 8.67 = 17.33\text{ ft} < 20$ ✓

$17.33\text{ ft} > 13.75$ X

Note: Based on Section 13.6.1.8, allowed to use Direct Design Method as long as Section 13.5.1 is satisfied.

2-14

Determine moments along gridline B between grids 2 and 3 (Same as moments along gridline C)

Compute M_o:

$$M_o = \frac{q_u l_2 l_n^2}{8}$$

$q_u = 298.4\,psf$
$l_2 = 32\,ft$
$l_n = 26' - \frac{30}{12}$

$$M_o = \frac{298.4 \cdot 32 \cdot 23.5^2}{8} = 659.2\ ft \cdot k$$

Divide M_o into negative and positive moments:

Negative moment $= -0.65 M_o = -428.48\ ft \cdot K$

Positive moment $= 0.35 M_o = 230.72\ ft \cdot k$

Divide moments between the column and middle strips

Negative moments

Column-strip negative moment =

$l_2/l_1 = 32/26 = 1.23$
$\alpha f_1 = 0$ (no beams)

$0.75 \cdot -428.48\ ft \cdot k = -321.36\ ft \cdot k$

Middle-strip negative moment -

$0.25 \cdot -428.48 = -107.12\ ft \cdot k$

Positive Moments

Column-strip positive moment -

$l_2/l_1 = 1.23$
$\alpha f_1 = 0$ (no beams)

$0.6 \cdot 230.72\ ft \cdot k = 138.4\ ft \cdot k$

Middle-strip positive moment -

$0.4 \cdot 230.72\ ft \cdot k = 92.29\ ft \cdot k$

2-15

Determine moments along gridline 2 between grids B and C

Compute M_o:

$$M_o = \frac{q_u \, l_2 \, l_n^2}{8}$$

$q_u = 298.4$ psf

$l_2 = \frac{20+26}{2} = 23$ ft

$l_n = 32 - \frac{30}{12} = 29.5$ ft

$$M_o = \frac{298.4 \cdot 23 \cdot 29.5^2}{8} = 746.6 \text{ ft·k}$$

Divide M_o into negative and positive moments:

Negative Moment $= -0.65\, M_o = -485.3$ ft·k

Positive Moment $= 0.35\, M_o = 261.3$ ft·k

Divide Moments between column strips and middle strips

Negative moments

Column-strip negative moment-

$l_2/l_1 = 23/32 = 0.72$

$\alpha_{f1} = 0$ (no beams)

$0.75 \cdot -485.3$ ft·k $= -363.98$ ft·k

Middle-strip negative moment-

$0.25 \cdot -485.3$ ft·k $= -121.33$ ft·k

Positive Moments

Column-strip positive moment-

$l_2/l_1 = 0.72$

$\alpha_{f1} = 0$ (no beams)

$0.6 \cdot 261.3$ ft·k $= 156.78$ ft·k

Middle-strip positive moment-

$0.4 \cdot 261.3$ ft·k $= 104.52$ ft·k

2-16

Determine moments along grid line 3 between grids B and C

Compute M_o =

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

$q_u = 298.4$ psf

$\ell_2 = \frac{26 + 13.75}{2} = 19.88$ ft

$\ell_n = 32 - \frac{30}{12} = 29.5$ ft

$$M_o = \frac{298.4 \cdot 19.88 \cdot 29.5^2}{8} = 645.31 \text{ ft} \cdot \text{k}$$

Divide M_o into negative and positive moments:

Negative Moment = $-0.65\, M_o = -419.45$ ft·k

Positive Moment = $0.35\, M_o = 225.86$ ft·k

Divide Moments between column strips and middle strips

Negative moments

Column-strip negative moment-

$\ell_2 / \ell_1 = 19.88 / 32 = 0.62$

$\alpha_{f1} = 0$ (no beams)

$0.75 \cdot -419.45$ ft·k $= -314.59$ ft·k

Middle-strip negative moment -

$0.25 \cdot -419.45$ ft·k $= -104.86$ ft·k

Positive Moments

Column-strip positive Moment

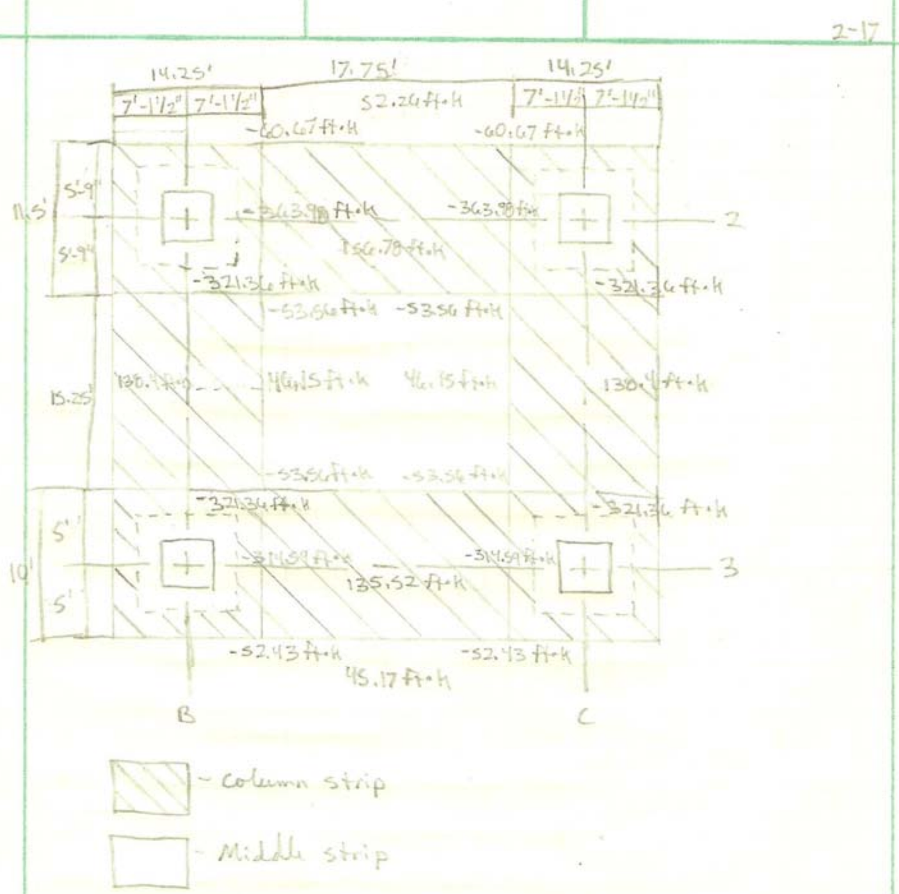
$\ell_2 / \ell_1 = 0.62$

$\alpha_{f1} = 0$ (no beams)

$0.6 \cdot 225.86$ ft·k $= 135.52$ ft·k

Middle-strip positive Moment

$0.4 \cdot 225.86$ ft·k $= 90.34$ ft·k



2-17
14.25'
17.75'
14.25'
7'-1½"
7'-1½"
52.24 ft·k
7'-1½"
7'-1½"
-60.67 ft·k
-60.67 ft·k
11.5'
5'-9"
5'-9"
-363.98 ft·k
-363.98 ft·k
156.78 ft·k
2
-321.36 ft·k
-321.36 ft·k
-53.56 ft·k
-53.56 ft·k
15.25'
130.4 ft·k
46.15 ft·k
46.15 ft·k
130.4 ft·k
-53.56 ft·k
-53.56 ft·k
-321.36 ft·k
-321.36 ft·k
10'
5'
5'
-314.59 ft·k
-314.59 ft·k
135.52 ft·k
3
-52.43 ft·k
-52.43 ft·k
45.17 ft·k
B
C
- Column strip
- Middle strip

2-18

Compute the area of Steel along grid 2

$A_s = \frac{M_u}{\phi f_y jd}$ (Eq. 13-44 from Wright and MacGregor)

$d = h - 0.75 - 0.75/2 = 8.875 \text{ in}$

assume $j = 0.95$

$A_s(\text{req'd}) = \frac{363.98 \text{ ft}\cdot\text{k} \cdot 12{,}000}{0.9 \cdot 60{,}000 \cdot 0.95 \cdot 8.875} = 9.59 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{9.59 \cdot 60{,}000}{0.85 \cdot 5000 \cdot 11.5 \cdot 12} = 0.98 \text{ in}$

$c = \frac{a}{\beta_1} = \frac{0.98}{0.80} = 1.23 \text{ in}$

$a = 0.98$

$\beta_1 = 0.85 - \frac{0.05}{1000}(f'_c - 4000) \geq 0.65$

$0.85 - \frac{0.05}{1000}(5000 - 4000) = 0.8 \geq 0.65$

$1.23 < \frac{3d}{8} = 3.33 \text{ in} \therefore \phi = 0.9$

$A_s = \frac{M_u \cdot 12{,}000}{0.9 \cdot 60{,}000 \cdot (8.875 - \frac{0.98}{2})}$ (assuming a is constant for all sections)

