

October 20, 2013

Heather Sustersic  
had132@psu.edu

Dear Professor Sustersic,

The following technical report was written to fulfill the requirements specified in the Structural Technical Report 3 assignment that was handed out on September 27, 2013.

Technical report 3 includes a detailed structural analysis of the existing floor system used in the New Library at the University of Virginia's College at Wise, located in Wise, Virginia. This analysis includes an evaluation of a typical bay floor framing under gravity loads, and an evaluation of interior and exterior column under these same gravity loads.

Technical report 3 also includes structural designs and analysis of three alternate framing systems. The design and analysis of each system includes calculations for preliminary sizing and checks for strength and deflections. These systems will be considered for possible options for the redesign to be completed next spring.

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

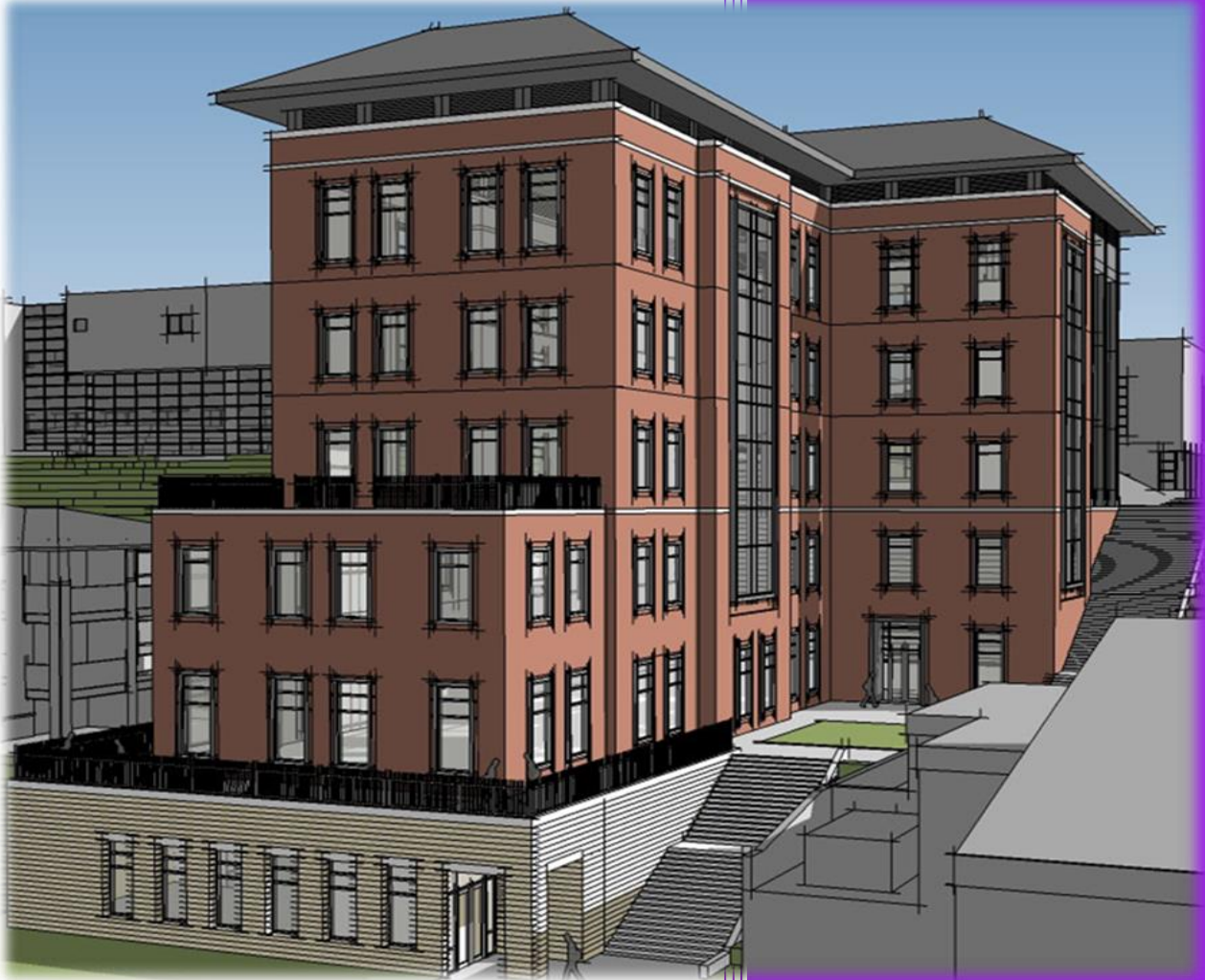
Sincerely,

Macenzie Ceglar

Enclosed: Technical Report 3

# Technical Report 3

## University of Virginia's College at Wise New Library



Macenzie Ceglar  
Structural Option  
Advisor: Heather Sustersic  
18 October 2013

## Executive Summary

The New Library at the University of Virginia's College at Wise will serve as a main link between the upper and lower campus areas, which are currently divided by a steep 60 foot hill. The new 6 story, 68,000 ft<sup>2</sup>, library will be integrated into the hillside, and will provide students with an easier and safer path across campus. The architectural design of the façade incorporates traditional materials found on campus, such as brick and stone. Construction on the New Library began in August 2012 and will be completed in August 2015.

Soil loads caused the foundation system for the New Library to be unique in its design. The foundation system utilizes a temporary leave-in-place soil retention system and foundation walls which are designed to resist future lateral soil loads. Other parts of the foundation system include piers, footings, and slabs-on-grade.

All six stories of the building have composite floor framing involving both composite steel wide flange members and composite decking. Framing layout in the building is fairly typical with bay sizes ranging between 25'-4" x 25'4" and 31'-0" x 25'-4". Steel wide flange columns are used as the vertical framing system and shear walls make up the building's lateral system.

Loading conditions considered in the building's design include live loads, gravity loads, snow loads, wind loads, seismic loads, and lateral soil loads.

The Virginia Uniform Statewide Building Code (USBC); along with "Facility Design Guidelines", governs the design of all buildings on the campus. The USBC adopts chapters 2-35 of International Building Code (IBC) 2009, which references codes and standards which include American Society of Civil Engineers (ASCE) 7-05, American Concrete Institute (ACI) 318-08, and the 13<sup>th</sup> edition of the Steel Construction Manual.

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# University of Virginia's College at Wise - New Library

Wise, VA

## General Information

**Full Height:** 119'  
**Number of Stories:** 6  
**Size:** 68,000 GSF  
**Cost:** \$43 Million  
**Date of Construction:** Aug 2012 – Aug 2015  
**Project Delivery Method:** Design-Bid-Build

## Project Team

**Owner:** UVA at Wise  
**Architect:** Cannon Design  
**Structural:** Cannon Design  
**MEP:** Thompson and Litton  
**Lighting:** Lafleur Associates  
**Construction:** Quesenberrys, Inc.  
**Civil:** Thompson and Litton  
**Landscape:** Hill Studio  
**AV/Acoustics:** Shen Milsom Wilke  
**Foodservice:** Culinary Advisors

*Project Sponsor:*



## Architecture

The goal of the façade design was to give the impression that the older existing buildings' architecture was based on the New Library's. This was achieved through use of materials such as brick and stone commonly found on the surrounding buildings.

## Construction

Limited site area due to existing campus buildings impacted the construction by requiring offset staging and storage areas, along with the construction of a 500 foot service road.



## Structural Systems

**Foundation:** Slab on grade with column piers, footings and foundation walls

**Framing:** Steel frame, composite wide flange steel members, and normal weight composite deck flooring

**Lateral:** 9 Reinforced concrete shear walls

**Soil Retention:** Temporary Leave-In-Place Soil Retention System, which includes the use of soil nails and shotcrete covering.

## Mechanical

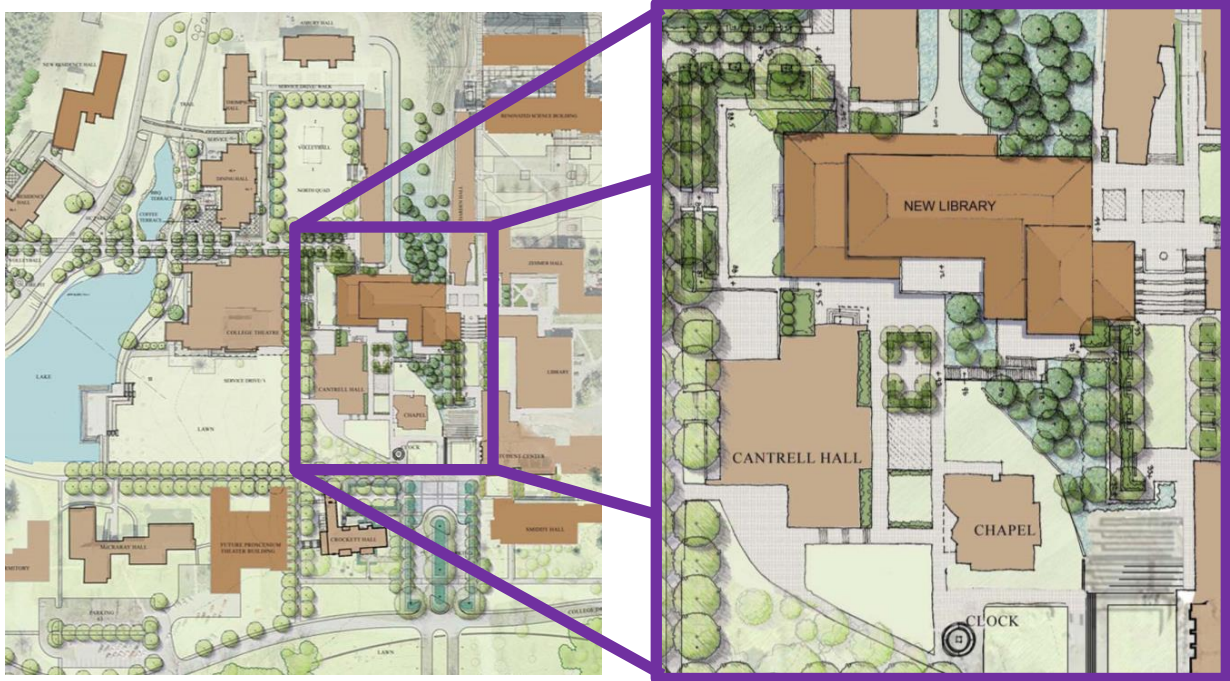
VAV system with a roof mounted chilled-water AHU and 145.9 ton chiller providing 41,300 CFM, and an economizer and an a heat recovery unit

## Electrical/Lighting

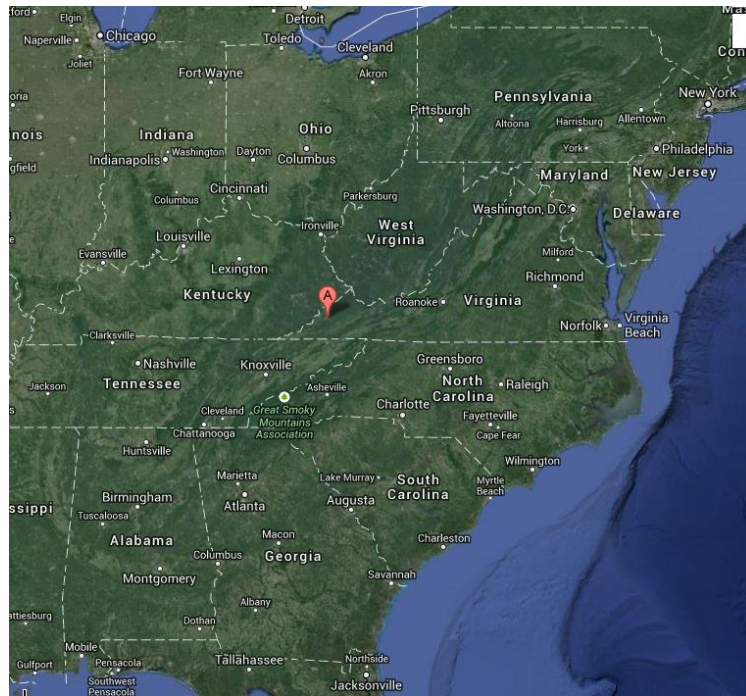
Five 480/277 3-phase panel boards  
Nine 280/120 3-phase panel boards

Wall switch and low voltage occupancy sensors used for lighting control

## Site Plan



## Location Plan



## Documents Used in Preparation of Report

Below is a list of the design codes and standards used in the structural analysis of the New Library at the University of Virginia's College at Wise.

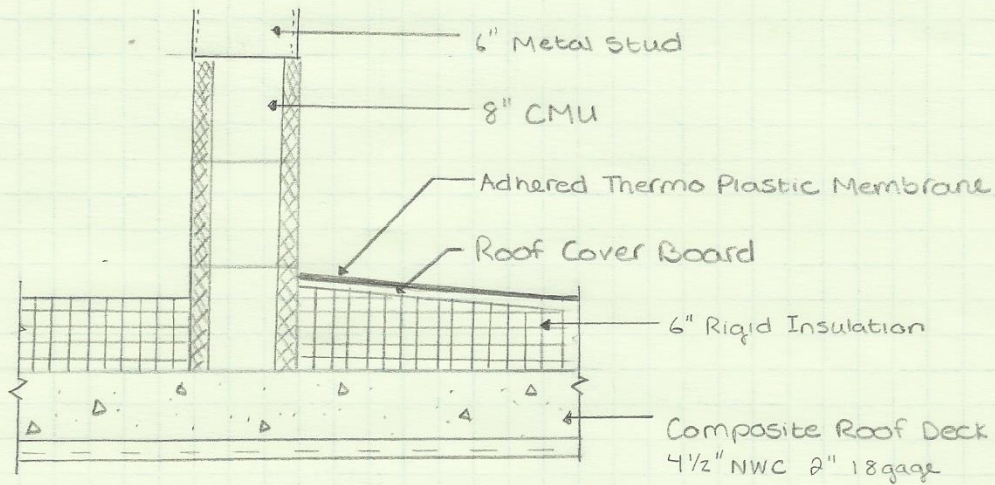
- **International Code Council**
  - International Building Code 2009 (Chapters 2-35 Adopted by Virginia Uniform Statewide Building Code)
- **American Society of Civil Engineers**
  - ASCE 7-05: Minimum Design Loads for Buildings and Other Structures
- **American Concrete Institute**
  - ACI 318-11: Building Code Requirements for Structural Concrete
- **American Institute of Steel Construction**
  - Steel Construction Manual 13<sup>th</sup> Edition – LRFD
- **Concrete Reinforcing Steel Institute**
  - CRSI Handbook 2008
- **Reinforced Concrete Mechanics and Design 6<sup>th</sup> Edition**
  - By: James K. Wight, James G. MacGregor
- **Vulcraft Deck Catalog**
- **University of Virginia Facilities Management and University Building Official**
  - Facility Design Guidelines
- **University of Virginia's College at Wise – New Library**
  - Construction Documents
  - Specifications

## **Gravity Loads from Technical Report 2**



## Typical Roof Bay Dead Loading

### Cross section of lower roof construction



### Uniformly Distributed Dead Loads

Composite roof deck = 69 psf

6" Rigid Insulation = 9 psf

Roof cover board = 2 psf

Adheared Membrane = 2 psf

Superimposed misc:

ceiling = 5 psf

Mechanical = 10 psf

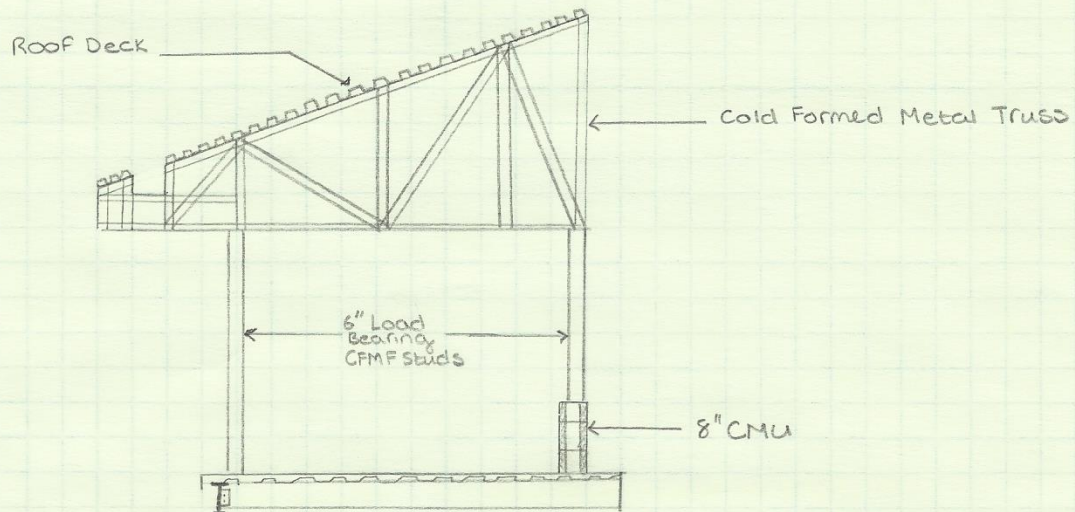
Sprinklers = 10 psf

Framing Allowance = 10 psf

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Total = 117 psf



Distributed Line Loads on RoofCross section of upper roof constructionDistributed line load from CMU wall bearing trusses

$$\text{Cold Formed Metal Trusses} = 2 \text{ psf}$$

$$\text{Spacing} = 12" \text{ o.c.}$$

$$\text{Truss Length} = 23.1'$$

$$2 \text{ psf} \times 23.1 = 46.2 \text{ plf}$$

$$\text{Load on CMU wall} = \frac{46.2}{2} = 23.1 \text{ plf}$$

$$8" \text{ CMU} = 55 \text{ psf}$$

$$\text{wall height} = 2 \text{ ft}$$

$$\text{Load from CMU wall} = 55 \times 2 = 110 \text{ plf}$$

$$\boxed{\text{Total} = 134 \text{ plf}}$$



Typical Roof Bay Live Loading

<u>LOADS</u>	<u>Design Value</u>	<u>Code Minimum</u>
Minimum Roof Live Load	30 psf	20 psf
Roof Area Below Sloped Roof	30 psf	-
Roof Mechanical Area	150 psf	-

