Letter of Transmittal

October 18, 2013

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Dear Professor Sustersic,

This report was written to fulfill the third of five technical report requirements set by the structural faculty for Penn State's Senior Thesis Capstone Project. This report, Technical Report 3, was assigned September 27, 2013.

Technical Report 3 has two main goals. First, I evaluate the adequacy of the members in an existing typical floor bay under gravity loading. Secondly, I compare three alternative gravity framing systems to the original system and discuss each system's advantages and disadvantages. The scope of this report is limited to gravity framing subject to gravity loads only. Typical bay dimensions of the original design are maintained for the analysis of the subsequent floor systems to generate a fair comparison that allows their validity as design solutions to the building's geometry and loading to be explored. Finally, the findings of this report are documented in a calculations binder and presented.

Thank you in advance for reviewing this report and the accompanying presentation. I look forward to hearing your feedback.

Sincerely,

Natasha Beck
Structural Option
Architectural Engineering Thesis Student

Enclosed: Technical Report 3

Reinsurance Group of America (RGA) Global Headquarters

Technical Report 3

Gravity Member Spot Checks & Alternate Bay Systems

16600 Swingley Ridge Rd.
Chesterfield, MO

Natasha Beck, Structural
Heather Sustersic
18 October 2013

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General Information

This section provides background information for RGA Global Headquarters.

Reinsurance Group of America (RGA) Global Headquarters

16600 Swingley Ridge Rd. Chesterfield, MO

Project Team

Owner: Reinsurance Group of America, Inc.
Owner Representative: Gateway Ridge LLC
General Contractor: Clayco
Architect: Gensler
Structural Engineer: Uzun & Case
Civil Engineer: Stock & Associates, Inc.
Landscape Architect: Forum Studio
Lighting Consultant: Randy Burkett Lighting Design, Inc.
MEP & Fire Protection: Environmental Systems Design, Inc.

Building Information

Occupancy: General office and training
Size: 405,000 gross square feet
Total Estimated Cost: $150 million
Project Delivery: Design-Build

Architecture

- Two skewed, 5 story office towers with curtain wall façades are linked by an amenities level
- Open plan office towers with a central core maximizing circulation, flexibility and daylight
- Amenities include kitchen and seating, fitness center, café and landscaped terrace
- Two story underground parking garage with limestone façade where it is exposed
- Three landscaped bio-retention basins
- Designed to achieve LEED Silver

Structural

- Two, 5 story steel office towers with composite floors with 3 1/2" semi-lightweight concrete topping
- Upper four levels cantilever 40' over the first level and is supported by a steel truss and plate girder system
- Office towers have a braced frame lateral system while the parking garage utilizes reinforced concrete shear walls
- Parking garage is post-tensioned, reinforced concrete
- Drilled concrete piers 36" to 78" in diameter with an allowable end bearing pressure of 80 ksf

Mechanical

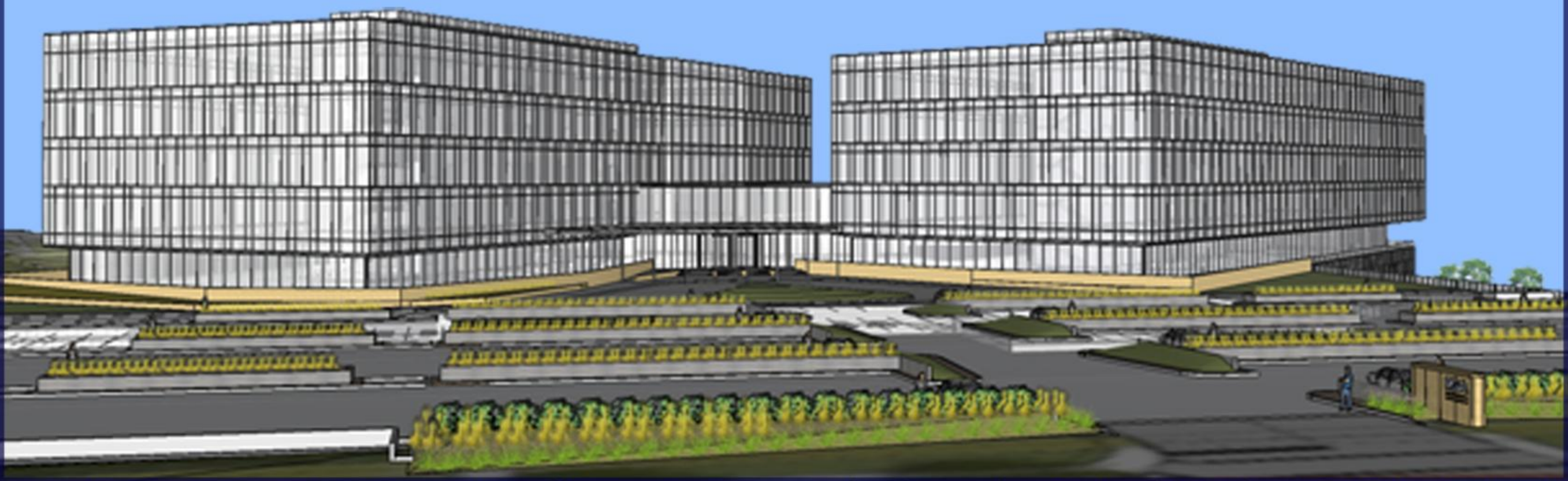
- Designed for year-round cooling
- Three, 350 ton water cooled chillers serviced by cooling towers
- Four 60,000 CFM air handling units serve the office towers
- A medium pressure loop is provided on each floor for VAV branches as needed for flexibility
- Separate fan powered terminal units (FPTU) heat the floor cavity of the cantilever space to counteract a potential heat sink