$A_s = 0.0265 M_u$

$A_{s,min} = 0.0018 bh$

$b = 11.5 \text{ ft}$
$h = 10 \text{ in}$

$A_{s,min} = 11.5 \cdot 12 \cdot 0.0018 \cdot 10 = 2.48 \text{ in}^2$

Compute the area of steel along grid 3

$A_s = \frac{M_u}{\phi f_y jd}$

$d = h - 0.75 - 0.75/2 = 8.875 \text{ in}$

assume $j = 0.95$

$A_s(\text{req'd}) = \frac{314.59 \text{ ft}\cdot\text{k} \cdot 12{,}000}{0.9 \cdot 60{,}000 \cdot 0.95 \cdot 8.875} = 8.29 \text{ in}^2$

2-19

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{8.29 \cdot 60{,}000}{0.85 \cdot 5000 \cdot 10 \cdot 12} = 0.98 \text{ in}$

$c = \frac{a}{\beta_1} = \frac{0.98}{0.80} = 1.23 \text{ in}$

$a = 0.96$

$\beta_1 = 0.80$

$1.23 < \frac{3d}{8} = 3.33 \therefore \phi = 0.9$

$A_s = \frac{M_u \cdot 12{,}000}{0.9 \cdot 60{,}000 \cdot (8.875 - \frac{0.98}{2})}$ (assuming a is constant for all sections)

$A_s = 0.0265\, M_u$

$A_{s,min} = 0.0018\, bh$

$b = 10 \text{ ft}$

$h = 10 \text{ in}$

$A_{s,min} = 0.0018 \cdot 10 \cdot 10 \cdot 12 = 2.16 \text{ in}^2$

Computer the area of steel along grid B and C

$A_s = \frac{M_u}{\phi f_y j d}$

$d = h - 0.75 - 0.75 - 0.75/2 = 8.125 \text{ in}$

assum $j = 0.95$

$A_s (\text{req'd}) = \frac{321.36 \text{ ft} \cdot k \cdot 12{,}000}{0.9 \cdot 60{,}000 \cdot 0.95 \cdot 8.125} = 9.25 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 \cdot f'_c \cdot b} = \frac{9.25 \cdot 60{,}000}{0.85 \cdot 5000 \cdot 14.25 \cdot 12} = 0.76 \text{ in}$

$c = \frac{a}{\beta_1} = \frac{0.76}{0.80} = 0.95$

$a = 0.76$

$\beta_1 = 0.80$

$0.95 < \frac{3d}{8} = 3.05 \therefore \phi = 0.9$

$A_s = \frac{M_u \cdot 12{,}000}{0.9 \cdot 60{,}000 \cdot (8.125 - \frac{0.76}{2})}$

$A_s = 0.0287\, M_u$

$A_{s,min} = 0.0018\, bh$

$b = 14.25$

$h = 10 \text{ in}$

$A_{s,min} = 0.0018 \cdot 14.25 \cdot 12 \cdot 10 = 3.08 \text{ in}^2$

2-20

Grid B and C Interior Negative Moment	Column Strip	Middle Strip
Total strip moment (kip-ft)	-321.36	-107.12
Required As (in²)	9.22	3.07
Minimum As (in²)	3.08	3.83
Selected Steel	#6@6in oc	#6@12in oc
As provided (in²)	12.32	7.48

Grid B and C Interior Positive Moment	Column Strip	Middle Strip
Total strip Moment (kip-ft)	139.4	92.3
Required As (in²)	3.97	2.65
Minimum As (in²)	3.08	3.83
Selected Steel	#6@7in oc	#6@12in oc
As provided (in²)	10.56	7.48

Grid 2 Interior Negative Moment	Column Strip	Middle Strip
Total Strip Moment (kip-ft)	-363.96	-121.33
Required As (in²)	9.65	3.22
Minimum As (in²)	2.48	3.29
Selected Steel	#6@6in oc	#6@12in oc
As provided (in²)	10.12	6.6

Grid 2 Interior Positive Moment	Column Strip	Middle Strip
Total Strip Moment (kip-ft)	156.78	104.52
Required As (in²)	4.15	2.77
Minimum As (in²)	2.48	3.29
Selected Steel	#6@7in oc	#6@12in oc
As provided (in²)	10.12	6.6

Grid 3 Interior Negative Moment	Column Strip	Middle Strip
Total Strip Moment (kip-ft)	-314.59	-104.86
Required As (in²)	8.34	2.78
Minimum As (in²)	2.16	3.29
Selected Steel	#6@6in oc	#6@12in oc
As provided (in²)	8.8	6.6

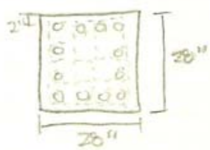
2-21

Grid 3 Interior Positive Moment	Column Strip	Middle Strip
Total Strip Moment (kip-ft)	135.52	90.34
Required As (in^2)	3.59	2.39
Minimum As (in^2)	2.16	3.79
Selected steel	#6@7in oc	#6@12in oc
As provided (in^2)	8.8	6.6

2-22

Interior Column

$f'c = 4000$ psi
$fy = 60{,}000$ psi
$A_{st} = 9.48$



$Pu = [1.2(178) + 0.5(20) + 1.2(162) \cdot 12 + 1.6(40.8) \cdot 12] \cdot 736$

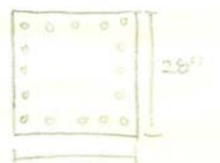
$= 2458$ k

$P_o = 0.85 f'c (A_g - A_{st}) + f_y A_{st}$
$= 0.85 \cdot 4000 \cdot (784 - 9.48) + 60{,}000 \cdot 9.48$
$= 4518$

$\phi P_o = 4518 \cdot 0.65 = 2936.7k > 2458$ k ✓

Exterior Column

$f'c = 4000$ psi
$fy = 60{,}000$ psi
$A_{st} = 20.32$
$d' = 3.01"$



$L = 80\left(0.25 + \frac{15}{\sqrt{32 \cdot 2 \cdot 29.29}}\right) = 80 \cdot 0.596 = 47.68$ psf

includes edge beam

$Pu = [1.2(178) + 0.5(20) + 1.2(206) \cdot 12 + 1.6(47.68) \cdot 12] \cdot 468.64$
$+ 1.2(6920.2)$
$= 1932.3$ k

$Mu = \frac{w_u l^2}{12} = \frac{298.4 \cdot 29.29}{12} = 728.3$ k·ft

$\gamma = \frac{h - 2d'}{h} = \frac{28 - 2(3.01)}{28} = 0.785$

$\rho g = \frac{16 \cdot 1.27}{28 \times 28} = 0.026 = 2.6\%$

Using Figure A-11a and A-11b from Wright and MacGregor

For $\gamma = 0.75$ $\phi P_n = 3.35$ b·h

For $\gamma = 0.9$ $\phi P_n = 3.35$ b·h

$\phi P_o = 3.35 \cdot \frac{bh}{\alpha} = 3.35 \cdot \frac{28 \cdot 28}{0.8} = 3283$ k > 1932.3 k ✓

2-23

for $\gamma = 0.75$ $\phi M_n = 0.54 \cdot b \cdot h^2$

for $\gamma = 0.90$ $\phi M_n = 0.62 \cdot b \cdot h^2$

$\phi M_n = 0.58 \cdot 28 \cdot 28^2 = 1061\ k \cdot ft > 728.3\ k \cdot ft$ ✓

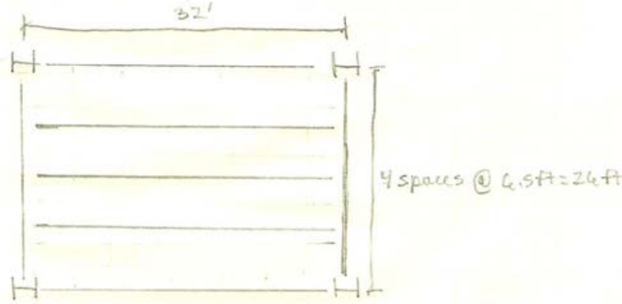
2-24

Alternate System 1: Non-composite steel

Loading

Live Load
office = 50 + 20 = 70 psf
corridor = 80 psf → used for building flexibility

Dead load
carpet with pad - 2 psf
Superimposed Dead Load - 15 psf



Determine Deck size
Use composite decking - more commonly used with rolled beams and girders.