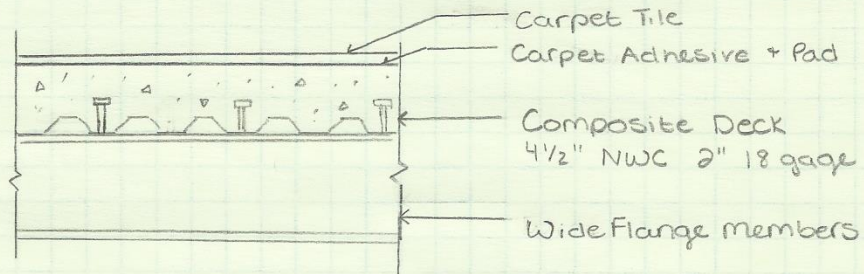
Reason for Differences

Minimum Roof Live Load : UVA Facility Guidelines specifies a minimum roof live load which overrules ASCE7-05

Roof Area Below Sloped Roof : unlikely that this area will see a live load so a minimum was used

Roof Mechanical Area : Final mechanical system was unknown so design team provided a large enough allowance



Typical Floor Bay Dead LoadingCross Section of Floor CalculationUniformly Distributed Dead Loads

Composite Deck = 69 psf

Carpet Tile = 1 psf

Pad + Adhesive = 0.5 psf

Super imposed misc:

ceiling = 5 psf

Mechanical = 10 psf

Sprinklers = 10 psf

Framing Allowance = 10 psf

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Total = 105.5 psf  $\Rightarrow$  106 psf



Non-Typical Dead Loads

Loads	Location	Value	Justification
Roof Deck 1/2" 20gauge	upper roof	2.16 psf	Vulcraft Catalog Pg 9 (1.5A Roof Deck)
Composite Deck 8 1/2" NWC 2" 18g	Level 4 supporting Vestibule area	105 psf	Vulcraft Catalog Pg 52 (6 psf / 0.5" of topping)
3/16" Terrazzo Tile 24" x 24"	Level 4 Vestibule and in Stair wells	2 psf	ASCE 7-10 Pg 402



Typical Floor Bay Live Loading

Loads	Design Value	Code Minimum	Justification
offices	50	50	ASCE7-05
Corridor (Not First Floor)	80	Same as area served	Office + Partitions $\approx$ 80
Partitions	27	-	Canon Design Standard

⇒ These loads pertain to the typical bay specified in Technically Report 1. They are found in a large majority of the building. Library stacks make up a large part of the live loading, but are not located in the specified bay.

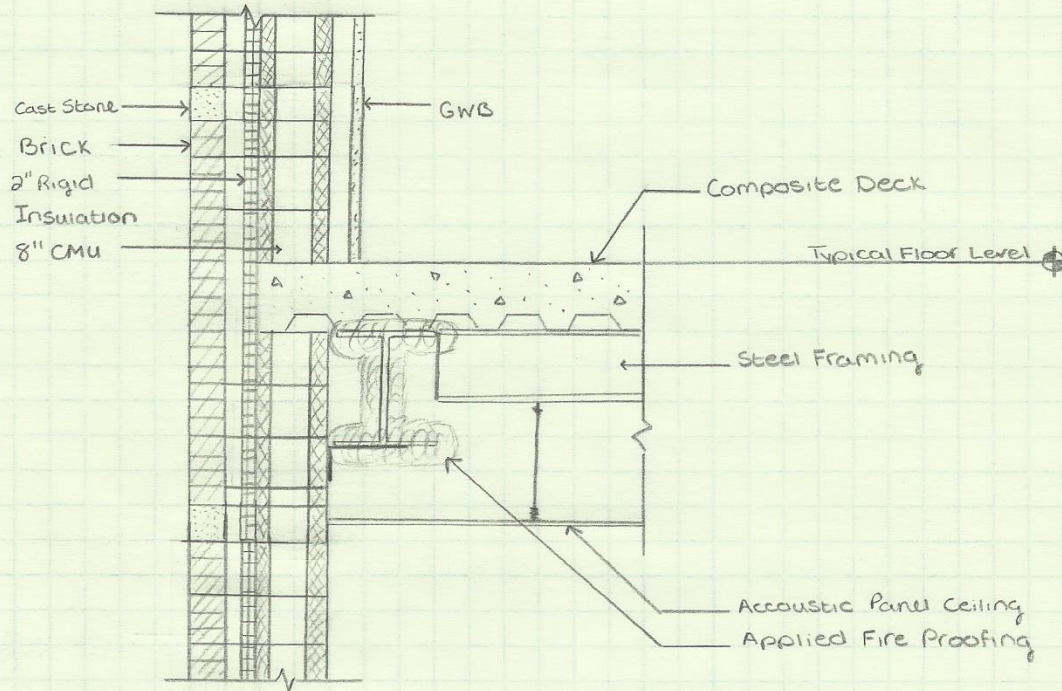


Non-Typical Live Loads

<u>Loads</u>	<u>Location</u>	<u>Design value</u>	<u>Code Min.</u>	<u>Justification</u>
Library Stack Rooms	Level 2, 3, 4, 5, 6 in various locations	150 psf	150 psf	ASCE 7-05
Mechanical Rooms	Level 2, lower roof	250 psf	-	Design load based on equipment weight
High Density Storage	Level 1	250 psf	250 psf	ASCE 7-05
Stairs	center, east corner, and south corner of building	100 psf	100 psf	ASCE 7-05

AMRAD



Typical Exterior Wall LoadingCross Section of Typical Exterior WallWall Loads (loads provided by cannon design)

$$\text{Brick} + 8" \text{ CMU} = 95 \text{ psf} \times 16' (\text{typical floor-to-floor height}) = 1520 \text{ pif}$$

$$2" \text{ rigid insulation} = 3 \text{ psf} \times 16' = 48 \text{ pif}$$

$$\text{Water proofing Allowance} = 1 \text{ psf} \times 16' = 16 \text{ psf}$$

$$\boxed{\text{Total} = 1584 \text{ pif}}$$



Snow LoadsLower Roof - Flat

$$P_f = 0.7 C_e C_t I P_g$$

$$C_e = 1.0 \quad (\text{Partially Exposed Roof, Exposure B})$$

$$C_t = 1.0$$

$$I = 1.1 \quad (\text{Occupancy Category 3})$$

$$P_g = 30 \text{ psf}$$

$$P_f = 0.7(1.0)(1.0)(1.1)(30) = 23.1 \text{ psf}$$

Upper Roof - Sloped

$$P_s = C_s P_f$$

$$C_e = 1.0 \quad (\text{contains large mechanical equipment})$$

$$C_t = 1.1$$

$$I = 1.1$$

$$P_g = 30$$

$$P_f = 0.7(1.0)(1.1)(1.1)(30) = 25.4 \text{ psf}$$

$$C_s = 1.0 \quad (\text{cold roof, roof surface obstructed})$$

$$P_s = 1.0(25.4) = 25.4 \text{ psf}$$

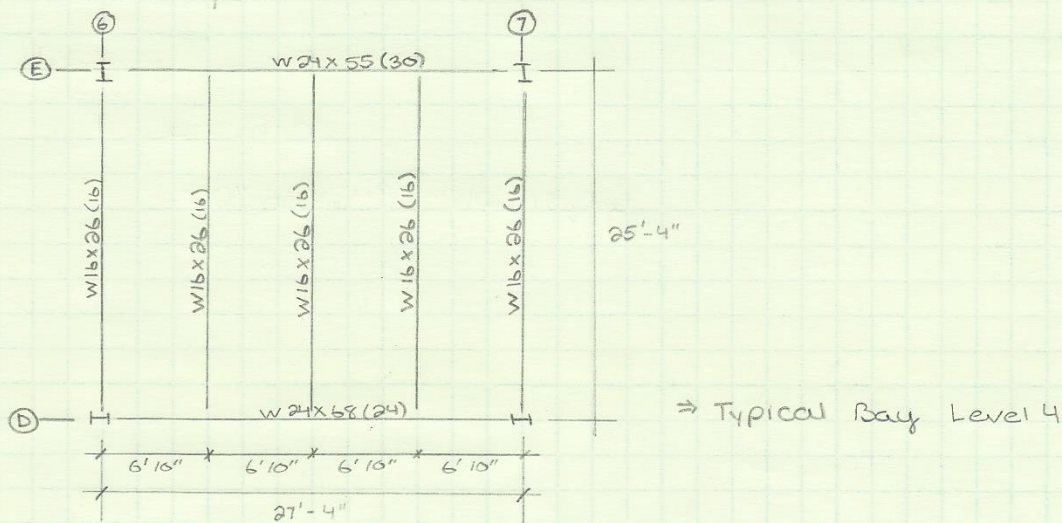
⇒ Design was conservative and used a design snow load of 26 psf for both the lower flat roof and upper sloped roof.

**Gravity Spot Checks for Existing System:  
Composite Steel System**



Gravity Check - Existing Structural Design

- Existing structural system is a composite beam + deck system



Decking - checked using Vulcraft Catalog

- 2" deck, 4 1/2" NMC, 18 gage

1. Check deck span

⇒ SDI Max Unshored Clear Span for 3<sup>+</sup> spans = 10'11"

$$\underline{10'11''} > \underline{6'10''} \quad \checkmark \quad * \text{Deck also passes for 1 + 2 span conditions}$$

2. Check  $W_u + W_{misc PL} \leq$  Superimposed Live Load

- 50psf + 27psf = 77psf (office + partitions)
- 80psf (corridor above first floor)

$$\Rightarrow W_{LL} = 80\text{psf}$$

- 10psf + 5psf = 15psf (mechanical + ceiling)

$$\Rightarrow W_{OL} = 15\text{psf}$$

- Allowable Super Imposed Live Load

$$\Rightarrow 80 + 15 = \underline{95\text{psf}} < \underline{400\text{psf}} \Rightarrow \text{Decking OK}$$



Framing - Beams

Span = 25' 4"

Spacing = 6' 10"

W16 x 26

LL = 80 psf

DL = 69 psf + 2 psf + 25 psf + 3.8 psf = 99.8 psf  $\Rightarrow$  100 psf  
(Deck/Topping + flooring + misc DL + Framing)

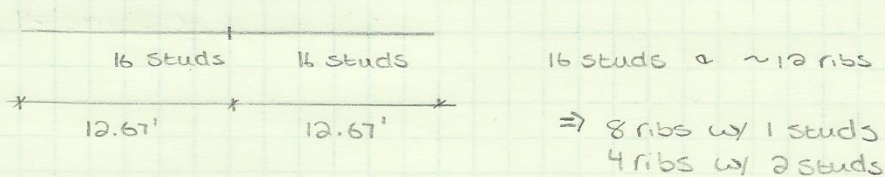
$$W_u = 1.2D + 1.6L$$

$$= [1.2(100) + 1.6(80)] \times 6.833 \text{ ft}$$

$$= 1695 \text{ plf}$$

$\Rightarrow 1.695 \text{ Klf}$

$$M_u = \frac{wL^2}{8} = \frac{1.695(25.33)^2}{8} = 135.9 \text{ k}$$

Strength

Qn:

Deck Perpendicular, weak studs (conservative), 3/4"  $\phi$ , f'c = 3 ksi

1 stud/rib  $\Rightarrow$  Qn = 17.2

2 studs/rib  $\Rightarrow$  Qn = 14.6

$\Sigma Q_n = 8(17.2) + 8(14.6) = 254.4 \text{ k}$

Other properties

$$d_{\text{eff}} = \begin{cases} 25.33(12)/8 = 38" \Rightarrow \times 2 = 76" \\ \text{min} \quad 1/2(6.833)(12) = 41" \end{cases}$$

$$V_{s, \text{max}} = 7.68(50) = 384 \text{ k} > \Sigma Q_n = 254.4 \text{ k}$$

$$V_{c, \text{max}} = 0.85(3)(76)(4.5) = 872 \text{ k}$$

$$a = \frac{254.4 \text{ k}}{0.85(3)(76)} = 1.31" \Rightarrow \gamma_2 = 5.85 \text{ in}$$

From Table 3-19 (being conservative)  $\phi M_n = 304 \text{ k}$ 

$$\underline{304 \text{ k} > 135.9 \text{ k} \Rightarrow \text{Beam passes for strength}}$$



Framing Cont. - Beams

General Structural Notes specify Shoring so unshored strength doesn't need checked

Check Wet Concrete Deflection

$$W_{wc} = 69(6.833) + 26 = 497.5 \text{ plf} \Rightarrow 0.4975 \text{ klf}$$

$$\Delta W_c = \frac{5wl^4}{384EI} = \frac{5(0.4975 \text{ klf})(25.33 \text{ ft})^4(1728)}{384(29000 \text{ ksi})(301 \text{ in}^4)} = 0.528 \text{ in}$$

$$\Delta W_{c, \text{max}} = \frac{25.33(12)}{240} = 1.267 \text{ in}$$

$$\Rightarrow \underline{1.267'' > 0.528''} \Rightarrow \underline{\text{Wet concrete deflection OK}}$$

Check Live Load Deflection

$$W_{LL} = 80 \text{ psf}(6.833) = 546.6 \text{ plf} \Rightarrow 0.5466 \text{ klf}$$

$$I_{LB} = 830 \text{ in}^4 \text{ (conservative choice)}$$

$$\Delta_{LL} = \frac{5(0.5466)(25.33)^4(1728)}{384(29000)(830)} = 0.210 \text{ in}$$

$$\Delta_{LL, \text{max}} = L/360 = \frac{25.33(12)}{360} = 0.844 \text{ in}$$

$$\Rightarrow \underline{0.844'' > 0.210''} \Rightarrow \underline{\text{Live Load Deflection OK}}$$

Summary

$\Rightarrow$  Typical W16x26 w/ 32 studs per beam passes for strength and deflections

$$\text{Equivalent Beam Weight} = 26(25.33) + 32(10) = 979 \#$$



# Spot Check - Girder | Tech Report 3 | Macenzie Ceglar

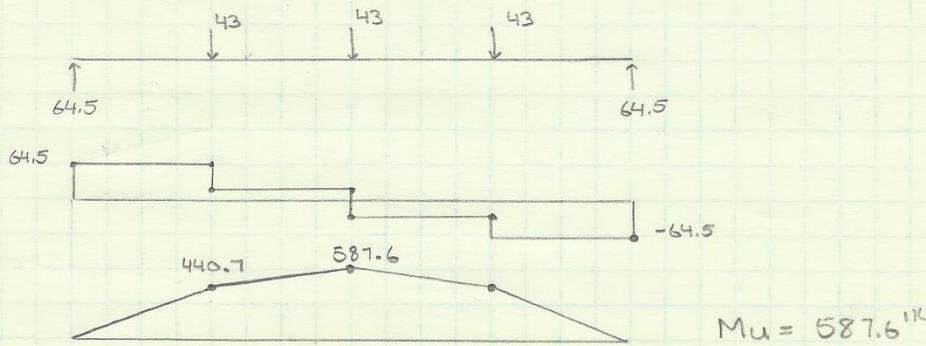
## Framing - Girders

Span = 27' 4"  
 Spacing = 25' 4"  
 W24x68

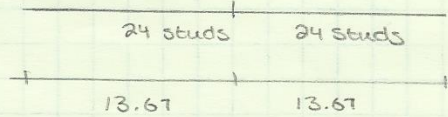
### Point Loads from beams

$$W_{u, \text{beam}} = 1.695 \text{ KIF}$$

$$P = \frac{1.695 \times (25.33)}{2} = 21.5^{\text{K}} \times 2 \text{ beams} = 43^{\text{K}}$$



### Strength



24 studs  $\sim$  13 ribs  
 $\Rightarrow$  Stud spacing = 6.83"

\* Sheet S-502 specifies studs to be placed in a single row unless spacing is less than 4 1/2"

$Q_n$ :

Deck parallel,  $w_f/n_r = 5/2 = 2.5 > 1.5$

$$Q_n = 21.0^{\text{K}}$$

$$\sum Q_n = 24(21) = 504^{\text{K}}$$

### Other properties

$$b_{\text{eff}} = \begin{cases} 28.33(12)/8 = 42.5" \times 2 = 85" \\ \min \quad 1/2(25.33)(12) = 152" \end{cases}$$

$$V_s, \text{max} = 20.1(50) = 1005 > \sum Q_n = 504^{\text{K}}$$

$$V_c, \text{max} = 0.85(3)(85)(4.5) = 975$$

$$a = 504 / (0.85 \times 3 \times 85) = 2.33" \Rightarrow \gamma_z = 5.34 \text{ in}$$