Lighting & Controls

- Occupancy sensors in restrooms
- Exterior and restroom lighting fixtures on 277 volts
- Fluorescent lamps and LED lamps specified to date
- Interior lighting design is currently in the final design stages

Electrical

- Mechanical and lighting serviced by a 480/277 volt system
- Office receptacles are serviced by 208/120 volt system
- Both systems are fed by 3-phase, 4-wire buses
- Four main switchboards (MSWBD) are rated at 3000 amperes
- Emergency equipment is supported by a diesel engine generator



Executive Summary

The purpose of this technical report is to evaluate the existing structural systems in the Reinsurance Group of America's Global Headquarters. This included preliminary analysis of the gravity and lateral systems and any unique structural features of the project. It looks at the main structural components and their influence on the load paths for wind, seismic, soil and gravity, which influence the main structural systems.

This preliminary research was executed by reviewing project documents, primarily drawings, and tracking these systems throughout the buildings. Findings of the systems' functionality and influence on other pieces of the project were then recorded and supporting information compiled into the body of this report.

In conclusion, critical structural features that will influence future analysis are the 40' cantilever truss system and maintaining the integrity of the soil load path so that it does not redistribute into the post-tensioned slabs.

Site Plan

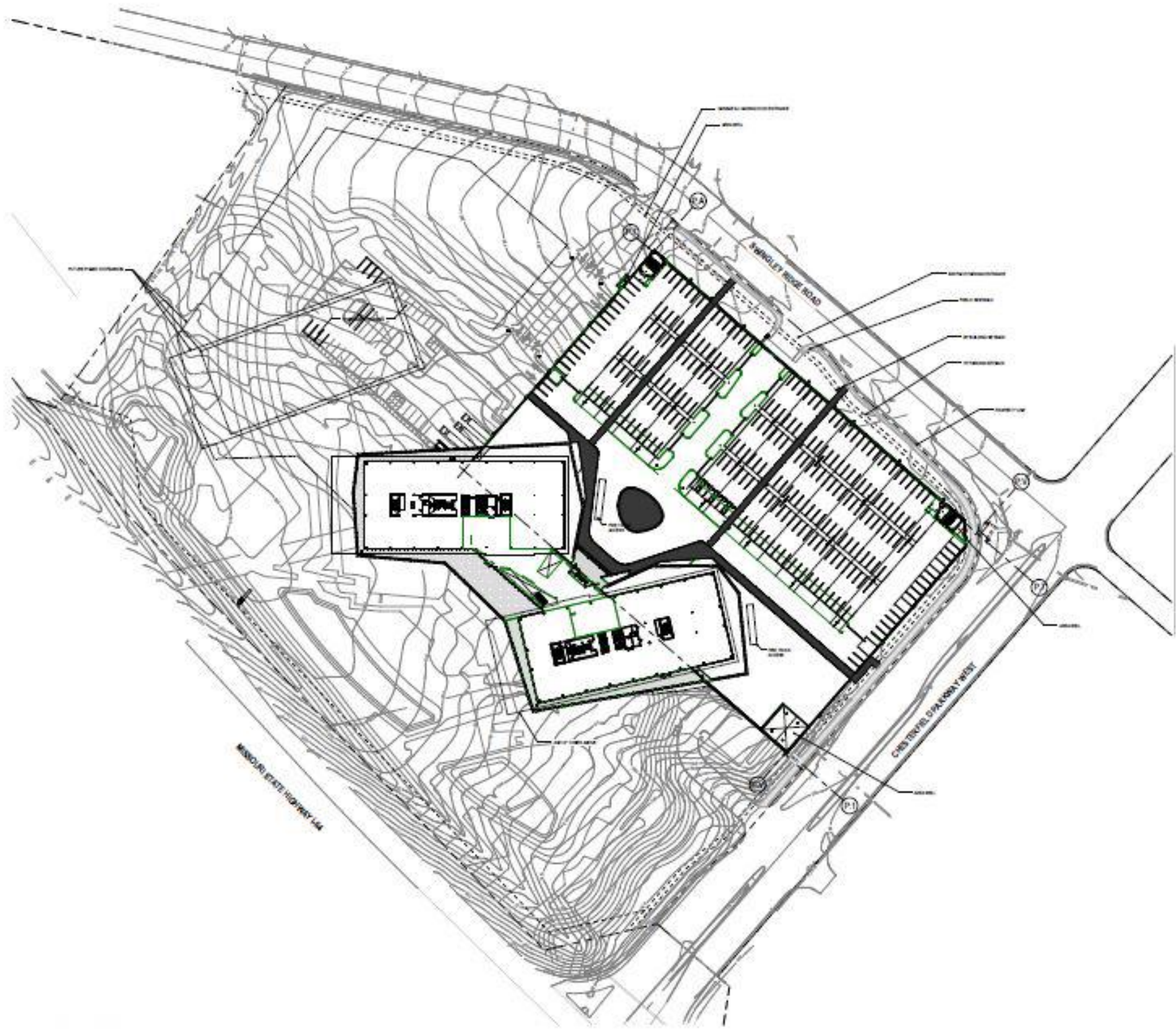
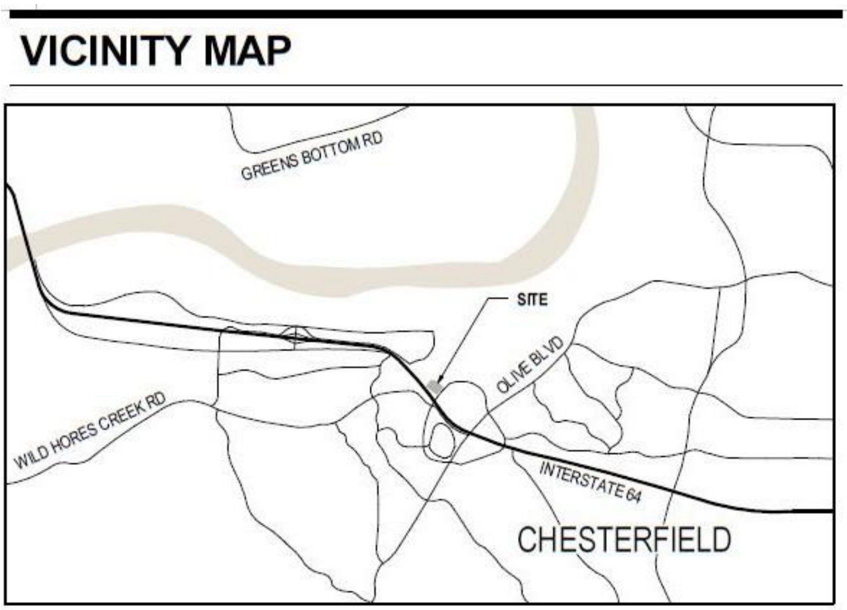