Use light weight concrete for lighter construction

Use a topping of 3 1/4" for 2 hour fire rating

Use unshored construction for more economical design

Try 1.5 VLR 22 guage

3 span unshored construction
- 7'7" > 6'6"

Superimposed Load - 97 psf

Superimposed Live Load for 6'-6" span - 260 psf

260 > 97

Use Vulcraft 1.5 VLR 22 guage deck with 3 1/4" LW topping

2-25

Determine Beam Size

Load

Live Load - 80 psf (reduceable)

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \geq 0.5 L_o$$

$$= 80 \cdot \left(0.25 + \frac{15}{\sqrt{\underbrace{13 \cdot 32}_{416 > 400\ \checkmark}}}\right) = 80 \cdot 0.985 = 78.8 \text{ psf}$$

Dead Load

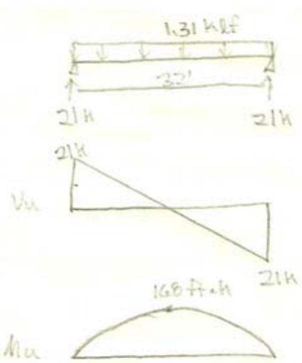
Deck - 41 psf
carpet with pad - 2 psf
Superimposed dead load - 15 psf
beam self weight allowance - 5 psf

Total - 63 psf

$w_u = 1.2(63) + 1.6(78.8) = 201.68$ psf

$w_u = \frac{201.68 \cdot 6.5}{1000} = 1.31$ klf

Shear and Moment Diagrams



2-26

Check Bending

Using Z_x tables in AISC 14th ed.

$-M_u \leq \phi_b M_{px}$

W14x30 $- \phi_b M_{px} = 177$ ft·k > 168 ft·k
W16x31 $- \phi_b M_{px} = 203$ ft·k > 168 ft·k

Check Shear

Using Z_x tables:

$-V_u \leq \phi_v V_{nx}$

W14x30 $- \phi_v V_{nx} = 112$ kips > 21 kips
W16x31 $- \phi_v V_{nx} = 131$ kips > 21 kips

Check Deflection

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

W14x30

$$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x} = \frac{5(0.517)(32)^4(1728)}{384 \cdot 29000 \cdot 291} = 1.43 \text{ in}$$

$$L/360 = \frac{32 \cdot 12}{360} = 1.067 < 1.43 \text{ in} \quad \text{No Good.}$$

$$I_{requ.} = 291 \cdot \frac{1.43}{1.067} = 390 \text{ in}^4$$

Using I_x tables

W18x35 $- I_x = 510$ in^4 > 390 in^4

Check Beam Weight assumption

weight of beam $- \frac{35}{6.5} = 5.38$ psf > 5 psf assumed

Recheck Bending with new loads

Loading

Live Load

- 78.8 psf (reduced)

2-27

Dead Load

Deck - 41psf
carpet with pad - 2psf
Superimposed dead Load - 15 psf
beam self weight allowance - 6 psf

Total = 64psf

$w_u = 1.2(64) + 1.6(78.8) = 202.88$ psf

$w_u = \frac{202.88 \cdot 6.5}{1000} = 1.32$ klf

Shear and Moment

$V_u = 21.12$k

$M_u = 169$ ft$\cdot$k

Check Bending

Using Z_x tables

W18x35 - $\phi_b M_{px} = 249$ k$\cdot$ft > 169 k$\cdot$ft ✓

Check Shear

Using Z_x tables

W18x35 - $\phi_V V_{nx} = 159$ k > 21.12k ✓

$\boxed{\text{Use W18x35 beams}}$

2-28

Determine Girder Size

Load

Live Load - 80 psf (reducable)

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \geq 0.5L$

$= 80\cdot\left(0.25 + \frac{15}{\sqrt{26\cdot 32\cdot 2}}\right) = 80\cdot 0.618 = 49.4$ psf

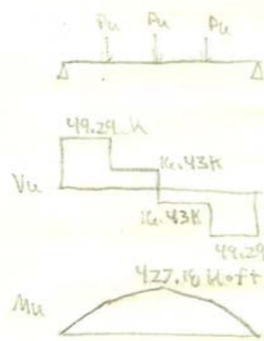
Dead Load

Deck - 41 psf
carpet with pad - 2 psf
superimposed dead load - 15 psf
beam self weight allowance - 6 psf
Girder self weight allowance - 2 psf

$W_u = 1.2(66\text{ psf}) + 1.6(49.4\text{ psf}) = 158$ psf

$P_u = 158(6.5)(32) = 32.86$ k

Shear and Moment Diagrams



Check Bending

Using Z_x tables
- $M_u \leq \phi_b M_{px}$

W21x55 - $\phi_b M_{px} = 473$ k·ft ≥ 427.18 k·ft

2-29

Check Shear

Using Z_x tables-
- $V_u \le \phi_v V_{ux}$

W 18x55 - $\phi_v V_{ux} = 234\ k > 49.29\ k$

Check Deflections

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

$\Delta_{LL} = \frac{P_{LL} L^3}{28 E I_x} = \frac{10.28 \cdot 26^3 \cdot 1728}{28 \cdot 29000 \cdot 1140} = 0.337\ in$

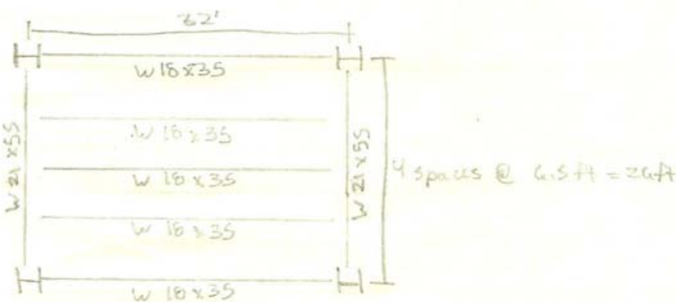
$L/360 = \frac{26 \cdot 12}{360} = 0.867\ in > 0.337\ in$

Check Girder weight assumption

weight of girder = 5.38 psf for beams + $\frac{55}{32}$ = 7.1 psf < 8 psf

OK

$\boxed{\text{Use W 21x55 for girders}}$



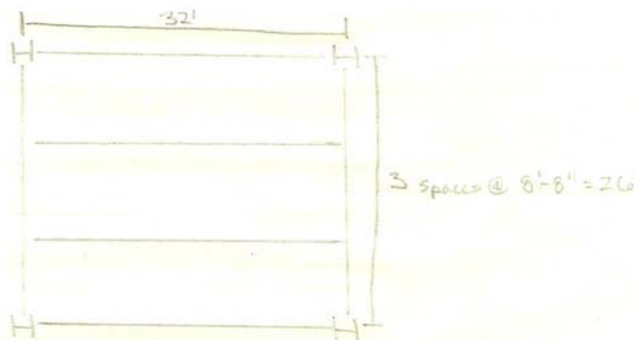
2-30

alternate system 2: composite steel

Loading

Live Load
office = 50+20 = 70 psf
corridor = 80 psf → used for building flexibility

Dead Load
carpet with pad - 2 psf
superimposed dead load - 15 psf



Determine Deck size

Use composite decking - more commonly used with rolled beams and girders

Use normal weight concrete

Use a topping of 4 1/2" for 2 hour fire rating

Use unshored construction for more economical design

Try 2 VLI 22 guage

3 span unshored construction
- 7'-11" < 8'-8" → use higher guage

Try 2VLI 20 guage

3 span unshored construction
- 9'-0" > 8'-8"

2-31

Superimposed Load - 97 psf

Superimposed Live Load for 9'-0" span - 213 psf

213 psf > 97 psf

$\boxed{\text{Use Vulcraft 2VLI 20 gauge deck with 4 1/2" NW topping}}$

Determine Beam Size

Load

Live Load - 80 psf (reducible)

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \geq 0.5\,L_o$$

$$= 80\left(0.25 + \frac{15}{\sqrt{\underbrace{17.333 \cdot 32}_{555 > 400\ \checkmark}}}\right) = 80 \cdot 0.887 = 70.96\ \text{psf}$$

Dead Load

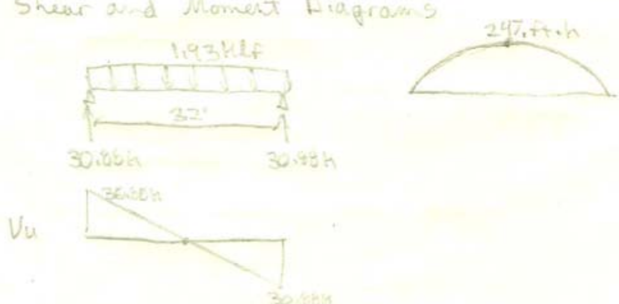
Deck - 69 psf
carpet with pad - 2 psf
Superimposed dead load - 15 psf
beam self weight allowance - 5 psf

total - 91 psf

$$w_u = 1.2(91) + 1.6(70.96) = 222.7\ \text{psf}$$

$$w_u = \frac{222.7 \cdot 8.67}{1000} = 1.93\ \text{klf}$$

Shear and Moment Diagrams



2-32

Determine ϕM_n and ΣQ_n

Use Table 3-19 from AISC 14th ed.

- assume $f'_c = 4000$ psi
- assume $a = 1$
- assume deck is perpendicular
- assume 1 weak stud per rib
- assume 3/4 stud
- assume 1 stud per foot

If $a = 1$; $y_2 = 6.50 - \frac{1}{2} = 6''$

W 14x26 $\Sigma Q_n = 135 = \frac{135}{17.2} = 7.85 \Rightarrow 8 \times 2 = 16$ studs/beam
$\phi M_n = 255$ ft·k $\quad = 992\#$

$\boxed{17.2}$ → from table 3-21 in AISC 14th ed.