From Table 3-19 (being conservative)  $\Rightarrow \phi M_n = 1060^{\text{K}}$   
 $1060^{\text{K}} > 587.6^{\text{K}} \Rightarrow$  Girder passes for strength



Framing Cont. - Girder

Shoring Provided so unshored strength not checked

Check web concrete deflection

$$P_{wc} = 0.4975 \text{ Klf} \times (25.33) = 12.6 \text{ K}$$

$$W_{sw} = 68 \text{ plf} = 0.068 \text{ Klf}$$

Eqn from pg 2-210

$$\Delta_{wc} = \frac{0.050 (12.6)(27.33)^3 (1728)}{29000 (1830 \text{ in}^4)} + \frac{5(0.068)(27.33)^4 (1728)}{384 (29000)(1830)}$$

$$= 0.419 + 0.0161 = 0.435$$

$$\Delta_{wc, \text{max}} = \frac{27.33 (12)}{240} = 1.367 \text{ in}$$

$$\Rightarrow \underline{1.367" > 0.435" \Rightarrow \text{web concrete deflection OK}}$$

Check Live Load Deflections

$$P_{LL} = 0.5466 \text{ Klf} \times (25.33) = 13.8 \text{ K}$$

$$I_{LB} = 3670 \text{ in}^4$$

$$\Delta_{LL} = \frac{0.050 (13.8)(27.33)^3 (1728)}{29000 (3670)} = 0.229 \text{ in}$$

$$\Delta_{LL, \text{max}} = \frac{27.33 (12)}{360} = 0.911$$

$$\Rightarrow \underline{0.911 > 0.229 \Rightarrow \text{Live Load Deflection OK}}$$

Summary

⇒ Typical w24x68 w/ 48 studs per beam  
passes for strength + deflections

$$\text{Equivalent Girder Weight} = 68(27.33) + 48(10) = 2338 \text{ lbs}$$



Framing - columnsInterior column D6

- W12 x 170

- supports levels 2-6 + lower roof

$$\text{Floor DL} = 69 + 2 + 5 + 10 + 10 + 6 = 102 \text{ psf}$$

↑ Framing Allowance

$$\text{Floor LL} = 80 \text{ psf}$$

$$\text{Roof DL} = 117 \text{ psf}$$

$$\text{Roof LL} = 150 \text{ psf}$$

$$\text{Roof Snow Load} = 26 \text{ psf}$$

using worst case load combination  $1.2D + 1.6L_r + L_s$

$$\text{trib area} = \left[ \frac{31}{2} + \frac{27.33}{2} \right] \times [25.33] = 739 \text{ ft}^2$$

$$P_u = [1.2(117) + 1.6(150) + 1.2(102)(5) + 80(5)] \times 739 \times \frac{1}{1000}$$

$$= 1029 \text{ k}$$

$$\Rightarrow + \text{Self weight of columns} = 170(36) + 120(32) + 65(34) = 12170.155$$

$$P_u = 1029 + 12170/1000 = 1041.2 \text{ k}$$

From table 4-1

$$\text{Effective length} = 18'$$

$$\phi P_n = 1620 \text{ k}$$

$$\Rightarrow \underline{1620 \text{ k} > 1041.2 \text{ k}} \Rightarrow \underline{\text{column passes for strength}}$$

\* Note - this increased load capacity may be due to larger floor loads in some locations on upper floors due to general collections



Exterior Column - E6

- W12 x 65
- supports level 4-6 + lower roof

Floor DL = 102 psf  
 Floor LL = 80 psf  
 Roof DL = 117 psf  
 Roof LL = 150 psf  
 Roof Snow Load = 26 psf

using worst case load combination  $1.2D + 1.6Lr + L$ :

$$\text{trib area} = [3/2 + 27.33/2] \times [25.33/2 + 2] = 427.7 \text{ ft}^2$$

$$\begin{aligned} P_u &= [1.2(117 \text{ psf}) + 1.6(150) + 1.2(102)(3) + 80(3)] \times 427.7 / 1000 \\ &\quad + [1.2(1584)(27.33)] / 1000 \\ &= 474.3 \text{ k} \end{aligned}$$

$$\Rightarrow + \text{ Self weight of columns} = 65(32) + 53(24) = 3882 \text{ lbs}$$

$$P_u = 474.3 + 3882 / 1000 = 478.2 \text{ k}$$

From Table 4-1

$$\phi P_n = 640 \text{ k}$$

$$\Rightarrow \underline{640 \text{ k} > 478.2 \text{ k}} \Rightarrow \underline{\text{Column passes for strength}}$$

# **Structural Redesign 1: Non-composite Steel System**



Structural Redesign - Non-composite steel

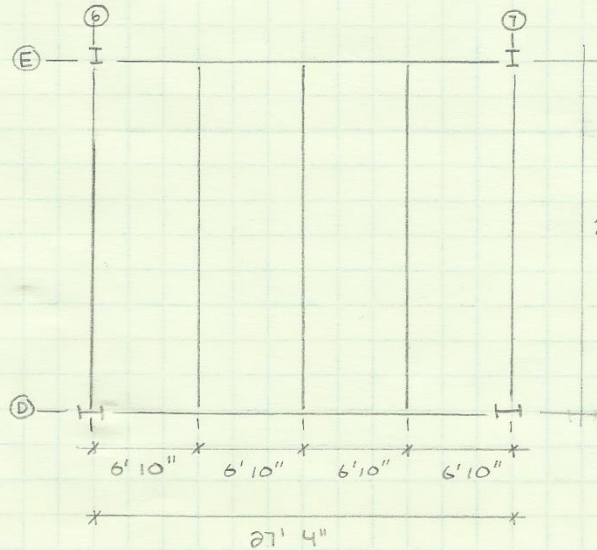
Decking

- use existing composite decking

Deck: 2" deck, 4 1/2" NWC, 18 gage  $\Rightarrow$  2vLI18 (vulcraft)

Beams

- Same layout as existing system so comparisons can be made



Span = 25' 4"  
Spacing = 6' 10"  
LL = 80 psf  
DL = 69 + 2 + 25 + 7 = 103 psf

$$w_u = 1.2(103) + 1.6(80) \times 6.833 / 1000 = 1,719 \text{ klf}$$

$$M_u = \frac{(1,719)(25.33)^2}{8} = 137.9 \text{ k}$$

Strength

$\Rightarrow$  Try W14 x 26  $\phi M_n = 151 \text{ k} > 137.9 \text{ k}$

Check Deflection

$$\Delta_{LL} = \frac{5 \left( \frac{80 \times 6.833}{1000} \right) (25.33)^4 (1728)}{384 (29000) (245)} = 0.713 \text{ ''}$$

$$L/360 = \frac{25.33(12)}{360} = 0.844 \text{ ''}$$

$\Rightarrow 0.844 \text{ ''} > 0.713 \text{ ''} \Rightarrow$  Deflection Passes

Check Allowance

$$\frac{26 \text{ plf}}{6.833 \text{ ft}} = 3.81 < 7 \text{ psf} \checkmark$$

$\Rightarrow$  Use W14 x 26 beams spaced at 6' 10"



Girders

Interior Girder

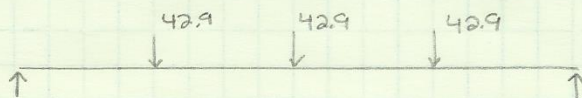
Span = 27'4"  
Spacing = 25'4"

Point loads from beams

$$w_{u, \text{beam}} = 1.2(69 + 2 + 25 + 4(\text{sw beam})) + 1.6(80) \times 6.833/1000 = 1.695 \text{ klf}$$

$$P_u = \frac{1.695(25.33)}{2} \times 2 \text{ point loads} = 42.9 \text{ k}$$

$$S_{WG} = 5(25.33) = 127 \text{ plf} = 0.127 \text{ klf}$$



$$M_u = (0.5)(42.9)(27.33) + 0.127 \frac{(27.33)^2}{2} = 598.1 \text{ k}$$

Strength

$$\Rightarrow \text{Try } w21 \times 68 \quad \phi M_n = 600 \text{ k} > 598.1 \text{ k}$$

Check Deflections

$$\Delta_{LL} = \frac{0.05(13.9 \text{ k})(27.33)^3}{(20000)(1480)} (1728) = 0.571$$

$$w_{LL} = \frac{80(6.833)}{1000} = 0.547$$

$$P_{LL} = 0.547(25.33) = 13.9 \text{ k}$$

$$L/360 = \frac{27.33(12)}{360} = 0.911$$

$\Rightarrow 0.911 > 0.571 \Rightarrow$  Girder passes deflections

Check Girder Allowance

$$\frac{68}{25.33} = 2.68 < 5$$

$\Rightarrow$  Use a w21 x 68 girder



Girders cont.

Exterior Girder

Span = 27' 4"

Interior Spacing = 25' 4"

Point Loads from beams

$w_u = 1.695 \text{ klf}$

$P_u = \frac{1.695(25.33)}{2} = 21.5^k$

Distributed Wall Loads

$w_u = 1.2(1584) = 1900.8 \Rightarrow 1.9 \text{ klf}$

Determine  $M_u$

$M_u = 0.5(21.5)(27.33) + (1.9 + 0.127) \frac{(27.33)^2}{8} = 483.1^k$   
↑  
self weight

Strength

$\Rightarrow \text{Try } W24 \times 55 \quad \phi M_n = 503^k > 483.1^k$

Check Deflections

$P_{LL} = 13.9^k / 2 = 6.95^k$

$\Delta_{LL} = \frac{0.05(6.95)(27.33)^3(1728)}{29000(1350)} = 0.313$

$L/360 = 0.911$

$\Rightarrow 0.911 > 0.313 \Rightarrow \text{Girder passes deflections}$

Check Allowance

$\frac{55}{25.33} = 2.17 < 5$

$\Rightarrow \underline{\underline{\text{use a } W24 \times 55}}$



Comparison of Composite vs Non-composite System

Decking: same for both systems

Beams:

<u>Composite</u>	<u>Non-composite</u>
W16 x 26 w/ 32 studs	W14 x 26
Equivalent weight = 979 lbs	weight = $26(25.33) = 659$ lbs

Girders:

	<u>Composite</u>	<u>Non-Composite</u>
Interior	W24 x 68 w/ 48 studs Equivalent weight = 2338 lbs	W21 x 68 weight = $90(27.33) = 2460$ lbs
Exterior	W24 x 55 w/ 30 studs Equivalent weight = 1803 lbs	W24 x 55 weight = 1503 lbs

⇒ From the comparison of the beams it seems as if the Non-composite beam is a more economical choice. Something not considered in this analysis/design is the load due to the general collections (150 psf) located in other bays. Please see next page for a more detailed look at this factor.

A similar situation may occur w/ the girders



## Composite vs Non-Composite Beams

⇒ Considering general collections

### Existing W16x26

$$w_u = [1.2(100) + 1.6(150)] \times 6.833 / 1000 = 2.46 \text{ klf}$$

$$M_u = \frac{(2.46)(25.33)^2}{8} = 197.3 \text{ k}$$

As calculated before  $\phi M_n = 304 \text{ k} > 197.3 \text{ k}$

### Check deflections

$$w_{LL} = 150(6.833) = 1025 \text{ plf} \Rightarrow 1.025 \text{ klf}$$

$$I_{LB} = 830 \text{ in}^4$$

$$\Delta_{LL} = \frac{5(1.025)(25.33)^4(1728)}{384(29000)(830)} = 0.394 \text{ in}^4$$

$$\Delta_{LL, \text{max}} = 0.844$$

$$\Rightarrow 0.844 > 0.394$$

⇒ Existing design can carry increased load

### Redesign W14x26

$$w_u = [1.2(103) + 1.6(150)] \times 6.833 / 1000 = 2.484 \text{ klf}$$

$$M_u = \frac{(2.484)(25.33)^2}{8} = 199.2 \text{ k}$$

$$\phi M_n = 151 \text{ k} < 199.2 \text{ k}$$

⇒ Redesign not capable of carrying increase

\* Try W18x35  $\phi M_n = 249 \text{ k} > 199.2 \text{ k}$

### Check Deflections

$$\Delta_{LL} = \frac{5 \left( \frac{150 \times 6.833}{1000} \right) (25.33)^4 (1728)}{(384)(29000)(510)} = 0.642 \text{ in}$$

$$L/360 = 0.844 \text{ in}$$

$$\Rightarrow 0.844 \text{ in} > 0.642 \text{ in}$$



Final Comparison of W16x26 w/ 32 studs vs W18x35

W16x26 w/ 32 studs  $\Rightarrow 979^{lb}$

W18x35  $\Rightarrow 887^{lb}$

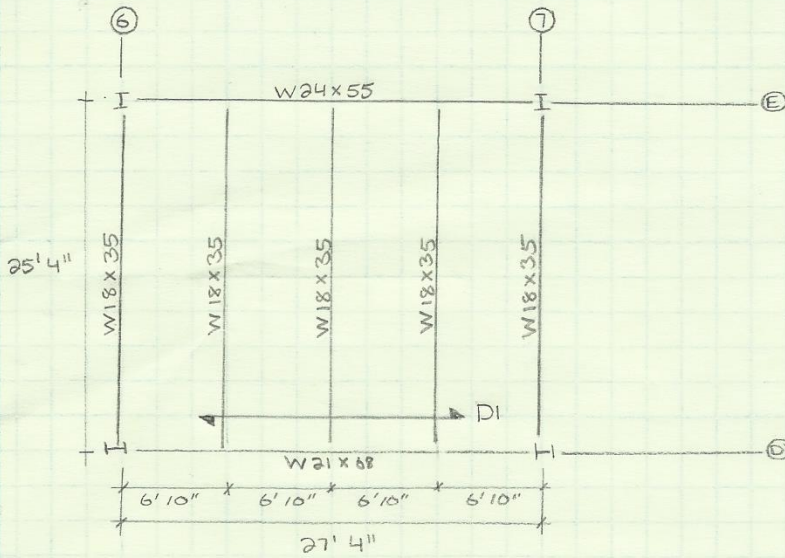
Summary

Once the 150psf load is considered it can be seen that the non-composite beam size increases in depth and weight but the composite beam remains the same.

Although the W18x35 has a lower equivalent weight, which may be used for cost concerns, it does require increased space for depth and increased weight per foot on girder.

One issue not addressed in this report is the impact of vibrations on the system. Vibrations on the composite system typically are much lower than w/ non-composite. If vibrations increase the size of the non-composite beams the decision to use a composite system may be clearer.

Non-composite Design Summary



Notes: D<sub>1</sub> - 6 1/2" composite deck, 2" deck, 4 1/2" NWC, 18 gage (2VLI 18)

Infill beams shown are those calculated using LL = 150 psf

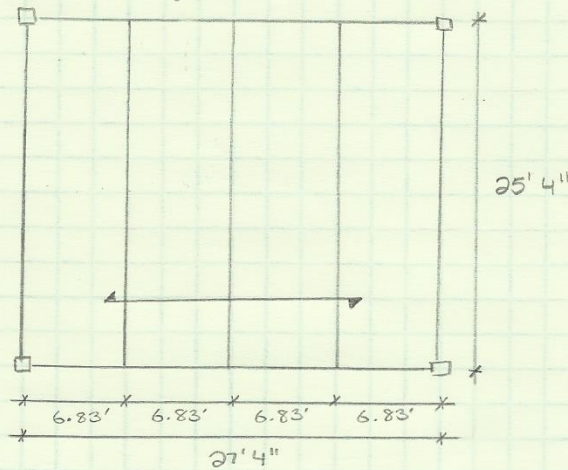


## **Structural Redesign 2: One-Way Slab with Beams**

## Structural Redesign - One Way Slab w/ Beams

### Bay Layout

- Keeping same bay size as existing



### Slab Design

#### Determine Thickness

⇒ Table 9.5(a) cannot be used for preliminary sizing due to the structure supporting partitions and large sustained loads causing increased deflections

⇒ Table A-9 from Reinforced Concrete Mechanics and Design 6<sup>th</sup> Edition by MacGregor and Wight was used to determine a starting thickness because it accounts for partitions + loads

⇒ For members supporting partitions + a sustained load/total load > 0.5 both ends continuous:

$$h_{min} = l/10 = \frac{6.833(12)}{10} = 8.2$$

⇒ Use 8.5" Slab

#### Factored Loads

$$LL = 150 \text{ psf}$$

$$SW = (8.5/12)(150) = 106.3 \text{ psf}$$

$$DL = 106.3 + 25 = 131.3 \text{ psf}$$

$$W_u = 1.2(131.3) + 1.6(150) = 398 \text{ psf} \Rightarrow 398 \text{ pif (Per 1ft strip)}$$



Design Moments

$$M_u^- = \frac{w_u l^2}{11} = \frac{398(6.83)^2}{11} \cdot \frac{1}{1000} = 1.69 \text{ k}$$

$$M_u^+ = \frac{w_u l^2}{16} = \frac{398(6.83)^2}{16} \cdot \frac{1}{1000} = 1.16 \text{ k}$$

⇒ using moment coefficients to estimate  $+$  &  $-$  to be conservative

Estimate Steel Reinforcement - Negative

$$- f'_c = 4000 \text{ psi}$$

$$- \text{Assume } d = 7.5" \text{ (\#4 w/ } 3/4" \text{ cover)}$$

$$A_s^- = \frac{M_u^-}{4d} = \frac{1.69 \text{ k}}{4(7.5")} = 0.056 \text{ in}^2$$