Figure 1: RGA Global Headquarters Site Plan by Gensler

Vicinity and Location Plans



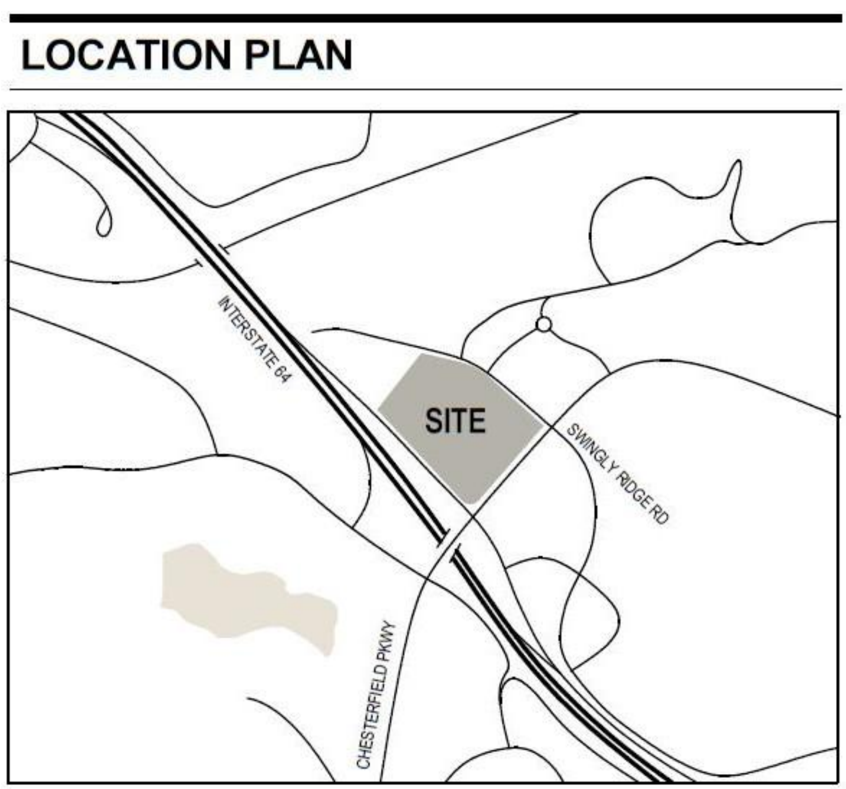


Figure 2: Vicinity and Location Plans by Gensler

Documents List

Listed below are the documents used in preparation of Technical Report 3.