Most economical by inspection → W 16x26 $\Sigma Q_n = 96 = \frac{96}{17.2} = 5.58 \Rightarrow 6 \times 2 = 12$ studs/beam
$\phi M_n = 252 \quad = 952\#$

W 12x26 $\Sigma Q_n = 198 = \frac{198}{17.2} = 11.5 \Rightarrow 12 \times 2 = 24$ studs/beam
$\phi M_n = 269 \quad = 1072\#$

W 12x22 $\Sigma Q_n = 230 = \frac{230}{17.2} = 13.3 \Rightarrow 14 \times 2 = 28$ studs/beam
$\phi M_n = 265 \quad = 984\#$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}}$$

$$b_{eff} = \min\begin{cases} \frac{32 \cdot 12}{8} = 48 \\ \frac{8.67 \cdot 12}{2} = 52.02 \end{cases} + \min\begin{cases} \frac{32 \cdot 12}{8} = 48 \\ \frac{8.67 \cdot 12}{2} = 52.02 \end{cases}$$

$b_{eff} = 48 + 48 = 96$ in

$a = \frac{96}{0.85 \cdot 4 \cdot 96} = 0.294 \therefore y$ actual $= 6.50 - \frac{0.294}{2} = 6.35'' \Rightarrow 6.5''$

$\therefore$ assumption incorrect

Use $y_2 = 6.5''$
$\Sigma Q_n = 96 \Rightarrow 12$ studs/beam
$\phi M_n = 255$ ft·k

Check unshored strength

W16x26, $\phi M_n = 262$ ft·k

$w_u = 1.4(69)(8.67) + 1.4(26) = 0.874$ klf

$w_u = 1.2[(69)(8.67) + 26] + 1.6(20)(8.67) = 1.03$ klf → controls

$M_u = \frac{1.03 \cdot 32^2}{8} = 131.8$ ft·k < 262 ft·k → ok for unshored construction

2-33

Check wet concrete deflection

$w_{wc} = 69(8.67) + 26 = 0.624\ \text{klf}$

$$\Delta_{wc} = \frac{5(0.624)(32)^4(1728)}{384(29000)(199)} = 1.69\ \text{in}$$

$$\Delta_{wc\,max} = \frac{32 \cdot 12}{240} = 1.6\ \text{in} < 1.69 \therefore \text{need to chamber by } 1.5''$$

$0.8(1.69) = 1.5''$

Check Live Load Deflection

$w_{LL} = 70.96 \cdot 8.67 = 0.615\ \text{klf}$

I_{LB} @ $y = 6.5$ and $\Sigma Q_n = 96$

$I_{LB} = 617$

$$\Delta_{LL} = \frac{5(0.615)(32)^4(1728)}{384(29000)(617)} = 0.811\ \text{in}$$

$$\Delta_{LL\,max} = \frac{L}{360} = \frac{32 \cdot 12}{360} = 1.07\ \text{in} > 0.811\ \text{in} \therefore \text{ok}$$

$\boxed{\text{Use W14x26 with 12 studs per beam for beams}}$

Determine Girder Size

Load

Live Load - 80 psf (reducible)

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \geq 0.5\,L_o$$

$$= 80\left(0.25 + \frac{15}{\sqrt{26 \cdot 32 \cdot 2}}\right) = 80 \cdot 0.618 = 49.44\ \text{psf}$$

$\rightarrow 1664 > 400$

Dead Load

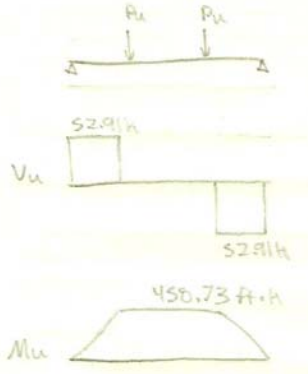
Deck - 69 psf
carpet with pad - 2 psf
Superimposed dead load - 15 psf
Beam self weight allowance - 5 psf
Girder self weight allowance - 2 psf

2-34

$w_u = 1.2(93) + 1.6(49.44) = 190.7$ psf

$P_u = 190.7(8.67)(32) = 52.91$ k

Shear and Moment Diagrams



Determine ϕM_n and ΣQ_n

Use table 3-19

- assume $F'_c = 4000$ psi
- assume $y = 6.5"$
- assume deck is parallel
- assume 1 weak stud per rib
- assume 3/4" stud
- assume 1 stud per foot

$y = 6.5"$

W18x46 $\Sigma Q_n = 169 = \frac{169}{21.5} = 7.86 \Rightarrow 8 \times 2 = 16$ studs per girder
$\phi M_n = 507$ k·ft $= 1356$ #
$\hookrightarrow w_r/h_r = 5/2 = 2.5$

W18x40 $\Sigma Q_n = 211 = \frac{211}{21.5} = 9.81 \Rightarrow 10 \times 2 = 20$ studs per girder
$\phi M_n = 466$ k·ft $= 1240$ #

W16x40 $\Sigma Q_n = 237 = \frac{237}{21.5} = 11.02 \Rightarrow 12 \times 2 = 24$ studs per girder
$\phi M_n = 463$ k·ft $= 1280$ #

most economical by inspection ← W18x35 $\Sigma Q_n = 260 = \frac{260}{21.5} = 12.09 \Rightarrow 13 \times 2 = 26$ studs per girder
$\phi M_n = 465$ k·ft $= 1170$ #

2-35

check unshored strength

$P_u = [1.2[(69 \cdot 8.67) + 35] + 1.6(20 \cdot 8.67)]32 = 31.12k$

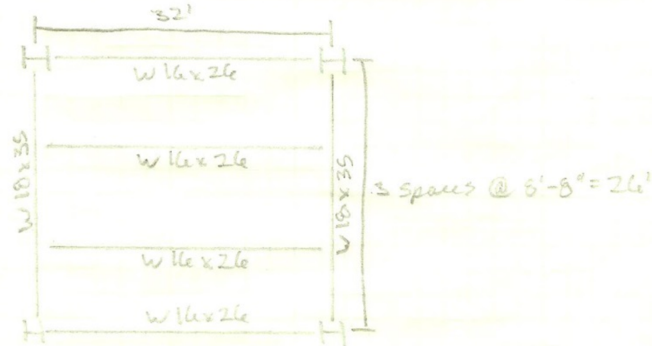
$M_u = 31.12k \cdot 8.67 = 269.81 \text{ ft} \cdot k < 465 \text{ k} \cdot \text{ft}$

check Live Load Deflection:

$\Delta_{LL} = \dfrac{13.72 \cdot 26^3 \cdot 1728}{28 \cdot 29000 \cdot 1320} = 0.389 \text{ in}$

$\Delta_{LLmax} = \dfrac{L}{360} = \dfrac{26 \cdot 12}{360} = 0.867 \text{ in} > 0.389 \text{ in ok}$

$\boxed{\text{Use W18x35 with 24 studs per girder for girders}}$



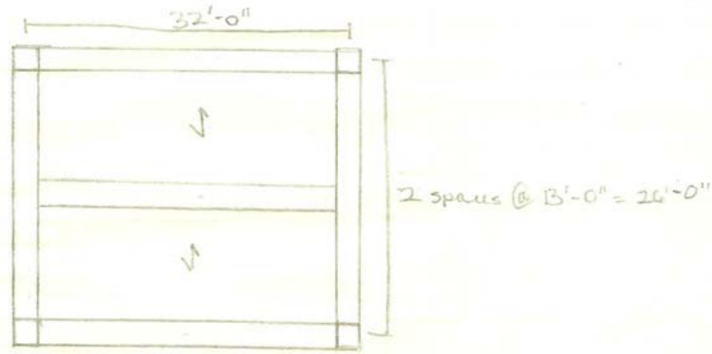
2-36

alternate System 3: one way slab with beams and girders

Loading

Live Load
office = 50 + 20 = 70 psf
corridor = 80 psf → used for building flexibility

Dead Load
carpet with pad - 2 psf
Superimposed dead load - 15 psf



Determine slab size

assume $f'_c = 5000$ psi
assume $f_y = 60$ ksi

Minimum Thickness of slab
Using ACI 318-11 Table 9.5(a)

$$h_{min} = \frac{l}{28} = \frac{13 \cdot 12}{28} = 5.57 \text{ in} \Rightarrow 6 \text{ in}, \ d = 5 \text{ in}$$

Loads

Live Loads - 80 psf

Dead Loads
carpet with pad - 2 psf
Superimposed dead load - 15 psf
slab dead load - $\frac{6}{12} \cdot 150 = 75$ psf

total - 92 psf

$w_u = 1.2(92) + 1.6(80) = 238.4$ psf

2-37

Determine design moment

$$M_u = \frac{w_u l^2}{12} = \frac{238.4 \cdot 13^2}{12} = 3357.5 \text{ lb-ft} = 3.36 \text{ k-ft}$$