Estimate Steel Reinforcement - Positive

$$A_s^+ = \frac{M_u^+}{4d} = \frac{1.16}{4(7.5)} = 0.0387 \text{ in}^2$$

Check Require  $A_s$  for Temp + Shrinkage

$$\begin{aligned} A_{s, \min} &= 0.0018 bh \\ &= 0.0018(12)(8.5) \\ &= 0.1836 \text{ in}^2 \leftarrow \text{controls both neg + pos} \\ &\quad \text{reinforcement.} \end{aligned}$$

Determine Max Spacing for Crack Control

$$\begin{aligned} s &\leq 15 \left( \frac{40,000}{f_s} \right) - 2.5 C_c \leq 12 (40,000 / f_s) \\ &\leq 15 \left( \frac{40,000}{\frac{2}{3}(60,000)} \right) - 2.5(7.5) \leq 12 (40,000 / \frac{2}{3}(60,000)) \\ &\leq 13.1 \leq 12 \end{aligned}$$

⇒ Spacing must be  $\leq 12"$

⇒ Provide #4 @ 12" O.C. for both positive and negative reinforcement

$$\underline{\underline{0.2 \text{ in}^2 > 0.1836 \text{ in}^2}}$$



Check Flexural Strength

Negative + Positive Moments

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

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{0.2(60)}{0.85(4)(12)} = 0.294$$

$$= 0.9(0.2)(60)(7.5 - 0.294/2) / 12$$

$$= 6.62 \text{ k}$$

$\Rightarrow 6.62 \text{ k} > 1.69 \text{ k}$  Slab OK for flexural strength

Check One-Way Shear

$$q_u = 398 \text{ psf} \Rightarrow 0.398 \text{ ksf}$$

$$V_u = 0.398(25.33 \times 74.5 / 12) = 62.6 \text{ k}$$

$$V_c = 2\sqrt{f_c} b_w d$$

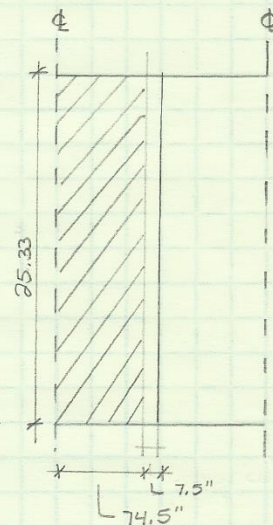
$$= 2(1.0)\sqrt{4000}(25.33 \times 12)(7.5)$$

$$= 288.4 \text{ k}$$

$$\phi V_c = 0.75(288.4) = 216.3 \text{ k}$$

$$\Rightarrow 216.3 \text{ k} > 62.6 \text{ k}$$

Slab OK for one-way shear



Slab Design Summary

Slab Thickness	E-W Direction		N-S Direction
	TOP	BOTTOM	# 4 @ 12"
8.5"	# 4 @ 12"	# 4 @ 12"	For Temperature + Shrinkage

Notes:

$$f_c = 4000 \text{ psi NWC}$$



Beam DesignDesign Loads

$$w_u = [1.2(131.3) + 1.6(150)](6.83) = 2718 \text{ pif}$$

Design Moments

$$M_u = \frac{w_u l^2}{8} = \frac{(2.72)(25.33)^2}{8} \times 1.1 = 240 \text{ k}^{\text{K}}$$

↑ Estimate of SW

Estimate size

$$bd^2 = 20M_u$$

$$\text{Try } b = \frac{2}{3}d \Rightarrow \frac{2}{3}d^3 = 20(240)$$

$$d = 19.3''$$

$$h = d + 2.5 = 21.8 \Rightarrow 22'' \Rightarrow h = 22'' + b = 13''$$

(d = 19.5)

Compute self weight Effect

$$w_{sw} = \frac{13(22)}{144} \times 150 = 298 \text{ pif}$$

$$w_u = 2718 + 1.2(298) = 3075.6 \text{ pif}$$

$$M_u = \frac{3.076(25.33)^2}{8} = 246.7 \text{ k}^{\text{K}}$$

Required Steel

$$A_s = \frac{M_u}{4d} = \frac{246.7}{4(19.5)} = 3.16 \Rightarrow (4)\#8 = 4(79) = 3.16 \text{ in}^2$$

Check Flexural Strength

$$a = \frac{(3.16)(60)}{0.85(4)(13)} = 4.29 \quad c = a/\beta_1 = 4.29/0.85 = 5.05$$

$$M_n = 3.16(60)(19.5 - \frac{4.29}{2}) / 1.2 = 274.2 \text{ k}^{\text{K}}$$

$$\epsilon_s = \frac{\epsilon_y}{c}(d-c) = \frac{0.003}{5.05}(19.5 - 5.05) = 0.00858 > 0.00207 \checkmark$$

$$\phi M_n = 0.9(274.2) = 246.8 \text{ k}^{\text{K}} > 246.7 \text{ k}^{\text{K}}$$

⇒ Beam OK for Flexure



Check Minimum Reinforcement

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} bwd \geq \frac{200}{f_y} bwd$$

$$= \frac{3\sqrt{4000}}{60,000} (13)(19.5) \geq \frac{200(13)(19.5)}{60,000}$$

$$= 0.802 \geq 0.845$$

$$\Rightarrow \text{Provided } A_s = 3.16 \text{ in}^2 > 0.802 \text{ in}^2 \checkmark$$

Check Maximum Reinforcement

$$A_{s,max} = 0.85\beta_1 \frac{f'_c}{f_y} \left( \frac{\epsilon_u}{\epsilon_u + \epsilon_y} \right) bwd$$

$$= \frac{(0.85)^2 (4)}{60} \left( \frac{0.003}{0.003 + 0.004} \right) (13)(19.5)$$

$$= 5.24 \text{ in}^2$$

$$\Rightarrow \text{Provided } A_s = 3.16 \text{ in}^2 < 5.24 \text{ in}^2 \checkmark$$



Check Minimum Spacing

$$\begin{array}{l} \text{Min clear distance} \\ \text{between bars} \end{array} = \begin{array}{l} d_b \\ 1'' \end{array} \Rightarrow 1'' \Rightarrow 4/3'' \\ \text{max } 4/3 S_a \quad 4/3(1) = 4/3'' \quad \uparrow 1'' \text{ agg}$$

Actual Spacing

$$S = \frac{13 - 1.5(2) - 0.5(2) - 4}{3} = 1.67 > 1.33 \checkmark$$

Check Min Number of Bars:

⇒ Using Table A.8 adapted from Ref. 3.8 used by permission of American Concrete Institute

$$\Rightarrow 1.5'' \text{ clear cover} + \#4 \text{ stirrup} \Rightarrow 2 < 4 \checkmark$$

Check Shear Strength

$$V_u = (3.076 \times 25.33/2 - 11.5/2) = 34^k$$

$$V_c = 2 \sqrt{f'_c} b_w d = 2(1.0) \sqrt{4000} (13)(19.5) \Rightarrow 32.1^k$$

$$0.5 \phi V_c = 12.04^k < 34^k \Rightarrow \text{Shear Stirrups Needed}$$

Shear strength required by reinforcement

$$\begin{aligned} V_s &= V_u / \phi - V_c < 8 \sqrt{f'_c} b_w d \\ &= \frac{34}{0.75} - 32.1 < 8 \sqrt{4000} (13)(19.5) \\ &= 13.2^k < 128.3^k \checkmark \end{aligned}$$

Max Spacing of Shear Reinforcement

$$V_s \leq 4 \sqrt{f'_c} b_w d = 4 \sqrt{4000} (13)(19.5) / 1000 = 64.1$$

$$\Rightarrow S_{\max} = \min \left\{ \begin{array}{l} d/2 = 19.5/2 = 9.75'' \Rightarrow \underline{9''} \\ 24'' \end{array} \right.$$

Min Shear Reinforcing

$$A_v = \max \left\{ \begin{array}{l} 0.75 \sqrt{4000} (13)(9) / 60,000 = 0.0925 \Rightarrow \underline{0.0975} \text{ in}^2 \\ 50 (13)(9) / 60,000 = 0.0975 \end{array} \right.$$

⇒ use 2 legs of #3 stirrups @ 9"

$$0.11 \times 2 = 0.22 > 0.0975 \checkmark$$



Design Shear Reinforcement

$$S = \frac{A_v f_y d}{V_s} = \frac{0.22(60)(19.5)}{13.2} = 19.5" > S_{max}$$

⇒ Space @ 9"

Stirrup Layout

Terminate stirrups at  $V_u \leq 0.5\phi V_c = 12^k$

$$12 = \frac{3.076(25.33)}{2} - 3.076(d)$$

$$d = 8.76'$$

Number of stirrups:

$$2" + (n-1)(9") \geq 8.76(12)$$

$$n = 12.4 \Rightarrow 13 \text{ stirrups}$$

⇒ (13) # 3 x 11 @ 9" starting 2" from each support

BEAM SUMMARY

13" x 22" NWC, 4000 psi
(4) # 8 longitudinal bars (bottom)
(13) # 3 x 11 @ 9" starting 2" from each support



Girder Design - interiorDesign Loads

$$W_u = 3.076 \text{ KIF}$$

$$P_u = \frac{3.076 \text{ KIF} (27.33')}{2} \times 2 \text{ beams on point} = 78.9 \text{ K}$$

Design Moments

$$M_u = 0.5(78.9)(27.33) = 1066 \text{ K} \times 1.1 = 1173 \text{ K}$$

Estimate Size

$$bd^2 = 20 M_u$$

$$\text{Try } b = \frac{2}{3} d \Rightarrow \frac{2}{3} d^3 = 20(1173)$$

$$d = 32.7''$$

$$h = d + 2.5 = 35.2 \Rightarrow h = 36'' \quad b = 21''$$

$$(d = 33.5)$$

Compute self weight effect

$$W_{sw} = \frac{21(36)}{144} \times 150 = 787.5 \text{ plf} \Rightarrow 0.7875 \text{ K}$$

$$M_u = 1173 \text{ K} + \frac{(0.7875)(27.33)^2}{8} = 1246 \text{ K}$$

Required Steel

$$A_s = \frac{M_u}{4d} = \frac{1247}{4(33.5)} = 9.3 \text{ in}^2 \Rightarrow (6) \#11 \Rightarrow 6(1.56) = 9.36 \text{ in}^2$$

Check Flexural Strength

$$a = \frac{(9.36)(60)}{0.85(4)(21)} = 7.866 \quad C = 7.87/0.85 = 9.26$$

$$M_n = 9.36(60)(33.5 - \frac{7.87}{2})/12 = 1384 \text{ K}$$

$$\epsilon_s = \frac{0.003(33.5 - 9.26)}{9.26} = 0.0079 > 0.00207 \checkmark$$

$$\phi M_n = 0.9(1384) = 1246 \text{ K} = 1246 \text{ K} \checkmark$$

$\Rightarrow$  Girder OK for flexure



Check Minimum Reinforcement

$$\begin{aligned} A_{s, \min} &= \frac{3\sqrt{f'_c}}{f_y} bwd \geq 200 \frac{bwd}{f_y} \\ &= \frac{3\sqrt{4000}}{60,000} (21)(33.5) \geq \frac{200(21)(33.5)}{60,000} \\ &= 2.2 \text{ in}^2 \geq 2.35 \text{ in}^2 \end{aligned}$$

$$\Rightarrow \text{Provided Steel} = 9.36 \text{ in}^2 < 2.35 \text{ in}^2 \checkmark$$

Check Maximum Reinforcement

$$\begin{aligned} A_{s, \max} &= 0.85 \beta_1 \frac{f'_c}{f_y} \left( \frac{0.003}{0.003 + 0.004} \right) bwd \\ &= \frac{0.85^2 (4)}{60} \left( \frac{0.003}{0.003 + 0.004} \right) (21)(33.5) \\ &= 17.1 \text{ in}^2 \end{aligned}$$

$$\Rightarrow \text{Provided Steel} = 9.36 \text{ in}^2 < 17.1 \text{ in}^2 \checkmark$$



Check Minimum Spacing

$$\text{Min Clear Distance between bars} = \begin{cases} d_b \\ 1'' \\ \max \end{cases} \begin{cases} 1.41'' \\ 1'' \\ 4/3 S_a \\ 4/3(6) = 1.3'' \end{cases} \Rightarrow 1.41''$$

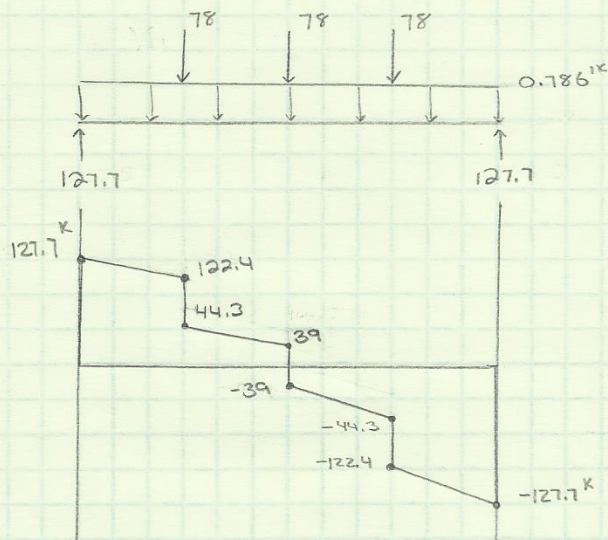
Actual Spacing

$$S = \frac{21 - 1.5(2) - 0.5(2) - 1.41(6)}{5} = 1.7 > 1.4$$

Check Min Number of Bars

$$\Rightarrow 1.5'' \text{ clear cover} + \#4 \text{ stirrups} \Rightarrow 3 < 6 \checkmark$$

Check Shear Strength



$$V_u @ d = 127.7 - 0.786(33.5)/2 = 125.5^k$$

$$V_c = 2(1)\sqrt{4000}(21)(33.5)/1000 = 89^k$$

$$0.5\phi V_c = 0.5(0.75)(89) = 33.4^k < 125.5^k \Rightarrow \text{stirrups needed}$$

Shear strength required by reinforcement

$$\begin{aligned} V_s &= V_u/\phi - V_c < 8\sqrt{F_c} b_w d \\ &= \frac{125.5}{0.75} - 89 < 8\sqrt{4000}(21)(33.5)/1000 \\ &= 78.3^k < 356^k \checkmark \end{aligned}$$



Max Spacing of Shear Reinforcing

$$V_s \leq 4\sqrt{f'_c} b_w d = 4\sqrt{4000} (21)(33.5) / 1000 = 178^k$$

$$\Rightarrow S_{\max} = \min \begin{cases} d/2 = 33.5/2 = 16.75 \\ 24 \end{cases} \Rightarrow 16.75 \Rightarrow \underline{\underline{16''}}$$

Min Shear Reinforcing

$$A_v = \max \begin{cases} 0.75\sqrt{4000} (21)(16) / 60,000 = 0.266 \\ 50(21)(16) / 60,000 = 0.28 \end{cases} = \underline{\underline{0.28 \text{ in}^2}}$$

$\Rightarrow$  Use 2 legs of #4 stirrups @ 16"

$$0.2 \times 2 = 0.4 > 0.28 \checkmark$$

Design Shear Reinforcement

$$S = \frac{A_v f_y d}{V_s} = \frac{0.4 (60)(33.5)}{78.3} = 10.3''$$

$\Rightarrow$  Space @ 10"

Stirrup Layout

$\Rightarrow$  Terminate stirrups at  $V_u \leq 33.4^k$

$\Rightarrow$   $V_u$  never  $< 33.4^k$  so stirrups are continuous

$$2''(2) + (n-1)(10) = 27.33(12)$$

$$n = 33.4 \Rightarrow 35 \text{ stirrups}$$

$\Rightarrow$  (35) #4 x M1 evenly spaced starting 2" from each support

GIRDER SUMMARY  $\Rightarrow$  INTERIOR

21" x 36" NWC, 4000psi  
 (6) #11 longitudinal bars (bottom)  
 (35) #4 x M1 evenly spaced starting 2" from each support



Girder Design - ExteriorDesign Loads

$$w_u = 3.076 \text{ Klf}$$

$$w_u(\text{wall}) = 1.9 \text{ Klf}$$

$$P_u = \frac{3.076 (25.33')}{2} = 39 \text{ K}$$

Design Moments

$$M_u = \left[ 0.5 (39 \times 27.33) + 1.9 (27.33)^2 / 8 \right] \times 1.1 = 781.4 \text{ K}$$

Estimate Size

$$bd^2 = 20 M_u$$

$$\text{Try } b = \frac{2}{3}d \Rightarrow \frac{2}{3}d^3 = 20(781.4)$$

$$d = 28.6$$

$$h = d + 2.5 = 31.1 \Rightarrow h = 32 \text{ + } b = 20''$$

(29.5 = d)

Compute Self-Weight Effect

$$w_{sw} = \frac{20(33) \times 150}{144} = 687.5_{\text{plf}} \Rightarrow 0.6875 \text{ Klf}$$

$$M_u = 781.4 + \frac{(0.6875)(27.33)^2}{8} = 845.6 \text{ K}$$

Required Steel

$$A_s = \frac{M_u}{4d} = \frac{845.6}{4(29.5)} = 7.2 \Rightarrow (s)^{\#11} = 7.8 \text{ in}^2$$