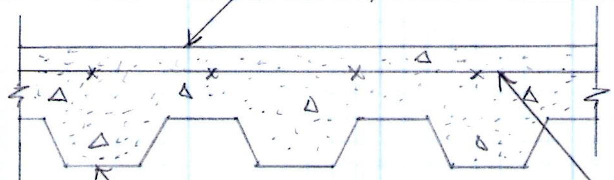
- *RGA Core and Shell Addendum A* Design Documents by the Project Team (See Abstract)
- Minimum Design Loads for Buildings and Other Structures, ASCE 7-05
- AISC Steel Construction Manual, AISC 360-10
- Vulcraft Composite Deck Tables
- Vulcraft Steel Roof and Floor Deck Tables
- Vulcraft Steel Joist Tables
- Nitterhouse Hollow Core Load Tables
- *Reinforced Concrete Mechanics & Design* by Wight and MacGregor
- PCI Handbook, 6th Ed.
- *RS Means Assemblies Cost Data 2013*
- Design of Steel Structures Class Notes
- Design of Masonry Structures Class Notes
- Design of Concrete Structures Class Notes

Gravity Load Revisions

This section presents revisions to the gravity load determination.

GRAVITY LOAD REVISIONS

- FLOOR LOADS



- DEAD LOAD

CONC: 116 PCF (3.5"/12) = 33.8 PSF
DECK: 2.14 PSF
SDL: 20 PSF
Total: 56 PSF

FRAMING ALLOWANCE:

30'(62+94) = 4680#
45'(4)(44) = 7920#
14'(2)(106) = 2968#
14'(2)(53) = 1484#
Total: 17052#

$\frac{17052\#}{30' \times 45'} = 13 \text{ PSF}$

FLOOR DL = 56+13 = **69 PSF**
LIVE LOAD = **50 PSF** [ASCE 7-05 TABLE 4-1]

- CURTAIN WALL LOADS

(187 PLF + 18 PLF)(1.03) = **211 PLF**

↑ GLASS ↑ INSUL. ↑ CONNECTIONS ALLOWANCE

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TITLE: GRAVITY LOAD REVISIONS FLOOR + CURTAIN WALL
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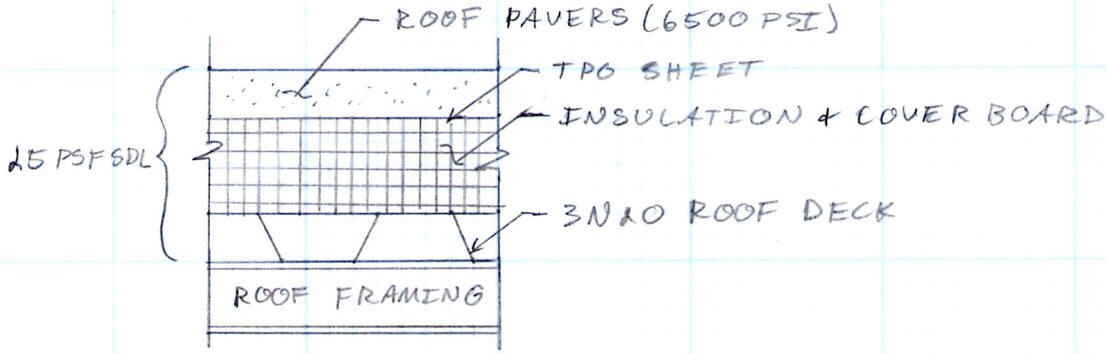
• ROOF LOADS