Estimate As

$$A_s = M_u/4d = \frac{3.36}{4 \cdot 5} = 0.168 \text{ in}^2/\text{ft} \Rightarrow \text{Provide \#4 bar @ 12" o.c.}$$
(0.2 in²/ft)

$$d = 6" - 0.75" - \frac{0.5}{2} = 5"$$

Check $\emptyset M_n > M_u$

assume $\varepsilon_s > \varepsilon_y$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.2 \cdot 60}{0.85(5)(12)} = 0.235"$$

$$c = \frac{a}{\beta_1} = \frac{0.235}{0.8} = 0.294"$$

$$\beta_1 = 0.85 - \frac{0.05}{1000}(5000 - 4000) = 0.8 \geq 0.65$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c) = \frac{0.003}{0.294}(5 - 0.294) = 0.048 > 0.005 \rightarrow \emptyset = 0.9$$

$$\emptyset M_n = \emptyset A_s \cdot f_y (d - a/2) = 0.9(0.2)(60)\left(5 - \frac{0.235}{2}\right) = 4.39 \text{ k-ft} > 3.36 \text{ k-ft}$$
ok

Temperature and shrinkage Reinforcement

$$A_t = 0.0018\, b \cdot h = 0.0018(12)(6) = 0.13 \text{ in}^2/\text{ft}$$
$$\#4 @ 12" \text{ o.c.} = 0.2 \text{ in}^2 > 0.13 \text{ in}^2$$

Crack control

$$s \leq 15\left(\frac{40000}{2/3 f_y}\right) - 2.5 c_c = 15\left(\frac{40000}{\frac{2}{3} 60000}\right) - 2.5(0.75) = 13 \tfrac{1}{8}" \Rightarrow s = 6" \text{ ok}$$

2-38

Determine Beam Size

assume $f'c = 5000$ psi
Assume $fy = 60$ ksi

Loads

Live Load - 80 psf (reducable)

$L = Lo\left(0.25 + \frac{15}{\sqrt{K_{LL} \cdot A_T}}\right) \geq 0.5 Lo$

$= 80\left(0.25 + \frac{15}{\sqrt{24 \cdot 32}}\right) = 80 \cdot 0.771 = 61.68$ psf

$\hookrightarrow 832 > 400$

Dead Loads

Slab $= \frac{6}{12} \cdot 150 = 75$ psf

carpet with pad = 2 psf
Superimposed dead load = 15 psf

Total - 92 psf

$w_u = [1.2(92) + 1.6(61.68)]\left(\frac{13+13}{2}\right) = 2.72$ klf

$M_u = \frac{w_u \cdot l_n^2}{8} = \frac{2.72 \cdot 30.33^2}{8} = 312.8$ k·ft $\cdot 1.1 = 344.1$ k·ft

assuming girder = 20"   estimate of self weight

Estimate size:

$b \cdot d^2 = 20 M_u$, $b = 4/5 d$

$d^3 = 20(344.1)\left(\frac{5}{4}\right) = 20.5$ in

$h = d + 2.5$ in $= 23$ in $\Rightarrow 24''$, $b = 18''$

$b \cdot d^2 = 18 \cdot 21.5^2 = 8320.5$ in^3

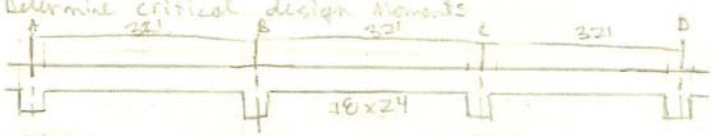
Compute self weight effects

$w_{sw} = \frac{18 \cdot 24}{144} \times 150 = 450$ plf

$w_u = 2720 + 1.2(450) = 3260$
$M_u = \frac{3.26 \cdot 30.33^2}{8} = 374.9$ k·ft $20 \cdot 374.9 = 7498$ in$^3 < 8320.5$ in^3
ok

2-39

Determine critical design Moments



Use Moment Coefficient Method (ACI 318-11 Section 8.3)

- two or more spans ✓
- spans are equal ✓
- Loads are uniformly distributed ✓
- L does not exceed 3D ✓
- members are prismatic ✓

Loads - 61.68 psf (reduced)

Dead Load

Slab = 75 psf

Beam - $\frac{18(24-6")\cdot 150}{144\cdot 13}$ = 26 psf

carpet with pad - 2 psf

superimposed dead load - 15 psf

$w_u = [1.2(118) + 1.6(61.68)]\cdot 13 = 3.12$ klf

Critical Moments

$M_{u,L} = w_u l_n^2/11 = \frac{3.12\cdot 30.33^2}{11} = 260.9$ k·ft

$M_{u,r} = \frac{3.12\cdot 30.33^2}{11} = 260.9$ k·ft

$M_u^+ = \frac{w_u l_n^2}{16} = \frac{3.12\cdot 30.33^2}{16} = 179.4$ k·ft

Determine Reinforcement

Check if T-beam behavior

$b_{eff} \leq \left[\begin{array}{l} 1/4\ \text{span length} = 32\cdot 12/4 = 96" \\ b_w + 16 h_f = 18 + 16\cdot 6 = 114" \\ b_w + 2\cdot 1/2\ \text{clr spa} = 18 + (13\cdot 12 - 20) = 156" \end{array}\right.$

$b_{eff} = 96"$

$M_{u,T\text{-}bm} = \phi\, 0.85\cdot f'c\cdot b\cdot h_f\cdot (d - h_f/2)$

$= 0.9\cdot 0.85\cdot 3\cdot 96"\cdot 6\cdot (21.5 - 6/2) = 40759$ k·in > 3130.8 k·in

since $M_u < M_{uT}$ then not T-beam behavior

2-40

Top-

$$A_{s\,req'd} = \frac{M_u}{4d} = \frac{260.1\ k\cdot ft}{4 \cdot 21.5} = 3.02\ in^2 \Rightarrow 4\ \#9\ \text{bars}$$

Check d

$$d = 24 - 1.5'' - 3/8'' - \frac{1.128}{2} = 21.6''$$

Check As,min

$$A_{s,min} \geq \left| \begin{array}{l} \frac{3\sqrt{f'c}}{fy} b \cdot d = \frac{3\sqrt{5000} \cdot 18 \cdot 21.6}{60000} = 1.37\ in^2 \\ \frac{200\ b \cdot d}{fy} = \frac{200 \cdot 18 \cdot 21.6}{60000} = 1.30\ in^2 \end{array} \right.$$

$$A_s = 4 \cdot (1.00) = 4.00\ in^2$$

$A_s > A_{s,min}$ ✓

Check As,max

$$\rho_{max} = 0.85\ \beta_1 \frac{f'c}{fy} \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85 \cdot 0.80 \frac{5}{60} \cdot \frac{0.003}{0.003 + 0.004}$$
$$= 0.0243$$

$$A_{s,max} = 0.0243 \cdot 18 \cdot 21.6 = 9.45\ in^2$$

$A_s < A_{s,max}$ ✓

Determine Mn

assume $f_s = f_y$

$$a = \frac{A_s \cdot f_y}{0.85\ f'c \cdot b} = \frac{4.0 \cdot 60}{0.85 \cdot 5 \cdot 18} = 3.14\ in$$

$$c = a/\beta_1 = \frac{3.14}{0.80} = 3.93\ in$$

Check $\varepsilon_s > \varepsilon_y$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c) = \frac{0.003}{3.93}(21.6 - 3.93) = 0.0135 > 0.005$$

$$\phi M_n = \phi A_s \cdot f_y (d - a/2) = 0.9 \cdot 4 \cdot 60 \cdot (21.6 - \frac{3.14}{2}) = 360.54\ k\cdot ft$$

$360.5\ k\cdot ft > 260.9\ k\cdot ft$

Use 4 #9 @ top

2-41

Bottom

$A_s\,req'd = \frac{M_u}{4d} = \frac{179.4\ k\cdot ft}{4 \cdot 21.5} = 2.09\ in^2 \Rightarrow$ 3 #9 bars

check d

$d = 24 - 1.5 - 3/8'' - \frac{1.128}{2} = 21.6''$

Check $A_{s,min}$

$$A_{s,min} \geq \begin{cases} \frac{3\sqrt{f'_c}\, b \cdot d}{f_y} = \frac{3\sqrt{5000} \cdot 18 \cdot 21.6}{60000} = 1.37\ in^2 \\ \frac{200\, b \cdot d}{f_y} = \frac{200 \cdot 18 \cdot 21.6}{60000} = 1.3\ in^2 \end{cases}$$

$A_s = 3 \cdot 1.00 = 3.00\ in^2$

$A_s > A_{s,min}$ ✓

Check $A_{s,max}$

$$\rho_{max} = \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85 \cdot 0.80 \cdot \frac{5}{60} \cdot \frac{0.003}{0.003 + 0.004}$$