Check Flexural Strength

$$a = \frac{(7.8)(60)}{0.85(4)(29)} = 6.89 \quad c = 6.89 / 0.85 = 8.11 \quad d = 32 - 1.5 - 0.5 - \frac{1.41}{2} = 29.3$$

$$M_n = (7.8)(60)(29.3 - \frac{6.89}{2}) / 12 = 1008.3$$

$$E_s = \frac{0.003}{8.11} (29.3 - 8.11) = 0.008 > 0.00207 \checkmark$$

$$\phi M_n = 0.9 (1008.3) = 907.5 \text{ K} > 845.6 \text{ K} \checkmark$$

$\Rightarrow$  Girder OK for flexure

Check Minimum Reinforcement

$$\begin{aligned} A_{s, \min} &= \frac{3\sqrt{f'_c}}{f_y} b_w d \geq \frac{200}{f_y} b_w d \\ &= \frac{3\sqrt{4000}}{60,000} (20)(29.3) \geq \frac{200(20)(29.3)}{60,000} \\ &= 1.85 \geq 1.95 \end{aligned}$$

$$\Rightarrow \text{provided } A_s = 7.8 \text{ in}^2 > 1.95 \text{ in}^2 \checkmark$$

Check Maximum Reinforcement

$$\begin{aligned} A_{s, \max} &= 0.85 \beta_1 \frac{f'_c}{f_y} \left( \frac{0.003}{0.003 + 0.004} \right) b_w d \\ &= (0.85)^2 \frac{(4)}{60} \left( \frac{0.003}{0.003 + 0.004} \right) (20)(29.3) \\ &= 12.1 \text{ in}^2 \end{aligned}$$

$$\Rightarrow \text{provided } A_s = 7.8 \text{ in}^2 < 12.1 \text{ in}^2 \checkmark$$



Check minimum spacing

$$\text{Min clear distance between bars} = \begin{matrix} d_b \\ 1" \\ \text{Max } 4/3 S_a \end{matrix} \Rightarrow \begin{matrix} 1.410 \\ 1 \\ 1.3" \end{matrix} \Rightarrow 1.410$$

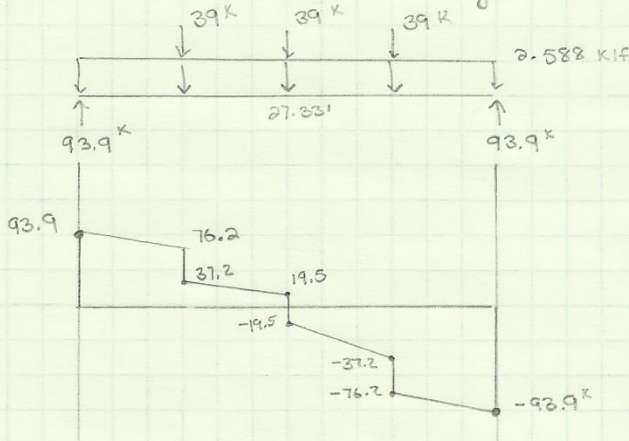
Actual spacing

$$S = \frac{21 - 1.5(2) - 0.5(2) - 5(1.410)}{4} = 2.49 > 1.410 \checkmark$$

Check min number of bars

$$\Rightarrow 1.5" \text{ clear cover} + \#4 \text{ stirrups} = 3 < 5 \checkmark$$

Check Shear Strength



$$V_u @ d = 93.9 - 2.588(29.5)/2 = 87.5 \text{ k}$$

$$V_c = 2(1)\sqrt{4000}(20)(29.5)/1000 = 74.6 \text{ k}$$

$$0.5\phi V_c = 0.5(0.75)(74.6) = 27.98 \text{ k} < 87.5 \text{ k} \Rightarrow \text{stirrups needed}$$

Shear strength required by reinforcement

$$V_s = V_u/\phi - V_c < 8\sqrt{f'_c} b w d$$

$$= \frac{87.5 \text{ k}}{0.75} - 74.6 < 8\sqrt{4000}(20)(29.5)$$

$$= 42.1 \text{ k} < 299 \checkmark$$

Max Spacing of Shear Reinforcing

$$V_s \leq 4\sqrt{f_c'} b_w d = 4\sqrt{4000} (20)(29.5)/1000 = 149.3$$

$$\Rightarrow S_{max} = \min \left\{ \begin{array}{l} d/2 = 29.5/2 = 14.75 \Rightarrow 14'' \\ 24 \end{array} \right.$$

Min Shear Reinforcing

$$A_v = \max \left\{ \begin{array}{l} 0.75\sqrt{4000} (20)(14)/60,000 = 0.221 \Rightarrow 0.233 \\ 50(20)(14)/60,000 = 0.233 \end{array} \right.$$

$\Rightarrow$  use 2 legs of #4 stirrups @ 14"

$$0.2 \times 2 = 0.4 > 0.233$$

Design Shear Reinforcement

$$S = \frac{A_v f_y d}{V_s} = \frac{0.4(60)(29.5)}{42.1} = 16.8$$

$\Rightarrow$  Space @ 14"

Stirrup Layout

$\Rightarrow$  Terminate stirrups at  $V_u < 27.98^k$

$$27.98 = 37.2 - 2.588(d)$$

$$d = 3.56'$$

Distance from support = 10.39 ft

Number of stirrups:

$$2'' + (n-1)(14) \geq 10.39(12)$$

$$n = 9.76 \Rightarrow 10 \text{ stirrups}$$

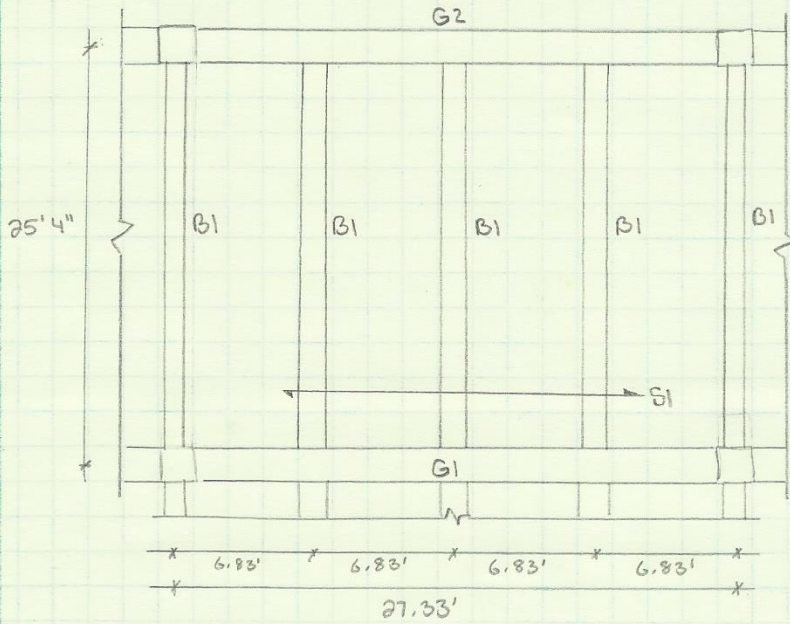
$\Rightarrow$  (10) #4 x 1 @ 14" starting 2" from each support

GIRDER SUMMARY  $\Rightarrow$  EXTERIOR

20" x 32" NWC, 4000psi  
 (5) #11 longitudinal bars (bottom)  
 (10) #4 x 1 spaced at 14" starting 2" from each support



One-Way Slab w/ Beams Design Summary



SLAB	THICKNESS	E-W DIRECTION		N-S DIRECTION
		TOP	BOTTOM	
S1	8.5"	#4 @ 12"	#4 @ 12"	#4" @ 12" For temperature & shrinkage

BEAM	SIZE	CONCRETE	f'c	REINFORCEMENT	
				LONGITUDINAL	TRANSVERSE
B1	13" x 22"	NWC	4000 psi	(4) #8	(13) #3 x M @ 9"

GIRDER	SIZE	CONCRETE	f'c	REINFORCEMENT	
				LONGITUDINAL	TRANSVERSE
G1	21" x 36"	NWC	4000 psi	(6) #11	(35) #4 x M Evenly spaced
G2	20" x 32"	NWC	4000 psi	(5) #11	(10) #4 x M @ 14"

**Structural Redesign 3:  
Two-Way Slab with Drop Panels**



Structural Redesign - Two-way Slab w/ Drop Panels

⇒ In determining if I should design the slab w/ drop panels I used the CRSI Design Handbook. Pg 9-31 recommends the following:

For a 27' span w/ 150 psf alone (not including dead load) a 9" slab and 36" columns are required. The current column sizes (including sleeve) are 24" and the architect requested the columns are kept at this size or smaller

Thus, I chose to use drop panels

⇒ using the CRSI Handbook pg 10-23 recommends:

For a 27' span w/ super imposed load = 400 psf a 9" slab and 21" columns.

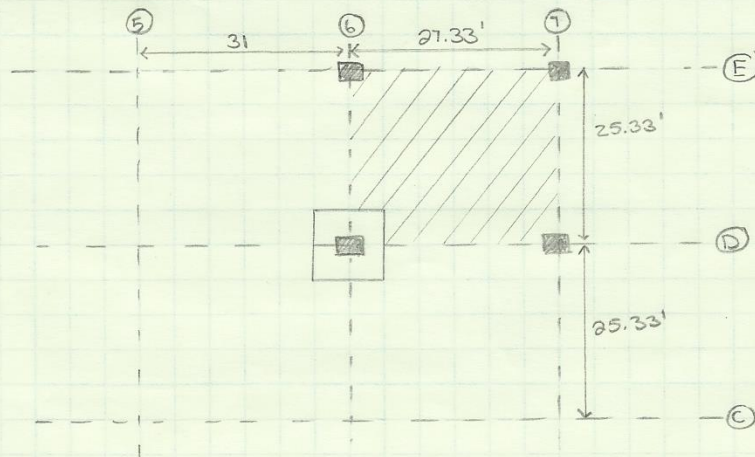
This is much more reasonable for my design.

Determine Slab Thickness

⇒ using Table 9.5c from ACI 318-11

- Slab w/out interior beams
- w/ Drop Panels
- exterior Panels
- w/o edge beams
- use 24" x 24" Columns (See SP Slab printout in Appendix)

$$h = \frac{l_n}{36} = \frac{31'0" (12) - 24"}{36} = 9.67 \Rightarrow \text{use } 10" \text{ Slab}$$



Trib Area of column D6 = 29.165' x 25.33'  
Trib Area of column E6 = 29.165' x 12.665'

Determine Loading on Slab

$$LL = 150 \text{ psf}$$

$$\text{Misc DL} = 25 \text{ psf}$$

$$S_w \text{ slab} = 1/2 (150) = 120 \text{ psf}$$

$$q_u \text{ (w/o drop panels)} = 1.2(120 + 25) + 1.6(150) = 414 \text{ psf}$$

Amrad



### Interior Column (D6)

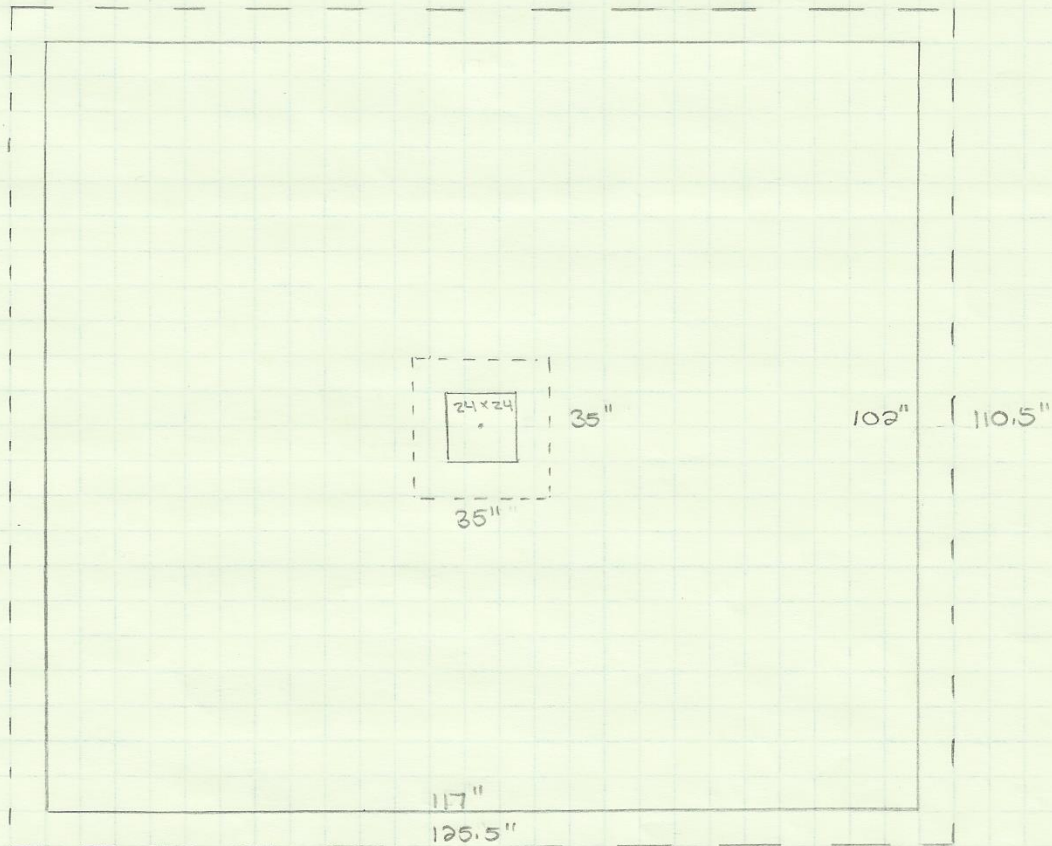
$$\text{ACI 318-11 13.2.5(a)} \Rightarrow t = 1/4 (10) = 2.5''$$

$$\text{ACI 318-11 13.2.5(b)} \Rightarrow l/b = 27.33'(12)/b = 54.66 \Rightarrow 55''$$

$$l/b = 25.33'(12)/b = 50.66 \Rightarrow 51''$$

$$l/b = 31(12)/b = 62''$$

Drop Panel Layout:



### Critical Sections

$$\text{At column: } t = 10 + 2.5 = 12.5 \Rightarrow d = 11$$

$$C.S. = 24'' + 11 = 35''$$

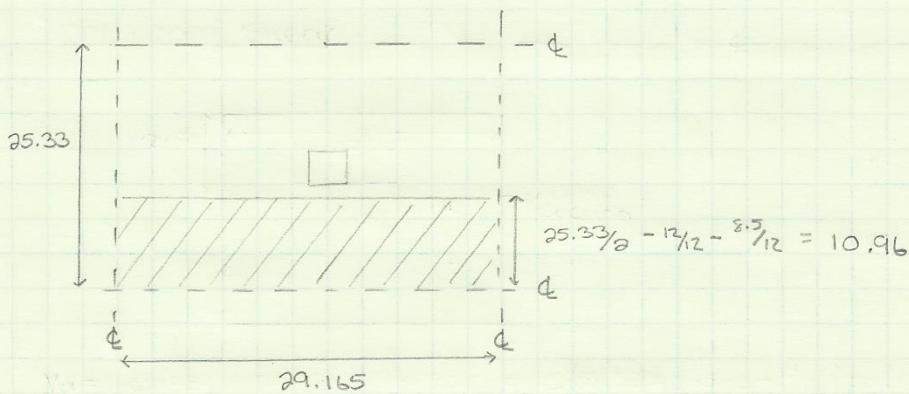
$$\text{At drop Panel: } t = 10 \Rightarrow d = 8.5$$

$$C.S. = 117 + 8.5 = 125.5$$

$$C.S. = 102 + 8.5 = 110.5$$

AMRAD

Check Shear Strength - one way Shear

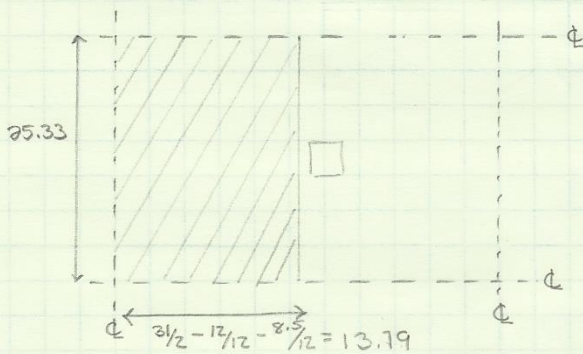


$$V_u = 0.414 (10.96 \times 29.165) = 132.3^k$$

$$V_c = 2 \sqrt{f_c} b w d = 2 (1.0) \sqrt{4000} (29.165 \times 12) \times 8.5 / 1000 = 376.3^k$$

$$\phi V_c = 0.75 (376.3) = 282.2^k$$

$$= 282.2^k > 132.3^k \quad \checkmark$$



$$V_u = 0.414 (25.33 \times 13.79) = 144.6^k$$

$$V_c = 2 (1) (\sqrt{4000}) (25.33 \times 12) \times 8.5 / 1000 = 326.8^k$$

$$\phi V_c = 0.75 (326.8) = 245.1^k$$

$$= 245.1^k > 144.6^k \Rightarrow \text{Slab OK for one-way Shear}$$



Check Shear Strength - two way shear⇒ At critical section of column ( $d=11$  in)

$$q_u (\text{drop panel}) = 1.2 \left( \frac{2.5}{12} \right) 150 = 37.5 \text{ psf} \Rightarrow 0.0375 \text{ ksf}$$

$$V_u = 0.414 \left[ (29.165 \times 25.33) - \left( \frac{35}{12} \right)^2 \right] + 0.0375 \left[ \left( \frac{117}{12} \times \frac{110.5}{12} \right) - \left( \frac{35}{12} \right)^2 \right]$$

$$= 302.32 + 3.05$$

$$= 305.37^k$$

 $\phi V_n$ :

$$b_o = 2(c_1 + c_2 + 2d_i) \\ = 2(24 + 24 + 2(11)) \\ = 140$$

$$\lambda = 1.0$$

$$\beta_1 = 1$$

$$\alpha_s = 40$$

$$\text{Min} \begin{cases} \left( \frac{2 + \frac{4}{\beta}}{\beta} \right) = 6 \\ \left( \frac{\alpha_s \cdot d}{b_o} + 2 \right) = \left( \frac{40(11)}{140} + 2 \right) = 5.14 \Rightarrow 4 \\ 4 \end{cases}$$

$$\phi V_n = \phi V_c = \phi 4 \lambda \sqrt{f_c} b_o d = 0.75(4)(1) \sqrt{4000} (140)(11) / 1000 \\ = 292.2^k$$

$$\Rightarrow 292.2^k < 305.37 \text{ NOT GOOD}$$

Must increase drop panel thickness

$$\Rightarrow \text{Try } t = 3'' \Rightarrow \text{total thickness} = 10 + 3 = 13''$$
$$d = 11.5''$$