- DEAD LOAD

25 PSF + 13 PSF = **38 PSF DL**
(25 PSF = SDL; 13 PSF = FRAMING)

- LIVE LOAD

L_r = 20 PSF *

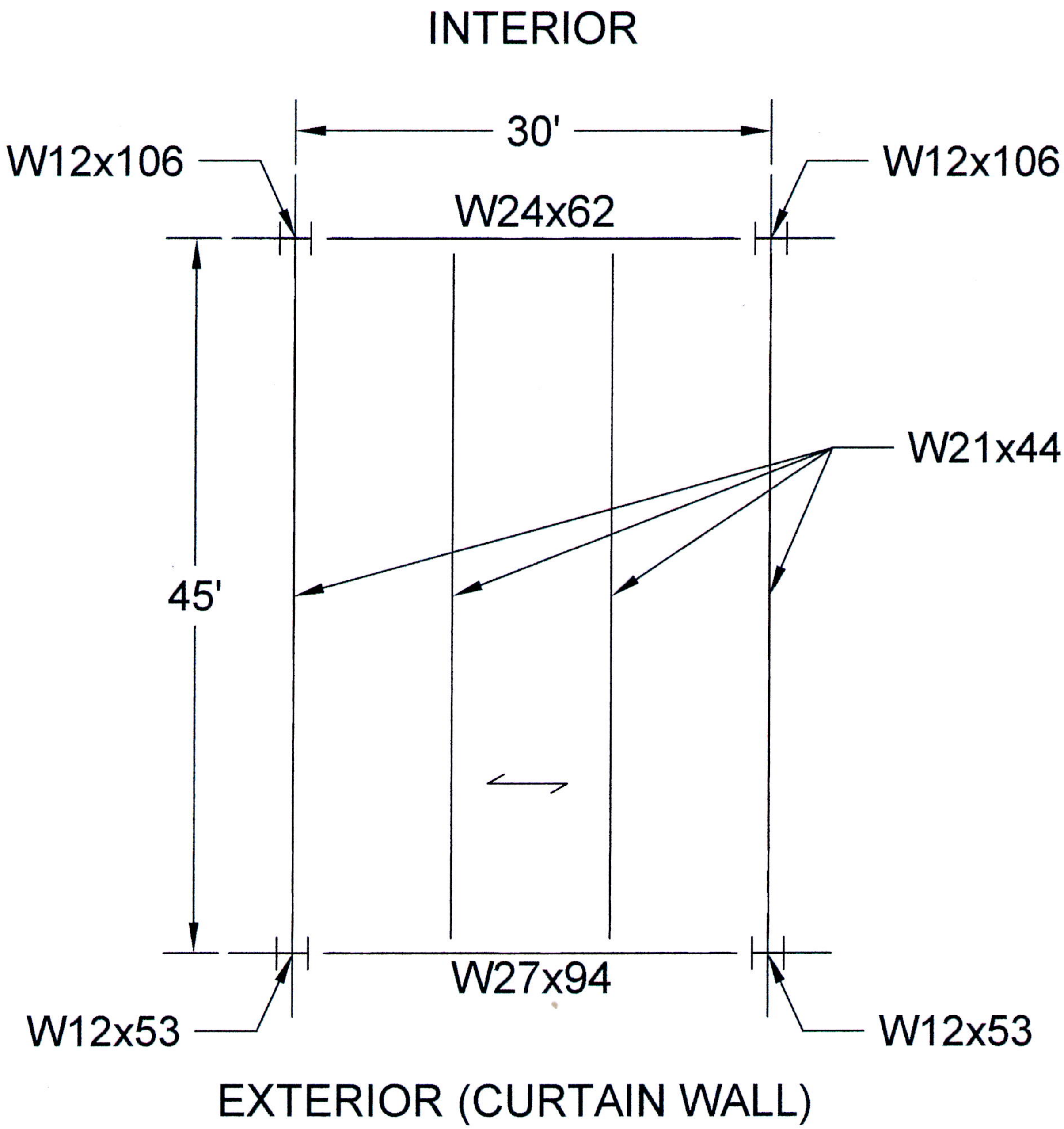
- SNOW LOAD

P_f = 22 PSF *

*WERE NOT REVISED - REPEATED FOR CLARITY

Existing System: Composite Steel

This section presents member evaluations of the existing composite steel system for a typical bay.



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NOTES:
Fy=50 KSI
Es=29,000 KSI
2 HR FIRE RATING
CAMBER=1 1/2" OR 1 3/4"

TYPICAL BAY-ORIGINAL SYSTEM
SCALE: 3/32"=1'-0"

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EXISTING SYSTEM GRAVITY CHECKS

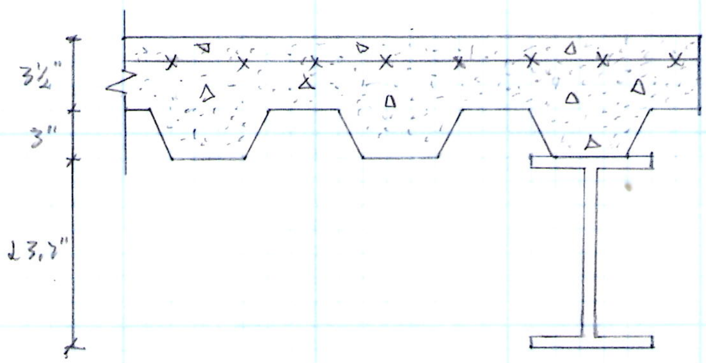
CHECK DECKING

- 3.0SB20 COMPOSITE DECK, 3½" 3000 PSI SEMI-LTWT CONC. TOPPING (116 PCF) => 3.0 VLI20 IS EQUIVALENT

- CHECK DECK SPAN
  - 3 SPAN CONDITION [S0.001]
  - FROM VULCRAFT COMPOSITE DECK TABLES
    11'-9" > 10'-0" <u>OK</u> UNSHORED

- CHECK CAPACITY
  - LL = 50 PSF [OFFICE LL ASCE 7-05]
    MAX 80 PSF [LL CORRIDORS ABOVE 1st FLR ASCE 7-05]
  - SDL = 20 PSF, 13 PSF FRAMING ALLOWANCE
  - SUPERIMPOSED LOAD: 80 + 20 + 13 = 113 PSF
  - AT 10'-0": 158 PSF ALLOWABLE > 113 PSF <u>OK</u>

SYSTEM DEPTH



TOTAL DEPTH: 30" = 2'-6"

FIRE RATING

- FROM VULCRAFT FLOOR-CEILING ASSEMBLIES WITH COMPOSITE DECK (pg 70-71)
  - UNPROTECTED DECK
  - 3½" LW TOPPING => 2 HOUR
  - 3 VLI DECK (EQUIV.)