$= 0.0243$

$A_{s,max} = 0.0243 \cdot 18 \cdot 21.6 = 9.45\ in^2$

$A_s < A_{s,max}$ ✓

Determine M_n

assume $f_s = f_y$

$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{3.0 \cdot 60}{0.85 \cdot 5 \cdot 18} = 2.35\ in$

$c = a/\beta_1 = \frac{2.35}{0.8} = 2.94\ in$

Check $\varepsilon_s > \varepsilon_y$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c) = \frac{0.003}{2.94}(21.6 - 2.94) = 0.019 > 0.005$

$\phi M_n = \phi A_s \cdot f_y (d - a/2) = 0.9 \cdot 3 \cdot 60 \cdot (21.6 - \frac{2.35}{2}) = 275.7\ k\cdot ft$

$275.7\ k\cdot ft > 179.4\ k\cdot ft$ use 3 #9 @ bottom

2-42

Determine Shear Reinforcement

Determine Shear Strength without stirrups

$V_c = 2\sqrt{f'_c} \cdot b_w \cdot d = 2 \cdot \sqrt{5000} \cdot 18 \cdot 21.6/1000 = 54.98\,k$

$\emptyset V_n = 0.5 \emptyset V_c = 0.5 \cdot 0.75 \cdot 54.98\,k = 20.62\,k$

Determine shear Strength required by reinforcing

$V_u = \frac{w_u l_n}{2} = \frac{3.12 \cdot 30.33}{2} = 47.31\,k$

$V_s = V_u/\emptyset - V_c = \frac{47.31}{0.75} - 54.98 = 8.1\,K$

$V_{s,max} = 8\sqrt{f'_c}\, b_w \cdot d = 8\sqrt{5000} \cdot 18 \cdot 21.6 = 219.9\,k > V_s$ ok

Maximum spacing of shear reinforcement

$4\sqrt{f'_c} \cdot b_w \cdot d = 4 \cdot \sqrt{5000} \cdot 18 \cdot 21.6 = 110\,k > V_s$

$S_{max} = \min \begin{cases} d/2 = 10.8'' \leftarrow \text{controls} \\ 24'' \end{cases}$

Minimum shear reinforcement

$A_{v,min} = \max \begin{cases} \frac{0.75\sqrt{f'_c}\, b_w \cdot s}{f_{yt}} = \frac{0.75 \cdot \sqrt{5000} \cdot 18 \cdot 10.8}{60000} = 0.172\,in^2 \\ \frac{50\, b_w \cdot s}{f_{yt}} = \frac{50 \cdot 18 \cdot 10.8}{60000} = 0.162\,in^2 \end{cases}$

(2) legs of #3 $= 2 \cdot 0.11 = 0.22\,in^2 > 0.172\,in^2$

Design Shear Reinforcement

$V_s = A_v \cdot \frac{d}{s} f_{yt} \Rightarrow s = \frac{A_v \cdot f_{yt} \cdot d}{V_s} = \frac{0.22 \cdot 60 \cdot 21.6}{6.38} = 44.7'' > s_{max}$

Use #3 @ 10" o.c.

Minimum thickness to control deflections

$h = l/21 = \frac{32 \cdot 12}{21} = 18.29'' < 24''$

2-43

Determine Girder A size

assume $f'c = 5000$ psi
assume $f_y = 60$ ksi

Loads

Live Load - 80 psf (reduceable)

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} \cdot A_T}}\right) \geq 0.5 L_o$$

$$= 80\left(0.25 + \frac{15}{\sqrt{\underbrace{24 \cdot 32}_{832 > 400}}}\right) = 80 \cdot 0.771 = 61.68 \text{ psf}$$

Dead Loads

Slab = 75 psf
carpet with pad - 2 psf
Superimposed dead load = 15 psf

total - 92 psf

$$w_u = [1.2(92) + 1.6(61.68)]\left(\frac{13+13}{2}\right) = 2.72 \text{ klf}$$

$$M_u = \frac{w_u \cdot l_n^2}{8} = \frac{2.72 \cdot 30.33^2}{8} = 312.8 \text{ k·ft} \cdot 1.1 = 344.1 \text{ k·ft}$$

assuming column 20"

estimate of self weight

Estimate size

$b \cdot d^2 = 20 M_u$; $b = 20$ in

$$20 = \frac{20 \cdot 344.1}{d^2} \quad d = 18.6 \text{ in} \rightarrow h = 24 \text{ in min}, \; d = 21.5 \text{ in}$$

$b \cdot d^2 = 20 \cdot 21.6^2 = 9245 \text{ in}^3$

Compute self weight effects

$$W_{sw} = \frac{20 \cdot 24}{144} \times 150 = 500 \text{ plf}$$

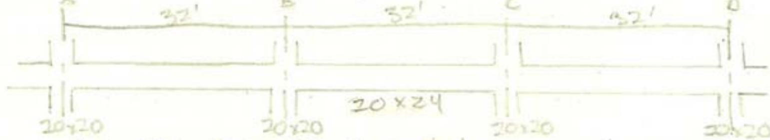
$$w_u = 2720 + 1.2(500) = 3320 \text{ plf}$$

$$M_u = \frac{3.32 \cdot 30.33^2}{8} = 381.76 \text{ k·ft} \quad 20 \cdot 381.76 = 7635 < 9245 \text{ in}^3$$

ok

2-44

Determine critical design Moments



Use Moment Coefficient Method

- two or more spans ✓
- spans are equal ✓
- Loads are uniformly distributed ✓
- L does not exceed 3L ✓
- Members are prismatic ✓

Loads - 61.68 psf

Dead Load

Slab = 75 psf

girder - $\frac{20 \cdot (24-6) \cdot 150}{144 \cdot 13} = 29$ psf

carpet with pad - 2 psf

Superimposed dead load - 15 psf

$w_u [1.2(121) + 1.6(61.68)] \cdot 13 = 3.17$ K/ft

Critical Moments

$M_u^-,l = \frac{w_u l_n^2}{11} = \frac{3.17 \cdot 30.33^2}{11} = 265.1$ k·ft

$M_u^-,r = \frac{w_u l_n^2}{11} = \frac{3.17 \cdot 30.33^2}{11} = 265.1$ k·ft

$M_u^+ = \frac{w_u l_n^2}{16} = \frac{3.17 \cdot 30.33^2}{16} = 182.3$ k·ft

Determine Reinforcement

Check if T-beam behavior

$b_{eff} \leq \begin{cases} 1/4 \text{ span length} = 32 \cdot 12/4 = 96'' \\ b_w + 16h_f = 20 + 16 \cdot 6 = 116'' \\ b_w + \Sigma 1/2 \text{ bm clr span} = 20 + (15 \cdot 12 - 18) = 158'' \end{cases}$

$b_{eff} = 96''$

$M_{u,T\text{-}bm} = \phi 0.85 \cdot f'c \cdot b \cdot h_f \cdot (d - h_f/2)$

$= 0.9 \cdot 0.85 \cdot 5 \cdot 96 \cdot 6 \cdot (21.5 - 6/2) = 40759$ k·in > 3130.8 k·in

Since $M_u < M_{u,T\text{-}beam}$ then no T-beam behavior

2-45

Top -

$A_s\ req'd = \frac{M_u}{4d} = \frac{265.1}{4 \cdot 21.5} = 3.08\ in^2 \rightarrow 4\#9$ bars

Check d

$d = 24 - 1.5'' - 3/8'' - \frac{1.128}{2} = 21.6''$

check $A_{s,min}$

$$A_{s,min} \geq \left| \begin{array}{l} \frac{3\sqrt{f'c}}{fy} b \cdot d = \frac{3\sqrt{5000} \cdot 20 \cdot 21.6}{60000} = 1.53 in^2 \\ \frac{200\, b \cdot d}{fy} = \frac{200 \cdot 20 \cdot 21.6}{60000} = 1.44 in^2 \end{array} \right.$$

$A_s = 4 \cdot 1.00 = 4.00 in^2$

$A_s > A_{s,min}$ ✓

Check $A_{s,max}$

$$\rho max = 0.85\, \beta_1 \frac{f'c}{fy} \cdot \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85 \cdot 0.80 \frac{5}{60} \cdot \frac{0.003}{0.003 + 0.004}$$
$$= 0.0243$$

$A_{s,max} = 0.0243 \cdot 20 \cdot 21.6 = 10.5 in^2$

$A_s < A_{s,max}$ ✓

Determine Mn

assume $f_s = f_y$

$$a = \frac{A_s \cdot f_y}{0.85 f'c \cdot b} = \frac{4.0 \cdot 60}{0.85 \cdot 5 \cdot 20} = 2.82 in$$

$$c = a/\beta_1 = \frac{2.82}{0.8} = 3.53 in$$

Check $\varepsilon_s > \varepsilon_y$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c) = \frac{0.003}{3.53}(21.6 - 3.53) = 0.015 > 0.005$$

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9 \cdot 4 \cdot 60 (21.6 - \frac{2.82}{2}) = 363.42\ k \cdot ft$$