Critical section

$$\text{At column} = 24 + 11.5 = 35.5''$$

$$w_u (\text{drop panel}) = 1.2 \left( \frac{3}{12} \times 150 \right) = 45 \text{ psf}$$

$$V_u = 0.414 \left[ (29.165 \times 25.33) - \left( \frac{35.5}{12} \right)^2 \right] + 0.045 \left[ \left( \frac{117}{12} + \frac{110.5}{12} \right) - \left( \frac{35.5}{12} \right)^2 \right]$$
$$= 302.2 + 3.65$$
$$= 305.85^k$$

$$\phi V_n = \phi V_c = 0.75 \phi \left( \lambda \beta_1 \alpha \rho_s \frac{b_o d}{s} \right) = 0.75 (4) (1) \sqrt{4000} \left( \frac{142 (11.5)}{1000} \right) = 309.8^k$$

$$b_o = 2(24 + 24 + 2(11.5))$$
$$= 142$$

$$\lambda = 1.0$$

$$\beta_1 = 1.0$$

$$\alpha = 40$$

$$\text{Min } \left\{ \begin{array}{l} 6 \\ \left( \frac{\alpha_s \cdot d}{b_o} + 2 \right) = \left( \frac{40(11.5)}{142} + 2 \right) = 5.2 \Rightarrow 4 \\ 4 \end{array} \right.$$

$$\phi V_c = 0.75 (4) (1) \sqrt{4000} (142)(11.5) / 1000 = 309.8^k$$

$$\Rightarrow 309.8^k > 305.85^k \checkmark$$



⇒ Check Shear at critical section of drop panel

$$V_u = 0.414 \left[ (29.165 \times 25.33) - \left( \frac{125.5}{12} \times \frac{110.5}{12} \right) \right] = 266^k$$

$\phi V_n$ :

$$b_o = 2(117 + 102 + 2(8.5)) = 472 \text{ in}$$

$$\lambda = 1.0$$

$$\beta_1 = 1.0$$

$$\alpha = 40$$

$$\text{Min} \begin{cases} 6 \\ \left( \frac{\alpha \cdot d}{b_o} + 2 \right) = \left( \frac{40 + 8.5}{472} + 2 \right) = 2.10 \\ 4 \end{cases}$$

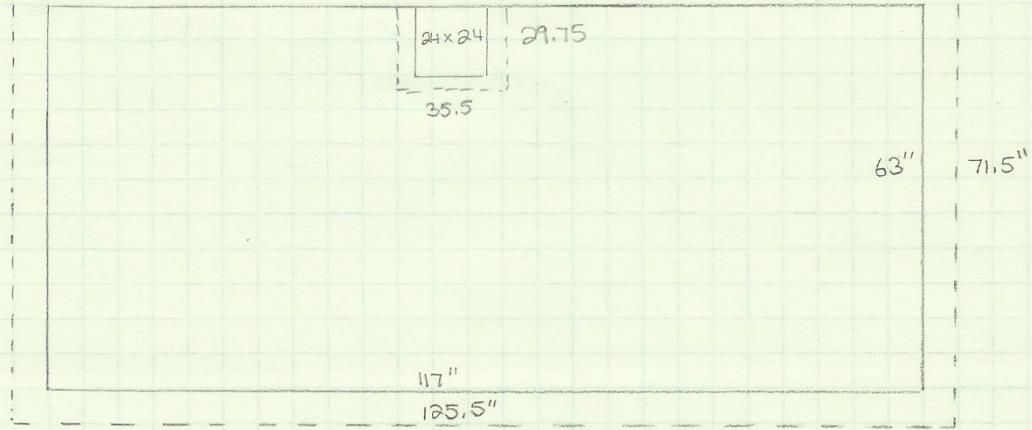
$$\phi V_c = 0.75(2.10)(1) \sqrt{4000} (472)(7.5) / 1000 = 399.6^k$$

$$\Rightarrow 399.6^k > 266^k \checkmark$$

A 117" x 102" drop panel with a 3 in projection below the slab will provide adequate shear strength at interior column.

### Exterior Column (E6)

⇒ Based on interior column design  $t = 3''$



#### Critical Sections

At column:  $t = 10 + 3 = 13'' \Rightarrow d = 11.5''$

CS =  $35.5'' \times 29.75''$

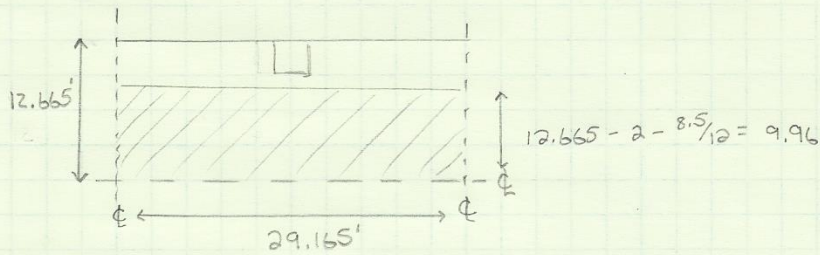
At drop panel:  $t = 10 \Rightarrow d = 8.5$

CS =  $117 + 8.5 = 125.5''$

CS =  $63 + 8.5 = 71.5''$



Check Shear Strength - one way shear



$$V_u = 0.414 (9.96 \times 29.165) = 120.3^k$$

$$V_c = 2(1)\sqrt{4000} (29.165 \times 12)(8.5) / 1000 = 376.3$$

$$\phi V_c = 282.2^k > 120.3^k \checkmark$$



$$V_u = 0.414 (12.665 \times 12.87) = 67.5^k$$

$$V_c = 2(1)\sqrt{4000} (12.665 \times 12)(8.5) / 1000 = 163.4^k > 67.5^k \checkmark$$

Two-way Slab w/ Drop Panels  
Ext - Two way Shear

### Check Shear Strength - two way Shear

⇒ At critical Section of Column ( $d = 11.5 \text{ in}$ )

$$\begin{aligned} V_u &= 0.414 \left[ (29.165 \times 12.665) - \left( \frac{29.75}{12} \times \frac{35.5}{12} \right) \right] + 0.045 \left[ \left( \frac{117}{12} \times \frac{63}{12} \right) - \left( \frac{29.75}{12} \times \frac{35.5}{12} \right) \right] \\ &= 149.9 + 1.97 \\ &= 151.87^k \end{aligned}$$

$\phi V_n$ :

$$\begin{aligned} b_o &= 29.75(2) + 35.5 = 95 \\ \lambda &= 1.0 \\ \beta_1 &= 1 \\ \alpha &= 40 \end{aligned}$$

$$\text{Min} \begin{cases} 6 \\ \left( \frac{40(11.5) + 2}{95} \right) = 1.68 \\ 4 \end{cases}$$

$$\begin{aligned} \phi V_c &= 0.75(4)(1) \sqrt{4000} (95)(11.5) / 1000 = 207.3^k \\ &\Rightarrow 207.3 > 151.87^k \end{aligned}$$

⇒ At critical Section of Drop Panel

$$V_u = 0.414 \left[ (29.165 \times 12.665) - \left( \frac{125.5}{12} \times \frac{71.5}{12} \right) \right] = 151.1^k$$

$\phi V_n$ :

$$b_o = 125.5 + 71.5 + 71.5 = 268.5$$

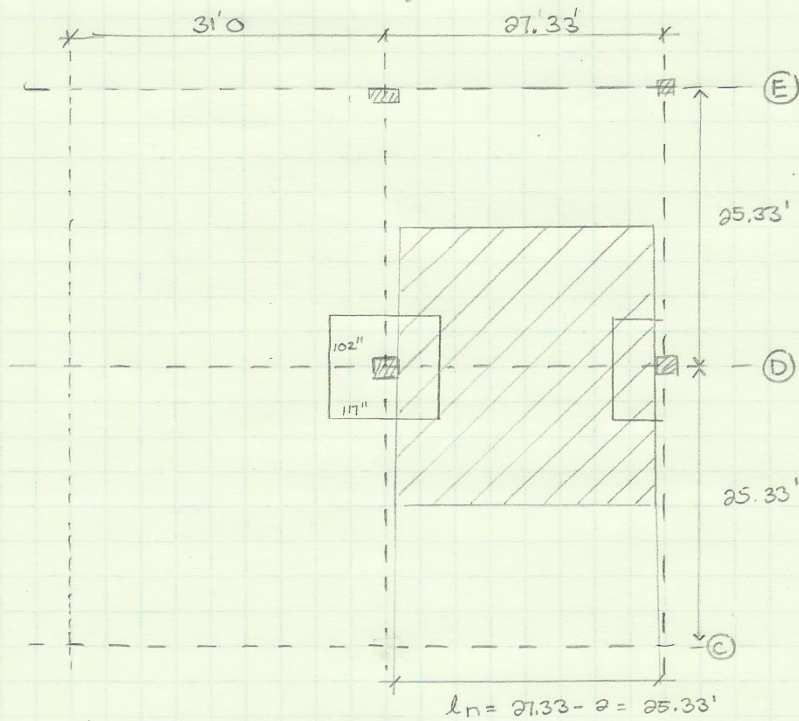
$$\text{Min} \begin{cases} 6 \\ \left( \frac{40 + 8.5 + 2}{268.5} \right) = 2.18 \\ 4 \end{cases}$$

$$\begin{aligned} \phi V_c &= 0.75(2.18)(1) \sqrt{4000} (268.5)(8.5) / 1000 = 236^k \\ &\Rightarrow 236^k > 151.1^k \end{aligned}$$

A 117" x 63" drop panel with a 3 in projection below the slab will provide adequate shear strength at exterior column



Check Flexural Strength



Interior Span

⇒ Use Direct Design Method

\* All limitations of ACI 318-11 13.6.1

\* using drop-panel thickness to be conservative

$$q_u = 414 \text{ psf} + 1.2 \left( \frac{13}{12} \times 150 \right) = 609 \text{ psf} \Rightarrow 0.609 \text{ ksf}$$

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{0.609 \text{ ksf} (25.33 \text{ ft}) (25.33)^2}{8} = 1237 \text{ k}$$

Distribution of Moment

$$M_u^- = 0.65 (1237) = 804.1 \text{ k}$$

$$M_u^+ = 0.35 (1237) = 433 \text{ k}$$

Transverse distribution of moments to column & middle strip

$$\text{Interior column Strip negative moment} = 0.75 (804.1) = 603.1 \text{ k}$$

$$\text{Interior middle Strip negative} = 0.25 (804.1) = 201 \text{ k}$$

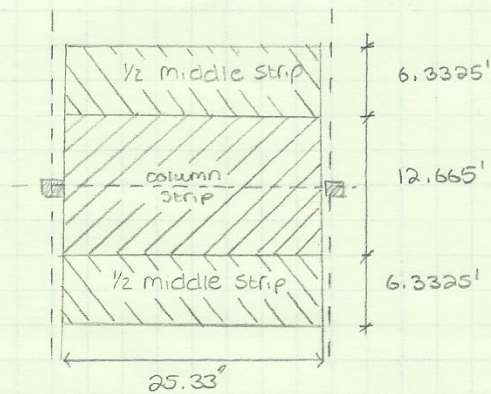
$$\text{Column Strip positive moment} = 0.6 (433) = 259.8 \text{ k}$$

$$\text{Middle Strip positive moment} = 0.4 (433) = 173.2 \text{ k}$$

Column Strip / Middle Strip Dimensions

$$l_1 = 27.33' \quad l_1 > l_2$$

$$l_2 = 25.33'$$

Determine Reinforcement - negative reinforcementColumn strip

$$A_s^- = \frac{M_u^-}{4d} = \frac{603.1^{1k}}{4(8.5)} = 17.7 \text{ in}^2 \Rightarrow \underline{\underline{(23) \# 8 = 18.17 \text{ in}^2}}$$

Check flexural strength

$$a = \frac{(18.17)(60)}{0.85(4)(12.665 \times 12)} = 2.11 \quad c = 2.11 / 0.85 = 2.48$$

$$M_n = 18.17 (60)(8.5 - 2.11/2) / 12 = 676.4^{1k}$$

$$\epsilon_s = \frac{0.003(8.5 - 2.48)}{2.48} = 0.007 > 0.00207 \checkmark$$

$$\phi M_n = 0.9(676.4) = 608.8 > 603.1^{1k} \checkmark$$

Check min reinforcement

$$A_{s \text{ min}} \geq \frac{200(12.665 \times 12)(8.5)}{60,000}$$

$$= 4.3 \text{ in}^2 < 18.17 \text{ in}^2 \checkmark$$

check max reinforcement

$$A_{s \text{ max}} = 0.85^2 \left( \frac{4}{60} \right) \left( \frac{3}{7} \right) (12.665 \times 12)(8.5)$$

$$= 26.67 \text{ in}^2 > 18.17 \text{ in}^2 \checkmark$$



Check min spacing

$$\text{Min clear dist} = \begin{cases} 1'' \\ 1'' \\ \text{max } 4/3 \end{cases} \Rightarrow 4/3$$

$$\text{Actual spacing} = \frac{12.665 \times 12 - 23(1)}{22} = 5.86'' > 4/3'' \checkmark$$

Middle Strip

$$A_s^- = \frac{M_u^-}{4d} = \frac{201}{4(8.5)} = 5.9 \text{ in}^2 \Rightarrow \text{Try } (19) \# 5 = 5.89 \text{ in}^2$$

Check flexural strength

$$a = \frac{(5.89)(60)}{0.85(4)(12.665 \times 12)} = 0.684 \quad c = 0.684/0.85 = 0.805$$

$$M_n = 5.89(60)(8.5 - \frac{0.684}{2})/12 = 240.3 \text{ k}$$

$$\epsilon_s = \frac{0.003}{0.805}(8.5 - 0.805) = 0.0291 > 0.00207 \checkmark$$

$$\phi M_n = 240.3 \text{ k} > 201 \text{ k} \checkmark$$

Check min reinforcement

$$A_{s \text{ min}} \geq \frac{200(12.665 \times 12)(8.5)}{60,000} = 4.31 < 5.89 \checkmark$$

Check max reinforcement

$$A_{s, \text{ max}} = 0.85^2 \left(\frac{4}{60}\right) \left(\frac{3}{7}\right) (12.665 \times 12)(8.5)$$

$$= 31.4 \text{ in}^2 > 5.89 \text{ in}^2 \checkmark$$

Check min spacing

$$\text{min} = 4/3$$

$$\text{Actual} = \frac{(12.665 \times 12) - 19(0.625)}{18} = 7.78'' > 4/3'' \checkmark$$

Determine Reinforcement - positive reinforcement

Column Strip

$$A_s^+ = \frac{M_u^+}{4d} = \frac{259^{1k}}{4(8.5)} = 7.62 \text{ in}^2$$

check min reinforcement

$$A_{smin} = 4.3 \text{ in}^2 < 7.62 \text{ in}^2 \checkmark$$

$$\Rightarrow \underline{\text{use (10) \# 8}} \Rightarrow 7.9 \text{ in}^2 > 7.62 \text{ in}^2 \checkmark$$

check max reinforcement

$$A_{smax} = 26.67 \text{ in}^2 > 7.9 \text{ in}^2 \checkmark$$

check min spacing

$$\text{Min spacing} = 4/3'' < \text{Act. Spacing} = 15.8'' \checkmark$$

check flexural strength

$$a = 0.917 \quad c = 1.08$$

$$\phi M_n = 283^{1k} > 259^{1k}$$

$$\epsilon_s = 0.0206 > 0.00207$$

Middle Strip

$$A_s^+ = \frac{M_u^+}{4d} = \frac{173.2^{1k}}{4(8.5)} = 5.09 \text{ in}^2 \Rightarrow (17) \# 5 \Rightarrow 5.27 \text{ in}^2 \checkmark$$