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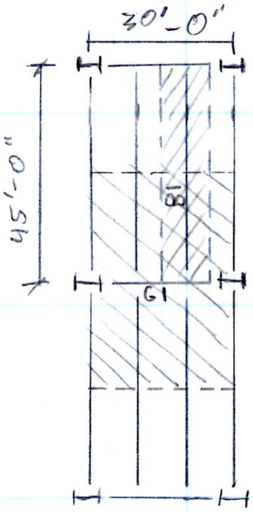
CHECK BEAMS



B1: W21x44 [26 STUDS] c = 1½" or 1¾"

$A_s = 13.0\ in^2$
$b_f = 6.50"$
$d = 20.7"$
$I_x = 843\ in^4$

LOADING: LL = 50 PSF (REDUCIBLE)
SLAB & DECK = 36 PSF
SDL = 20 PSF
SELF WT = 5 PSF

RELEVANT LOAD CASES
1.4D
1.2D+1.6L

$w_{MAX} = \left| \begin{array}{l} 1.4(36+20+5) = 85\ PSF \\ 1.2(36+20+5)+1.6(37.5) = 133\ PSF \end{array} \right.$

W = 133 PSF (10') = **1.33 KLF = W**

- LIVE LOAD REDUCTION
  $L_o = 50\ PSF$
  $A_T = 10'(45') = 450SF > 400SF$ ∴ REDUCIBLE
  $K_{LL} = 2$ B/C INTERIOR BEAM [ASCE 7-05 TABLE 4-2]

  $L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$ [ASCE 7-05 EQN 4-1]

  $L = 50\left(0.25 + \frac{15}{\sqrt{2 \cdot 450}}\right) = 37.5\ PSF = LL$

- DESIGN MOMENT
  $M_u = \frac{wl^2}{8} = \frac{(1.33\ klf)(45')^2}{8} = \mathbf{337\ FT \cdot K = M_u}$

- FIND b_{eff}
  $2 \times MIN \left| \begin{array}{l} SPAN/8 = 45'/8 = 5.625' \\ \tfrac{1}{2}(TRIB\ WIDTH) = 10/2 = 5' \end{array} \right.$ ∴ $b_{eff} = 10'$

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- FIND ΣQ_n

$$Q_n = \min \begin{cases} 0.5 A_{sc} \sqrt{f'_c E_c} \\ A_{sc} F_u \end{cases}$$

- 3/4" Ø, STRONG POSITION SHEAR STUDS
  - $A_{sc} = \pi (0.375)^2 = 0.4418 \text{ in}^2$
  - $F_u = 65$ ksi
  - $E_c = w_c^{1.5} \sqrt{f'_c (ksi)} = (116 \text{ pcf})^{1.5} \sqrt{3 \text{ ksi}} = 2164$ ksi
  - $R_p = 0.75$ [STRONG POSITION, $d/5 \geq 0.005$]
  - $\frac{w_r}{h_r} = \frac{4.75}{3} = 1.58 > 1.5$

$\therefore R_g = 1.0$

$$Q_n = \min \begin{cases} 0.5(0.4418 \text{ in}^2)\sqrt{3 \cdot 2164} = 17.8^k \\ 0.4418(65) = 28.7^k \end{cases} \quad \therefore Q_n = 17.8^k$$

$$\boldsymbol{\Sigma Q_n = 26(17.8^k) = 463^k}$$

- DETERMINE ϕM_n

$$\min \begin{cases} C_{cmax} = 0.85 f'_c b_{eff} t = 0.85(3)(10 \cdot 12)(3.5) = 1071^k \\ T_{smax} = A_s F_y = (13)(50) = 650^k \\ \Sigma Q_n = 463^k \Rightarrow \text{CONTROLS} \end{cases}$$

$\Sigma Q_n < T_{smax} < C_{cmax}$

$\therefore$ PARTIALLY COMPOSITE

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{463}{0.85(3)(120")} = \mathbf{1.51"}$$

$$x = \frac{A_s F_y - \Sigma Q_n}{2 F_y b_f} = \frac{650 - 463}{2(50)(6.5)} = 0.288$$

$$M_n = A_s F_y \left(\tfrac{d}{2}\right) + \Sigma Q_n \left(t - \tfrac{a}{2}\right) - 2 F_y b_f x \left(\tfrac{x}{2}\right)$$

$$= 650\left(\tfrac{20.7}{2}\right) + 463\left(3.5 - \tfrac{1.51}{2}\right) - 2(50)(6.5)(0.288)\left(\tfrac{0.288}{2}\right)$$

$M_n = 7970$ ft·k

$\phi = 0.9$

$\boldsymbol{\phi M_n = 7170}$ **ft·k**

$\boldsymbol{\phi M_n} = 7170$ ft·k $> \boldsymbol{M_u} = 337$ ft·k $\therefore$ **OK** FOR STRENGTH

- CHECK UNSHORED STRENGTH

$W21\times44 \Rightarrow \phi M_p = 358$ ft·K [AISC 360-10 TABLE 3-2]

$w_u = 1.4(\text{DECK}+\text{CONC})W_T + 1.4(\text{BM SELF})$

$= 1.4(2.14+33.8)(10') + 1.4(44) = 0.565$ klf

$w_u = 1.2(36\cdot10+44) + 1.6(20)(10) = 0.805$ klf ⇒ CONTROLS

CONST. LL

$$M_u = \frac{(0.805)(45^2)}{8} = 204 \text{ ft}\cdot\text{K}$$

$\phi M_p > M_u$ ∴ **NO SHORING REQ'D**

- CHECK WET CONCRETE DEFLECTION

$W_{wc} = 36\cdot10+44 = 0.404$ klf

$$\Delta_{wc} = \frac{5wL^4\cdot1728}{384EI} = \frac{5(0.404)(45^4)(1728)}{384(29000)(843)} = 1.52''$$

$$\Delta_{wc\,MAX} = \frac{L}{240} = \frac{45\cdot12}{240} = \mathbf{2.25'' > 1.52''} \therefore \mathbf{OK}$$