$363.4 k \cdot ft > 265.1 k \cdot ft$

Use 4#9 @ top

Bottom

$A_s \text{ req'd} = \frac{M_u}{4d} = \frac{182.3}{4 \cdot 21.5} = 2.12 \, in^2 \rightarrow 3 \ \#9 \text{ bars}$

Check d

$d = 24 - 1.5 - 3/8" - \frac{1.128}{2} = 21.6"$

Check As,min

$$A_{s,min} \geq \left| \begin{array}{l} \frac{3\sqrt{f'_c} \cdot b \cdot d}{f_y} = \frac{3 \sqrt{5000} \cdot 20 \cdot 21.6}{60000} = 1.53 \, in^2 \\ \frac{200 \, b \cdot d}{f_y} = \frac{200 \cdot 20 \cdot 21.6}{60000} = 1.44 \, in^2 \end{array} \right.$$

$A_s = 3 \cdot 1.00 = 3.00 \, in^2$

$A_s > A_{s,min}$

Check As, max

$\rho_{max} = 0.85 \cdot \beta_1 \cdot \frac{f'_c}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85 \cdot 0.80 \cdot \frac{5}{60} \cdot \frac{0.003}{0.003 + 0.004}$
$= 0.0243$

$A_{s,max} = 0.0243 \cdot 20 \cdot 21.6 = 10.5 \, in^2$

$A_s < A_{s,max}$ ✓

Determine Mn

assume $f_s = f_y$

$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{3.0 \cdot 60}{0.85 \cdot 5 \cdot 20} = 2.12 \, in$

$c = a / \beta_1 = \frac{2.12}{0.8} = 2.65 \, in$

Check $\varepsilon_s > \varepsilon_y$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c) = \frac{0.003}{2.65}(21.6 - 2.65) = 0.0215 > 0.005$

$\phi M_n = \phi A_s \cdot f_y (d - \frac{a}{2}) = 0.9 \cdot 3 \cdot 60 (21.6 - \frac{2.65}{2}) = 273.7 \, k \cdot ft$

$273.7 \, k \cdot ft > 182.5 \, k \cdot ft$

use 3 #9 @ bottom

2-47

Determine Shear Reinforcement

Determine shear strength without stirrups.

$V_c = 2\sqrt{f'_c}\, b_w \cdot d = 2 \cdot \sqrt{5000} \cdot 20 \cdot 21.6/1000 = 61.1\text{k}$

$\emptyset V_n = 0.5 \emptyset V_c = 0.5 \cdot 0.75 \cdot 61.1 = 22.9\text{k}$

Determine shear strength required by reinforcing

$V_u = \frac{w_u l_n}{2} = \frac{3.17 \cdot 30.33}{2} = 48.07\text{k}$

$V_s = V_u/\emptyset - V_c = \frac{48.07}{0.75} - 61.1 = 3\text{k}$

$V_{s,max} = 8\sqrt{f'_c}\, b_w \cdot d = 8\sqrt{5000} \cdot 20 \cdot 21.6 = 244.4\text{k} > V_s$ ✓

Maximum spacing of shear reinforcement

$4\sqrt{f'_c} \cdot b_w \cdot d = 4 \cdot \sqrt{5000} \cdot 20 \cdot 21.6 = 122.19\text{k} > V_s$

$S_{max} = \min \begin{cases} d/2 = 10.8'' \leftarrow \text{controls} \\ 24'' \end{cases}$

Minimum Shear Reinforcement

$A_{v,min} = \max \begin{cases} \frac{0.75\sqrt{f'_c} \cdot b_w \cdot s}{f_{yt}} = \frac{0.75 \cdot \sqrt{5000} \cdot 20 \cdot 10.8}{60000} = 0.19\ \text{in}^2 \\ \frac{50\, b_w \cdot s}{f_{yt}} = \frac{50 \cdot 20 \cdot 10.8}{60000} = 0.18\ \text{in}^2 \end{cases}$

(2) Legs of #3 $= 2 \cdot 0.11 = 0.22\ \text{in}^2 > 0.19\ \text{in}^2$

Design Shear Reinforcement

$V_s = A_v \cdot \frac{d}{s} f_{yt} \Rightarrow s = \frac{A_v \cdot f_{yt} \cdot d}{V_s} = \frac{0.22 \cdot 60 \cdot 21.6}{3} = 95\ \text{in} > s_{max}$

Use #3 @ 10" o.c.

Minimum thickness to control deflections

$h = l/21 = \frac{32 \cdot 12}{21} = 18.29 > 24''$

2-48

Determine Girder B size

Assume $f'_c = 5000$ psi
Assume $f_y = 60$ ksi

Loads

Live Load - 80 psf (reducable)

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} \cdot A_T}}\right) \geq 0.5 L_o$

$= 80\left(0.25 + \frac{15}{\sqrt{32 \cdot 2 \cdot 26}}\right) = 80 \cdot 0.563 = 45.04$ psf

$\to 1664 > 400$

Dead Loads

Slab - 75 psf
carpet with pad - 2 psf
superimposed dead load - 15 psf
beam self weight - 26 psf

total - 118

$w_u = 1.2 \cdot (118) + 1.6(45.04) = 213.67$ psf

$P_u = 213.67(13)(32) = 88.9$ k

$M_u = \frac{P l_n}{4} = \frac{88.9 \cdot (25 - \frac{28}{12})}{4} = 540.8$ k·ft $\cdot 1.2 = 649$ k·ft

estimate of self weight

Estimate Size

$b \cdot d^2 = 20 M_u$, $b = 20$ in

$20 = 20 \cdot \frac{649}{d^2}$ $d = 25.5$ in $\to h = 28$ in
$d = 25.5$ in

$b \cdot d^2 = 20 \cdot 25.5^2 = 13005$ in^3

Compute self weight effects

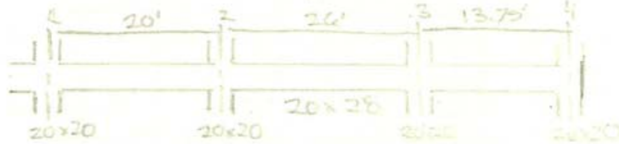
$w_{sw} = \frac{20 \cdot 28}{144} \times 150 = 583$ plf

$P_u = 88.9$ k $+ 0.583 \cdot 26 = 104.1$ k

$M_u = \frac{104.1 \cdot (26 - \frac{28}{12})}{4} = 633.3$ k·ft $20 \cdot 633.3 = 12666$ in^3
< 13005 in^3

2-49

Determine critical design moments



Use moment coefficient method

Loads

Live Load - 45.04 psf

Dead Load
slab - 75 psf
beam - 26 psf
girder - $\frac{20 \cdot (28-6) \cdot 150}{144 \cdot 32} = 14.3$ psf
carpet with pad - 2 psf
Superimposed dead load - 15 psf

$w_u = [1.2(132.3) + 1.6(45.04)] \cdot 32 = 7.39$ klf

Critical Moments

$M_{u^-,l} = \frac{w_u l_n^2}{11} = \frac{7.39 \cdot 24.33^2}{11} = 397.7$ k·ft

$M_{u^-,r} = \frac{w_u l_n^2}{11} = \frac{7.39 \cdot 24.33^2}{11} = 397.7$ k·ft

$M_u^+ = \frac{w_u l_n^2}{16} = \frac{7.39 \cdot 24.33}{16} = 273.41$ k·ft

Determine Reinforcement

Top:

A_s req'd $= \frac{M_u}{4d} = \frac{397.7}{4 \cdot 25.5} = 3.89$ in^2 → 4 #9 bars

check d

$d = 28 - 1.5'' - 3/8'' - \frac{1.128}{2} = 25.6$ in

2-50

Check $A_{s,min}$

$$A_{s,min} \geq \begin{cases} \dfrac{3\sqrt{f'c}\cdot b\cdot d}{f_y} = \dfrac{3\sqrt{5000}\cdot 20\cdot 25.6}{60000} = 1.81\,in^2 \\ \dfrac{200\, b\cdot d}{f_y} = \dfrac{200\cdot 20\cdot 25.6}{60{,}000} = 1.71\,in^2 \end{cases}$$

$A_s = 4\cdot 1.00 = 4\,in^2$

$A_s > A_{s,min}$ ✓

Check $A_{s,max}$

$$\rho_{max} = 0.85\,\beta_1 \frac{f'c}{f_y}\cdot\frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85\cdot 0.80\cdot\frac{5}{60}\cdot\frac{0.003}{0.003+0.004}$$
$$= 0.0243$$

$A_{s,max} = 0.0243\cdot 20\cdot 25.6 = 12.4\,in^2$

$A_s < A_{s,max}$ ✓

Determine M_n

assume $f_s \geq f_y$

$$a = \frac{A_s\cdot f_y}{0.85\cdot f'c\cdot b} = \frac{4.0\cdot 60}{0.85\cdot 5\cdot 20} = 2.82\,in$$

$$c = a/\beta_1 = \frac{2.82}{0.8} = 3.53\,in$$

Check $\varepsilon_s > \varepsilon_y$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{3.53}(25.6 - 3.53) = 0.0188 > 0.005$$