$\Rightarrow$  based on previous middle strip this passes  
max reinf, min reinf, & max spacing

Flexural Strength

$$a = 0.612 \quad c = 0.72$$

$$\epsilon_s = 0.032 > 0.00207$$

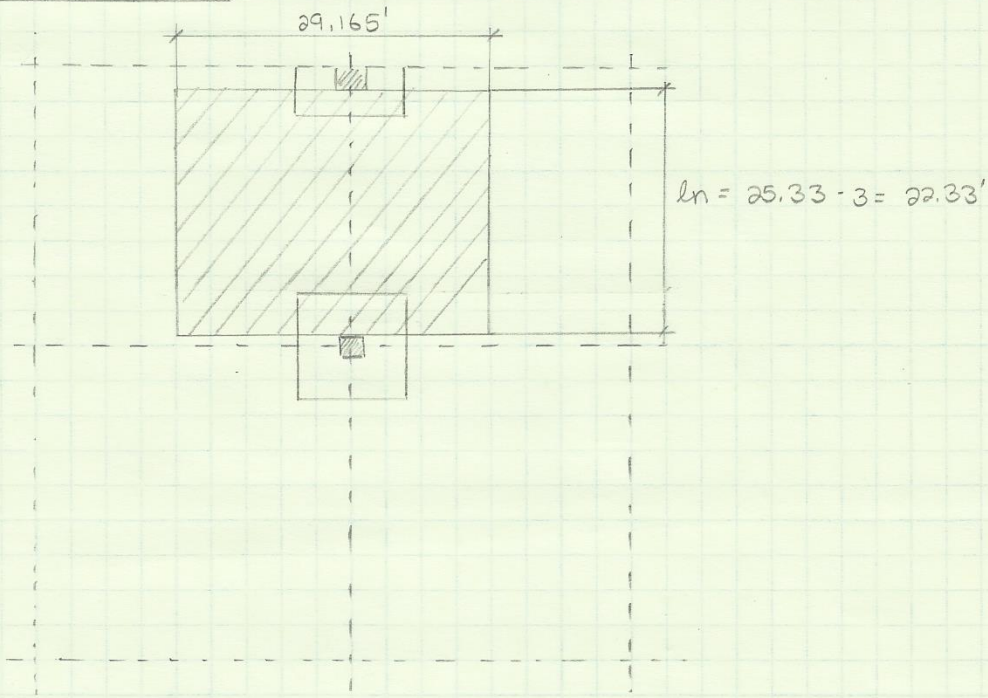
$$\phi M_n = 194^{1k} > 173.2^{1k} \checkmark$$

$\Rightarrow$  Interior Span Summary

Column Strip: Top = (23) # 8 @ 12.77"  
Bottom = (10) # 8 @ 15.8"

Middle Strip: Top = (19) # 5 @ 7.78"  
Bottom = (17) # 5 @ 8.83"



Exterior Span

⇒ Slab w/o beams between interior supports + w/o edge beams

$$M_o = \frac{q_u l_z l_n^2}{8} = \frac{0.609 \text{ ksf} (29.165) (22.33)^2}{8} = 1107 \text{ k}$$

Distribution of Moments

$$M_{\text{int}}^- = 0.7 (1107 \text{ k}) = 774.9 \text{ k}$$

$$M^+ = 0.52 (1107) = 575.6 \text{ k}$$

$$M_{\text{ext}}^- = 0.26 (1107) = 287.8 \text{ k}$$

Transverse Distribution of Moments to column/middle strip

$$\text{Interior column strip neg moment} = 0.75 (774.9) = -581.1 \text{ k}$$

$$\text{Interior middle strip neg moment} = 0.25 (774.9) = -193.7 \text{ k}$$

$$\text{Column strip positive moment} = 0.6 (575.6) = 345.4 \text{ k}$$

$$\text{Middle strip positive moment} = 0.4 (575.6) = 230.2 \text{ k}$$

$$\text{Exterior Column Strip neg moment} = 1.0 (287.8) = -287.8 \text{ k}$$

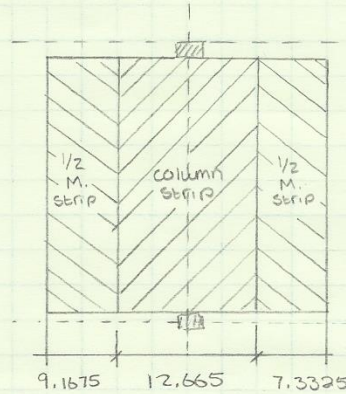
$$\text{Exterior Middle strip neg moment} = 0 \text{ k}$$



Column Strip / Middle Strip Dimensions

$$l_1 = 25.33 \Rightarrow l_2 > l_1$$

$$l_2 = 27.33$$



Total MS width = 16.5 ft

Determine Reinforcement - negative reinforcement

Column Strip

$$A_s = \frac{M_u}{4d} = \frac{581.1}{4(8.5)} = 17.09 \text{ in}^2 \Rightarrow (22)^{\#8} = 17.38 > 17.09 \checkmark$$

check min reinforcement

$$A_{s, \text{min}} \geq \frac{200(12.665 \times 12)(8.5)}{60,000} = 4.3 < 17.38 \checkmark$$

check max reinforcement

$$A_{s, \text{max}} = 0.85^2 \left(\frac{4}{60}\right)^{3/4} (12.665 \times 12)(8.5)$$

$$= 26.67 \text{ in}^2 > 17.38 \text{ in}^2 \checkmark$$

check min spacing

$$\text{Min clear dist} = \begin{cases} 0.75 \\ 1" \\ \text{max } 4/3" \end{cases} \Rightarrow 4/3"$$

$$\text{Actual Spacing} = \frac{12.665(12) - 22(1)}{21} = 6.2 > 4/3 \checkmark$$

check flexural strength

$$a = \frac{(17.38)(60)}{0.85(4)(12.665 \times 12)} = 2.02 \quad C = \frac{2.02}{0.85} = 2.38$$

$$E_s = \frac{0.003(8.5 - 2.38)}{2.38} = 0.0077 > 0.00207 \checkmark$$

$$\phi M_n = 0.9(17.38)(60)(8.5 - \frac{2.02}{2})/12 = 585.8 \text{ k} > 581.1 \checkmark$$



Middle Strip

$$A_s^- = \frac{M_u^-}{4d} = \frac{193.7}{4(8.5)} = 5.70 \Rightarrow \underline{(19)\#5} \Rightarrow 5.89 > 5.70 \checkmark$$

check min reinforcement

$$A_{min} \geq \frac{200(16.5 \times 12)(8.5)}{60,000} = 5.61 < 5.89 \checkmark$$

check max reinforcement

$$A_{smax} = 0.85^2 \left(\frac{4}{60}\right) \left(\frac{3}{7}\right) (16.5 \times 12)(8.5) \\ = 34.7 \text{ in}^2 > 5.89 \text{ in}^2 \checkmark$$

check min spacing

$$\text{Min clear dist} = \begin{array}{l} 0.625 \\ 1 \\ \text{max } 4/3 \end{array} \Rightarrow 4/3$$

$$\text{Actual } S = \frac{(16.5 \times 12) - 19(0.625)}{18} = 11''$$

check flexural strength

$$a = \frac{(5.89)(60)}{0.85(4)(16.5 \times 12)} = 0.525 \quad c = \frac{0.525}{0.85} = 0.618$$

$$\epsilon_s = \frac{0.003}{0.618} (8.5 - 0.618) = 0.0383 > 0.00207 \checkmark$$

$$\phi M_n = 0.9 (5.89)(60)(8.5 - \frac{0.525}{2}) / 12 = 218.3 \text{ k} > 193.7 \text{ k} \checkmark$$

Determine Reinforcement - Positive Reinforcement

Column Strip

$$A_s^+ = \frac{M_u^+}{4d} = \frac{345.4^k}{4(8.5)} = 10.2 \text{ in}^2 \Rightarrow (13)^\# 8 \Rightarrow 10.27 \checkmark$$

check min reinforcement

$$A_{s, \text{min}} = 4.3 > 10.27 \checkmark$$

check max reinforcement

$$A_{s, \text{max}} = 26.67 \text{ in}^2 > 10.27 \text{ in}^2 \checkmark$$

check min spacing

$$\text{Min clear dist} = 4/3$$

$$\text{Actual } S = \frac{12.665(12) - 13(1)}{12} = 11.58 > 4/3 \checkmark$$

check Flexural Strength

$$a = \frac{(10.27)(60)}{0.85(4)(12.665)(12)} = 1.19 \quad c = 1.19/0.85 = 1.4$$

$$\epsilon_s = \frac{0.003}{1.4} (8.5 - 1.4) = 0.0152 > 0.00207 \checkmark$$

$$\begin{aligned} \phi M_n &= 0.9 (10.27)(60)(8.5 - 1.19/2) \\ &= 365.3^k > 345.4^k \checkmark \end{aligned}$$



Middle Strip

$$A_s^+ = \frac{M_u^+}{4d} = \frac{230.2}{4(8.5)} = 6.77 \Rightarrow (22)\#5 = 6.82 > 6.77 \checkmark$$

Check min reinforcement

$$A_{s,min} = 5.61 \text{ in}^2 < 6.82 \text{ in}^2 \checkmark$$

Check max reinforcement

$$A_{s,max} = 34.7 \text{ in}^2 < 6.82 \text{ in}^2 \checkmark$$

Check min spacing

$$\text{Min clear} = 4/3$$

$$\text{Actual } S = \frac{(16.5 \times 12) - 22(0.625)}{21} = 8.77 \text{ in} > 4/3 \checkmark$$

Check flexural strength

$$a = \frac{(6.82)(60)}{0.85(4)(16.5 \times 12)} = 0.608 \quad C = \frac{0.608}{0.85} = 0.715$$

$$\epsilon_s = \frac{0.003}{0.715} (8.5 - 0.715) = 0.0327 > 0.00207 \checkmark$$

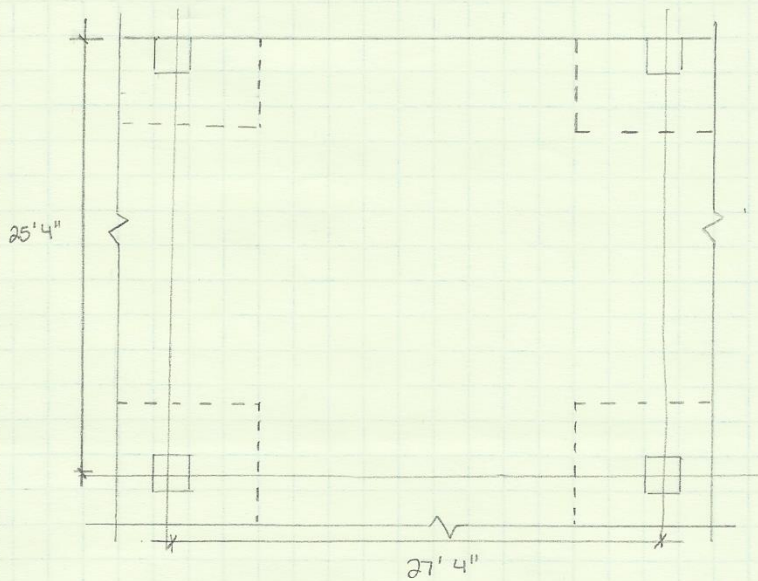
$$\phi M_n = 0.9(6.82)(60)(8.5 - \frac{0.608}{2}) / 12 = 251.5 \text{ k} > 230.2 \checkmark$$

 $\Rightarrow$  Exterior Span Summary

Column Strip : Top = (22) # 5 @ 6.2"  
Bottom = (13) # 8 @ 11.6"

Middle Strip : Top = (19) # 5 @ 11"  
Bottom = (22) # 5 @ 8.8"

## Two-Way Slab w/ Drop Panels Design Summary



Note: columns shown are 24" x 24" (see SP column in appendix)

### Slab Notes:

Thickness = 10"  
 $f'_c = 4000 \text{ psi}$   
NWC

### Reinforcing:

Top (E-W)  $\Rightarrow$  Column strip: (23) #8  
Middle strip: (19) #5

Top (N-S)  $\Rightarrow$  Column strip: (22) #8  
Middle strip: (19) #5

Bottom (E-W)  $\Rightarrow$  Column strip: (10) #8  
Middle strip: (11) #5

Bottom (N-S)  $\Rightarrow$  Column strip: (13) #8  
Middle strip: (22) #5

### Drop Panel Notes:

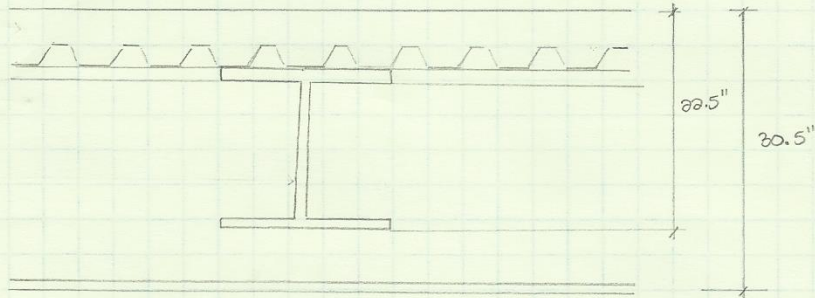
Exterior: 11" x 63" projecting 3" below slab

Interior: 11" x 102" projecting 3" below slab

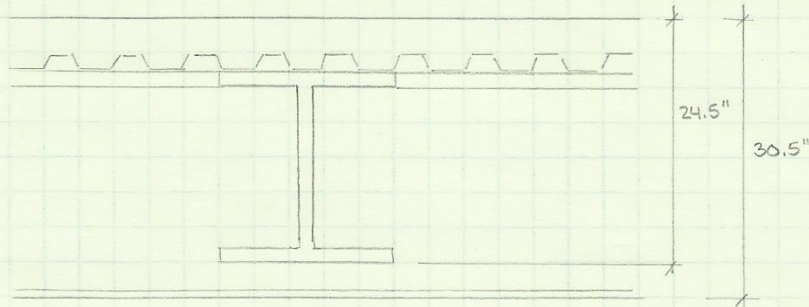


# Depth Comparison

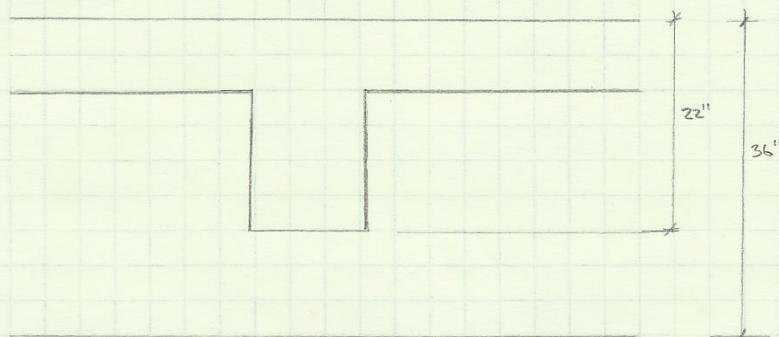
## Existing Composite Steel System



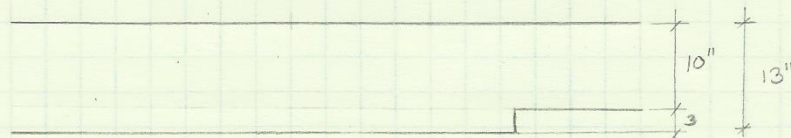
## Non-composite Steel System



## One-way Slab w/ Beams



## Two-way slab w/ drop Panels



AMPAD

Weight ComparisonExisting Composite Steel System

$$\text{Beams: } 26\text{plf } (25.33\text{ft}) \times (5 \text{ beams}) = 3293 \text{ lbs}$$

$$\text{Girders: } 55\text{plf } (27.33\text{ft}) + 68(27.33\text{ft}) = 3362 \text{ lbs}$$

$$\text{Deck: } 69\text{psf } (27.33 \times 25.33) = 47,767 \text{ lbs}$$

$$\text{Total weight} = \frac{54422 \text{ lbs}}{(27.33 \times 25.33)} = \boxed{78.6 \text{ psf}}$$

Non-composite Steel System

$$\text{Beams: } 35\text{plf } (25.33)(5 \text{ beams}) = 4433 \text{ lbs}$$

$$\text{Girders: } 55\text{plf } (27.33\text{ft}) + 68(27.33\text{ft}) = 3362 \text{ lbs}$$

$$\text{Deck: } 69\text{psf } (27.33 \times 25.33) = 47,767 \text{ lbs}$$

$$\text{Total weight} = \frac{55,562 \text{ lbs}}{(27.33 \times 25.33)} = \boxed{80.3 \text{ psf}}$$

One-way Slab with Beams

$$\text{Beams: } [(13 \times 22)/144 \times 25.33\text{ft}] \times 150\text{pcf} \times 5 \text{ beams} = 37,732 \text{ lbs}$$

$$\text{Girders: } \left[ \frac{(21 \times 36)}{144} \times 27.33 + \frac{(20 \times 32)}{144} \times 27.33 \right] \times 150 = 39,743 \text{ lbs}$$

$$\text{Slab: } \left[ \left( \frac{85}{12} \times 27.33 \times 25.33 \right) \times 150 \right] = 73,554 \text{ lbs}$$

$$\text{Total weight} = \frac{151,029}{(27.33 \times 25.33)} = \boxed{218.2 \text{ psf}}$$