- CHECK LIVE LOAD DEFLECTION

$\Sigma Q_n = 463^K$

$$Y_2 = 3.5 - \frac{1.51}{2} = 2.75$$

ΣQ_n	$Y_2 = 2.5$	$Y_2 = 2.75$	$Y_2 = 3.0$
504	1780		1850
463	1735	1770	1805
431	1700		1770

[AISC 360-10 TABLE 3-20]

$I_{LB} = 1770 \text{ in}^4$

$W_{LL} = 37.5(10) = 0.375$ klf

$$\Delta_{LL} = \frac{5(0.375)(45^4)(1728)}{384(29000)(1770)} = 0.674''$$

$\Delta_{LL\,MAX} = L/360 = 45/360 = 1.5''$

$\Delta_{LL} < \Delta_{LL\,MAX}$ ∴ OK

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- CHECK TOTAL LOAD DEFLECTION

$W_{TL} = 1.33$ KLF

$$\Delta_{TL} = \frac{5(1.33)(45^4)(1728)}{384(29000)(1770)} = 2.39''$$

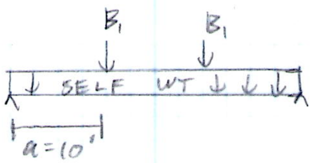
$$\Delta_{TL\,MAX} = L/240 = \frac{45 \cdot 12}{240} = 2.25'' < 2.39'' \quad \therefore \textbf{ CAMBER}$$

- CAMBER SPECIFIED AT 1 ½" AND 1 ¾"

CHECK GIRDER

W24x62: $A_s = 13.0\ in^2$ $\quad d = 23.7''$

[27 STUDS] $b_f = 7.04\ in$ $\quad I_x = 1550\ in^4$



$B_1 = 1.33$ klf $(45') = 59.9^k$

↳ PREVIOUSLY DETERMINED

$W_u = 1.2(62)/1000 = 0.0744$ klf

- DESIGN MOMENT

$$M_u = \frac{wL^2}{8} + B_1 a$$

$$= \frac{0.0744(30^2)}{8} + 59.9(10) = \mathbf{607\ ft \cdot k = M_u}$$

- FIND b_{eff}

$$b_{eff} = 2 \times \min \begin{cases} SPAN/8 = 30(12)/8 = 45'' \\ \frac{1}{2}(12)(TRIB\ WIDTH) = \frac{1}{2}(12)(45') = 270'' \end{cases}$$

$$\therefore 2(45) = \mathbf{90'' = b_{eff}}$$

- FIND ΣQ_n
  - $Q_n = 17.8^k$ (DETERMINED PREVIOUSLY)

$$\mathbf{\Sigma Q_n} = 17.8(27) = \mathbf{481^k}$$

- DETERMINE ϕM_n

$C_{c\,MAX} = 0.85 f'c\, b_{eff}\, t = 0.85(3)(90)(3.5) = 803^k$

$T_{s\,MAX} = A_s F_y = 13(50) = 650^k$

$\Sigma Q_n = 481^k \Rightarrow$ CONTROLS

$\Sigma Q_n < T_{s\,MAX} < C_{c\,MAX} \quad \therefore$ PARTIALLY COMPOSITE

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$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{481}{0.85(3)(90")} = 2.10"$$

$$x = \frac{A_s F_y - \Sigma Q_n}{2 F_y b_f} = \frac{650 - 481}{2(50)(7.04")} = 0.240$$

$$M_n = A_s F_y (d/2) + \Sigma Q_n (t - a/2) - 2 F_y b_f x (x/2)$$
$$= 650(23.7/2) + 481(3.5 - 2.10/2) - 2(50)(7.04)(0.240)(\tfrac{0.240}{2})$$

$M_n = 8861$ ft·k

$\phi = 0.9$

8861 ft·k (0.9) = **7975 ft·k** = ϕM_n

ϕM_n = 7975 ft·k > M_u = 607 ft·k ∴ **OK** FOR STRENGTH

• CHECK UNSHORED STRENGTH

W24x62: ϕM_p = 574 ft·k [AISC 360-10 TABLE 3-2]

P_u = [1.2 (DECK + CONC) w_T + 1.2 (BM. SELF) + 1.6 (CONST. LL)] L

= [1.2(2.14 + 33.8)(45') + 1.2(62) + 1.6(20)(45')] (30)/1000

P_u = 104 k CONTROLS

P_u = [1.4 (2.14 + 33.8)(45) + 1.4(62)] (30')/1000 = 70.5 k

$M_u = P_u a$ = 104 (10') = 1040 ft·k

$\phi M_p \not> M_u$ ∴ **SHORING REQ'D**

• CHECK LIVE LOAD DEFLECTION

ΣQ_n = 481 k

$Y_2 = 3.5 - 2.1/2 = 2.45"$

ΣQ_n	$Y_2 = 2.0$	$Y_2 = 2.45$	$Y_2 = 2.5$
495	2780		2870
481	2755	2834	2843
361	2540		2610

[AISC 360-10 TABLE 3-20]