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9\cdot 4\cdot 60\cdot\left(25.6 - \frac{2.82}{2}\right) = 435.4\,k\text{-}ft$$

$435.4\,k\text{-}ft > 397.7\,k\text{-}ft$

use 4 #9 @ top

2-51

Bottom

$A_s\,req'd = \frac{M_u}{4d} = \frac{273.41}{4 \cdot 21.5} = 3.12\ in^2 \rightarrow 4\ \#9\ bars$

Check $A_{s,min}$

$$A_{s,min} \geq \begin{cases} \frac{3\sqrt{f'_c} \cdot b \cdot d}{f_y} = \frac{3\sqrt{5000} \cdot 20 \cdot 25.6}{60{,}000} = 1.53\ in^2 \\ \frac{200\ b \cdot d}{f_y} = \frac{200 \cdot 20 \cdot 25.6}{60{,}000} = 1.3\ in^2 \end{cases}$$

$A_s = 4 \cdot 1.00 = 4.00\ in^2$

$A_s > A_{s,min}$

Check $A_{s,max}$

$\rho\,max = 0.85 \cdot \beta_1 \cdot \frac{f'_c}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85 \cdot 0.8 \cdot \frac{5}{60} \cdot \frac{0.003}{0.003 + 0.004}$
$= 0.0243$

$A_{s,max} = 0.0243 \cdot 20 \cdot 25.6 = 12.44\ in^2$

$A_s < A_{s,max}$ ✓

Determine M_n

assume $f_s > f_y$

$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{4.0 \cdot 60}{0.85 \cdot 5 \cdot 20} = 2.82\ in$

$c = a/\beta_1 = \frac{2.82}{0.8} = 3.53\ in$

Check $\varepsilon_s > \varepsilon_y$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{3.53}(25.6 - 3.53) = 0.0188 > 0.005$

$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(4)(60)(25.6 - \frac{2.82}{2}) = 435.4\ k\text{-}ft$

$435.4\ k\text{-}ft > 273.41\ k\text{-}ft$

Use 4 #9 @ bottom

2-57

Determine Shear Reinforcement

Determine shear strength without stirrups

$V_c = 2\sqrt{f'_c}\, b_w \cdot d = 2\sqrt{5000} \cdot 20 \cdot 25.6 / 1000 = 72.4k$

$\phi V_n = 0.5\, \phi V_c = 0.5 \cdot 0.75 \cdot 72.4k = 27.2k$

Determine shear strength required by reinforcing

$V_u = \frac{w_u l_n}{2} = \frac{7.39 \cdot 24.33}{2} = 89.9k$

$V_s = V_u/\phi - V_c = \frac{89.9}{0.75} - 72.4 = 47.5k$

$V_{s,max} = 8\sqrt{f'_c}\, b_w \cdot d = 8\sqrt{5000} \cdot 20 \cdot 25.6 = 289.6k > V_s$ ok

Maximum Spacing of Shear reinforcement

$4\sqrt{f'_c} \cdot b_w \cdot d = 4\sqrt{5000} \cdot 20 \cdot 25.6 = 144.8k > V_s$ ok

$s_{max} = \min \begin{cases} d/2 = 12.8'' \leftarrow \text{controls} \\ 24'' \end{cases}$

Minimum shear reinforcement

$A_{v,min} = \max \begin{cases} \frac{0.75\sqrt{f'_c} \cdot b_w \cdot s}{f_{yt}} = \frac{0.75 \cdot \sqrt{5000} \cdot 20 \cdot 12.8}{60000} = 0.226\,in^2 \\ \frac{50\, b_w \cdot s}{f_{yt}} = \frac{50 \cdot 20 \cdot 12.8}{60000} = 0.213\,in^2 \end{cases}$

(3) legs of #3 $= 3 \cdot 0.11 = 0.33\,in^2 > 0.226\,in^2$

Design Shear Reinforcement

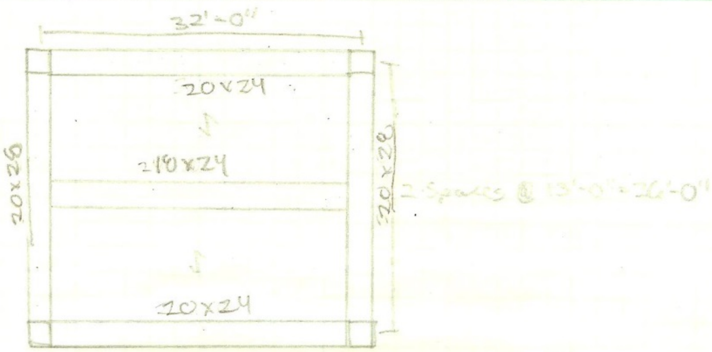
$V_s = A_v \cdot \frac{d}{s} f_{yt} \Rightarrow s = \frac{A_v \cdot f_{yt} \cdot d}{V_s} = \frac{0.33 \cdot 60 \cdot 25.6}{47.5} = 10.67\,in$

use (3) legs #3 @ 10" o.c.

Minimum Thickness to control deflections

$h = l/21 = \frac{26 \cdot 12}{21} = 14.85'' < 28''$

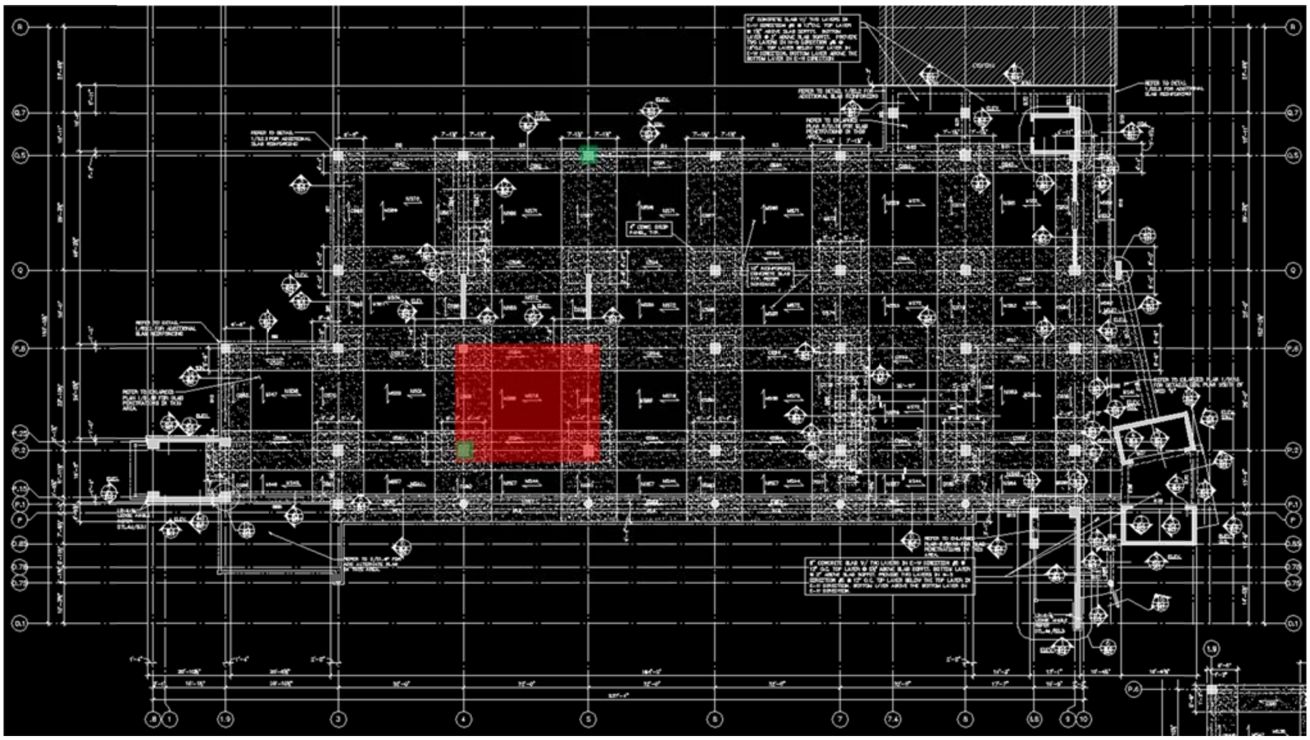
2-53



Beam Schedule				
Member	f'c	Top Flexural Reinforcement	Bottom Flexural Reinforcement	Shear Reinforcement
18x24	5000	4#9	3#9	(2) Legs #3 @ 10" o.c.
20x24	5000	4#9	3#9	(2) Legs #3 @ 10" o.c.
20x28	5000	4#9	4#9	(3) Legs #3 @ 10" o.c.

Member	f'c	Longitudinal Reinforcement	Transverse Reinforcement	Thickness
slab	5000	#4 bar @ 12 in o.c.	#4 bar @ 12 in o.c.	6"

Appendix A - Floor Plan with Typical Bay and Columns



Typical bay is outlined in red. Interior and exterior columns are outlined in green.