Two-way Slab with Drop Panels

$$\text{Slab: } \left[ \left( \frac{10}{12} \times 27.33 \times 25.33 \right) \times 150 \right] = 86,534 \text{ lbs}$$

$$\text{Drop Panels: } \left[ 2 \left( \frac{1}{2} \times 117 \times \frac{63}{144} \right) (3'') + 2 \left( \frac{1}{4} \times 117 \times \frac{102}{144} \right) (3'') \right] \times 150 = 41,682 \text{ lbs}$$

$$\text{Total weight} = \frac{128,216 \text{ lbs}}{(27.33 \times 25.33)} \Rightarrow \boxed{185.2 \text{ psf}}$$



## Cost Comparison

### Existing composite steel system

⇒ cost estimate done using RS Mean B1010 256  
w/ a location multiplier for VA = 0.85

includes: Composite beams w/ shear studs, deck, slab + WWF, girders

$$\text{Bay size} = 25.33' \times 27.33'$$

$$\text{Total Load} = 100 + 150 = 250 \text{ psf}$$

Bay Size	Total Load	Total cost per SF
25 x 25 = 625	252 psf	25.90
25 x 27 = 675	⇒ use 252 psf ( > 250 )	?
25 x 30 = 750	252 psf	26

$$\frac{750 - 625}{26 - 25.90} = \frac{750 - 675}{26 - x} \Rightarrow 25.94 (0.85) = \boxed{\$22.05/\text{SF}}$$

### Non-composite steel system

⇒ cost estimate done using RS mean B1010 254  
w/ location multiplier for VA = 0.85

includes: beams, girders, composite steel deck, slab  
reinforced w/ WWF

$$\text{Bay size} = 25.33' \times 27.33'$$

$$\text{Total Load} = 100 + 150 = 250 \text{ psf}$$

Bay Size	Total Load	Total cost per SF	Cost per SF @ 250psf
25 x 25 = 625	181 psf	29.40	33.10
	263 psf	33.80	
25 x 27 = 675	250 psf	—	?
25 x 30 = 750	180 psf	29.20	34.87
	259 psf	35.60	

$$\frac{263 - 181}{33.8 - 29.4} = \frac{263 - 250}{33.8 - x} \Rightarrow x = 33.10$$

$$\frac{259 - 180}{35.6 - 29.20} = \frac{259 - 250}{35.6 - x} \Rightarrow x = 34.87$$

$$\frac{750 - 675}{34.87 - 33.10} = \frac{750 - 675}{34.87 - ?} \Rightarrow ? = 33.21 (0.85) \Rightarrow \boxed{\$28.74/\text{SF}}$$



One-way Slab w/ Beams

⇒ cost estimate done using RS Mean B1010 219  
w/ a location multiplier for VA = 0.85

includes: solid concrete one way slab cast monolithically  
w/ reinforced concrete beams + girders

$$\text{Bay size} = 25.33 \times 27.33$$

$$\text{Slab thickness} = 8.5''$$

$$\text{Total load} = 131 + 150 = 281 \text{ psf}$$

Bay size	Total load	Cost per SF	Cost per SF @ 281 psf
25 x 25 = 625	227	20	20
25 x 27 = 675	281	-	?
30 x 35 = 1050	254	21.90	22.33
	332	23.15	

$$\frac{332 - 254}{23.15 - 21.90} = \frac{332 - 281}{23.15 - x} \Rightarrow x = 22.33$$

$$\frac{1050 - 625}{22.33 - 20} = \frac{1050 - 675}{22.33 - ?} \Rightarrow ? = 20.27 (-.85) = \boxed{\$17.23 / \text{SF}}$$

Two-way Slab w/ Drop Panels

⇒ cost estimate done using RS mean B1010 222 w/ location  
multiplier for VA = 0.85

includes: Flat solid slab w/ drop panels

$$\text{Bay Size} = 25.33 \times 27.33$$

$$\text{Slab thickness} = 10''$$

$$\text{Total load} = 145 + 150 = 295 \text{ psf}$$

Bay Size	Total load	Cost per SF	Cost per SF @ 295 psf
25 x 25 = 625	243	16.85	17.21
	329	17.45	
25 x 27 = 675	295	-	?
25 x 30 = 750	203	17.05	18.00
	256	17.60	



$$\frac{329-243}{17.45-16.25} = \frac{329-295}{17.45-x} \Rightarrow x = 17.21$$

$$\frac{295-203}{x-17.05} = \frac{256-203}{17.60-17.05} \Rightarrow x = 18.00$$

$$\frac{750-625}{18-17.21} = \frac{750-675}{18-?} \Rightarrow ? = 17.53 (.85) = \boxed{\$14.90/SF}$$

Award

## Fire Protection + Fire Rating

### Existing Composite Steel System

- ⇒ Fire proofing required for beams, girders + underside of deck
- ⇒ 4 1/2" NWC provides 2hr fire rating

### Non-composite steel system

- ⇒ Fire proofing required for beams, girders + underside of deck
- ⇒ 4 1/2" NWC provides 2hr fire rating

### One-way slab w/ Beams

- ⇒ No fire proofing required but necessary rebar cover is required

Required cover for 2hr rating:

Slabs ⇒ 3/4"

Beams ⇒ 3/4"

- ⇒ 2hr fire rating provided by 8.5" slab which is greater than the 4 1/2" required

### Two-way slab w/ Drop Panels

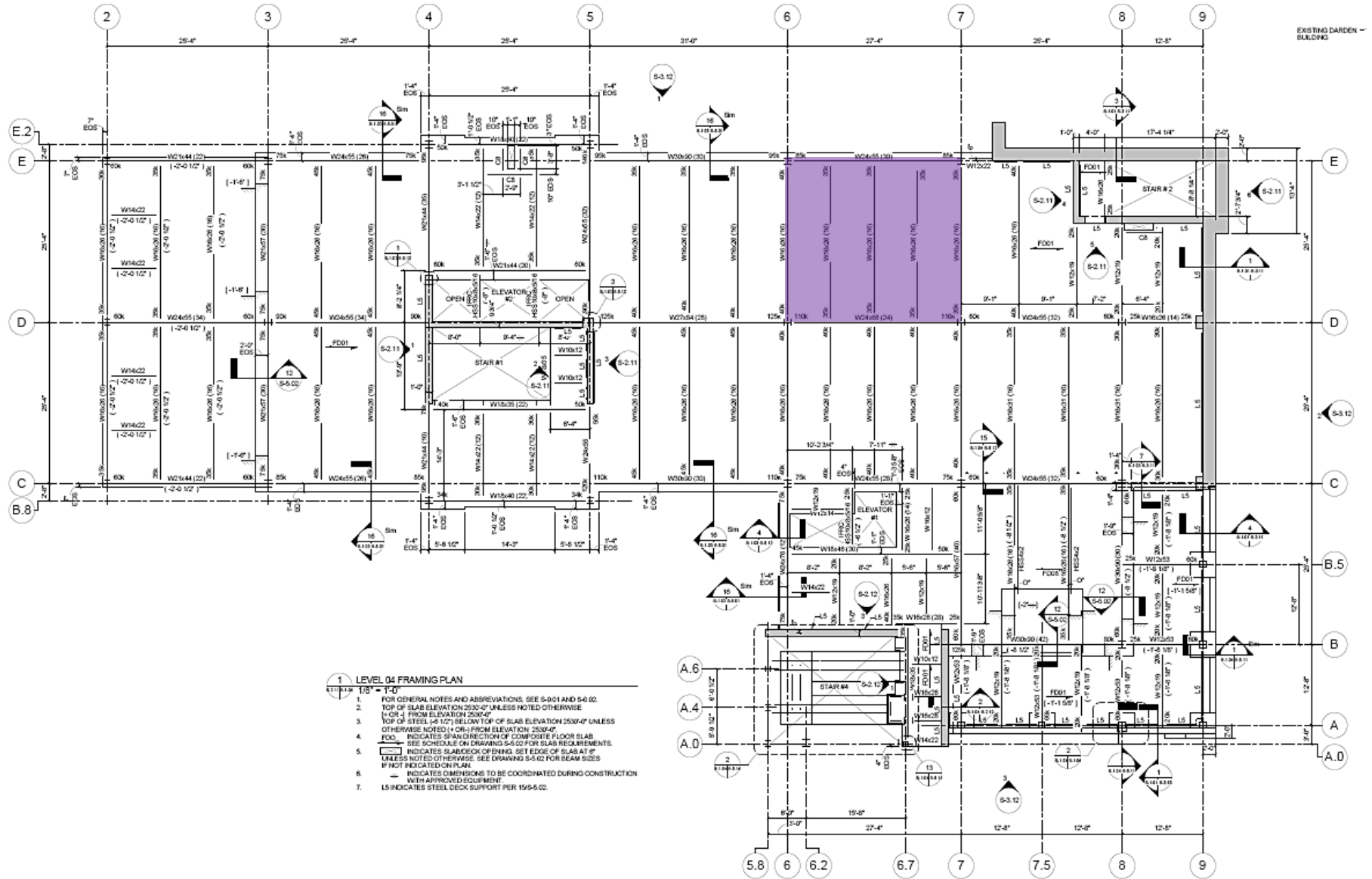
- ⇒ No fire proofing required but necessary rebar cover is required (same as one-way slab)
- ⇒ 2hr fire rating provided by 10" slab which is greater than the 4 1/2" required



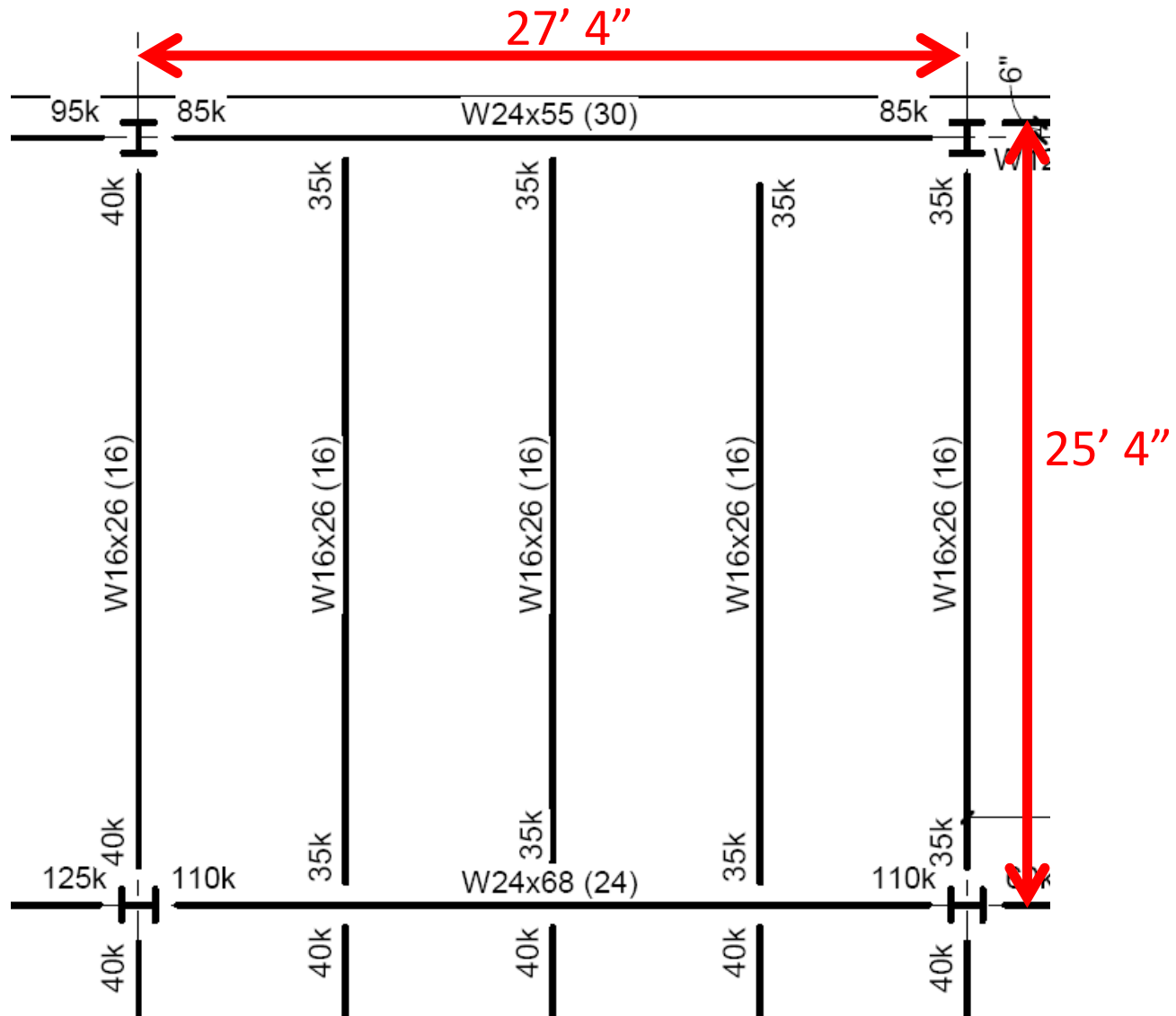
Floor System Designs				
Criteria	Composite Steel	Non-composite Steel	One-Way Slab with Beams	Two-Way Slab with Drop Panels
System Statistics				
Cost	\$22.05/SF	\$28.74/SF	\$17.23/SF	\$14.90/SF
Weight	78.6 PSF	80.3 PSF	218.2 PSF	185.2 PSF
Architectural				
Maximum Depth	30.5"	30.5"	36"	13"
Additional Fire Proofing Required	YES	YES	NO	NO
Fire Rating	2 HR	2 HR	2 HR	2 HR
Servicability				
Vibrations	Likely	Likely	Minimal	Minimal
Future Considerations				
Lateral Systems	Concrete Shear Wall	Concrete Shear Wall	Concrete Shear Wall, Concrete Moment Frame	Concrete Shear Wall, Concrete Moment Frame
Durability	Acceptable	Acceptable	Acceptable	Acceptable
Advantages	Light weight, average depth, minimal formwork required	Light weight, average depth, works well with multiple types of lateral systems, minimal formwork required	No additional fire proofing required, less expensive per SF, decreased vibrations	No additional fire proofing required, decreased depth, least expensive per SF, decreased vibrations
Disadvantages	More expensive per SF, additional fire proofing required, higher chance of vibrations	Most expensive per SF, additional fire proofing required, higher chance of vibrations	Increased depth, largest weight system, formwork required	Increased weight, formwork required
Feasible for Redesigning	N/A	NO	YES	YES

# Appendix





- 1 LEVEL 04 FRAMING PLAN**  
1/8" = 1'-0"
1. FOR GENERAL NOTES AND ABBREVIATIONS, SEE S-0.01 AND S-0.02.
  2. TOP OF SLAB ELEVATION 2530'-0" UNLESS NOTED OTHERWISE.
  3. 1'-0" FROM ELEVATION 2530'-0" UNLESS NOTED OTHERWISE.
  4. TOP OF STEEL (4'-10") BELOW TOP OF SLAB ELEVATION 2530'-0" UNLESS OTHERWISE NOTED (+ OR - FROM ELEVATION 2530'-0").
  5. INDICATES SPAN DIRECTION OF COMPOSITE FLOOR SLAB. SEE SCHEDULE ON DRAWING S-5.02 FOR SLAB REQUIREMENTS.
  6. INDICATES SLAB/DECK OPENING SET EDGE OF SLAB AT 6" UNLESS NOTED OTHERWISE. SEE DRAWING S-3.12 FOR BEAM SIZES IF NOT INDICATED ON PLAN.
  7. INDICATES DIMENSIONS TO BE COORDINATED DURING CONSTRUCTION WITH APPROVED EQUIPMENT.
  8. LS INDICATES STEEL DECK SUPPORT PER 15-5-5.02.





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General Information:

=====

File Name: untitled.col  
 Project: Tech Report 3  
 Column: D6  
 Code: ACI 318-11  
 Engineer: MAC  
 Units: English  
 Run Option: Design  
 Run Axis: X-axis  
 Slenderness: Not considered  
 Column Type: Structural

Material Properties:

=====

f'c = 4 ksi  
 Ec = 3605 ksi  
 Ultimate strain = 0.003 in/in  
 Beta1 = 0.85  
 fy = 60 ksi  
 Es = 29000 ksi

Section:

=====

Rectangular: Width = 24 in  
 Depth = 24 in  
 Gross section area, Ag = 576 in^2  
 Ix = 27648 in^4  
 rx = 6.9282 in  
 Xo = 0 in  
 Iy = 27648 in^4  
 ry = 6.9282 in  
 Yo = 0 in

Reinforcement:

=====

Bar Set: ASTM A615  

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Bar selection: Minimum number of bars  
 Asmin = 0.01 \* Ag = 5.76 in^2, Asmax = 0.08 \* Ag = 46.08 in^2

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.  
 phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular  
 Pattern: All Sides Equal (Cover to transverse reinforcement)  
 Total steel area: As = 31.20 in^2 at rho = 5.42%  
 Minimum clear spacing = 2.31 in

20 #11 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:

=====

Design/Required ratio PhiMn/Mu >= 1.00

No.	Pu kip	Mux k-ft	PhiMnx k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	450.00	0.00	1028.33	999.999	11.07	21.30	0.00277	0.710

\*\*\* End of output \*\*\*