∴ I_{LB} = 2834 in⁴

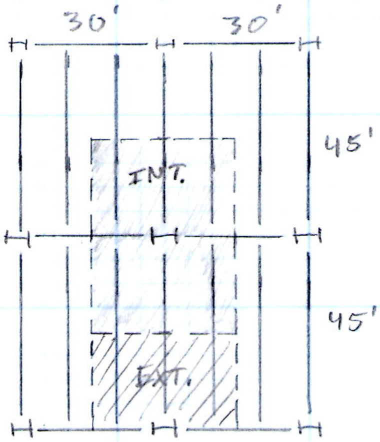
w_{LL} = 37.5 (45')/1000 = 1.69 klf

$$\Delta_{LL} = \frac{5 w L^4 (1728)}{384 E I_{LB}} = \frac{5(1.69)(30^4)(1728)}{384(29000)(2834)} = 0.375"$$

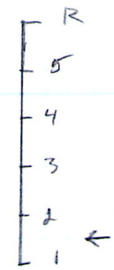
$\Delta_{LL\,MAX}$ = L/360 = 30·12/360 = 1.0"

$\Delta_{LL} < \Delta_{LL\,MAX}$ ∴ OK

CHECK INTERIOR COLUMN



INT: W12×106
EXT: W12×53



- LIVE LOAD REDUCTION

  $A_T = (45')(30') = 1350\ SF > 400\ SF$ ∴ REDUCE

  $K_{LL} = 4$ [ASCE 7-05 TABLE 4-2]

  - FLOOR: $L_o = 50\ PSF$

    $L = L_o \left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$ [ASCE 7-05 EQN. 4-1]

    $L = 50\left(0.25 + \frac{15}{\sqrt{4 \cdot 1350}}\right) = \mathbf{22.7\ PSF}$ CONTROLS

    $L_{MIN} = 0.4 L_o = 0.4(50) = 20\ PSF$

  - ROOF:

    $L_r = \max \begin{cases} L_r = 20\ PSF \\ S = 22\ PSF \Rightarrow \text{CONTROLS} \end{cases}$

- DEAD LOADS
  - FLOOR: $D = 69\ PSF$
  - ROOF: $D_r = 38\ PSF$

- LOAD COMBINATIONS

  1.4D ⇒ DOESN'T CONTROL BY ISPECTION

  1.2D + 1.6L + 0.5S

  1.2D + 1.6S + 0.5L

  THE CONTROLLING CASE WILL HAVE THE LARGER FACTOR ON THE LARGER LOAD: L > S

  ∴ 1.2D + 1.6L + 0.5S CONTROLS

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• DESIGN LOAD

$DL = (4\ FLOORS)(69\ PSF)(1350\ SF) + (38\ PSF)(1350\ PSF) = 424^K$

$LL = (4)(22.7)(1350) = 123^K$

$SL = (1)(22)(1350) = 29.7^K$

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

$1.2(424) + 1.6(123) + 0.5(29.7^K) = \mathbf{720^K = P_u}$

• CHECK STRENGTH

- UNBRACED LENGTH, $L = 15.3'$
- PIN-PIN: $K = 1.0$ [AISC 360-10 TABLE C-A-7.1]
- $KL = 15.3' \rightarrow$ TABLE USE $16'$
- $\mathbf{\phi P_n = 1060^K}$ [AISC 360-10 TABLE 4-1]

$\phi P_n = 1060^K > P_u = 720^K \quad \therefore$ OK

CHECK EXTERIOR COLUMN

• LIVE LOAD REDUCTION

$A_T = (30')(22.5') = 675\ SF > 400\ SF \quad \therefore$ REDUCE

$K_{LL} = 4$ [ASCE 7-05 TABLE TABLE 4-2]

$L = 50\left(0.25 + \frac{15}{\sqrt{4 \cdot 675}}\right) = 27.0\ PSF > 20\ PSF\ MIN.$

• OTHER LOADS

$S = 22\ PSF$

FLOOR: $D = 69\ PSF$

ROOF: $D_r = 38\ PSF$

} SAME AS INT. COLUMN

CURTAIN WALL LOAD: 211 PLF ALONG 30' DIRECTION

• LOAD COMBINATION $1.2D + 1.6L + 0.5S$ CONTROLS

• DESIGN LOAD

$DL = (4\ FLRS)(69)(675) + (38)(675) + (211)(30') = 218^K$

$LL = (4)(27)(675) = 72.9^K$

$SL = (1)(22)(675) = 14.9^K$

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

$= 1.2(218) + 1.6(72.9) + 0.5(14.9^K) \Rightarrow \mathbf{P_u = 386^K}$

• CHECK STRENGTH

- UNBRACED LENGTH = 15.3'
- PIN - PIN : K = 1.0 [AISC 360-10 TABLE C-A-7.1]
- KL = 15.3' → TABLE USE 16'
- $\phi P_n = 453^k$ [AISC 360-10 TABLE 4-1]

$\phi P_n = 453^k > P_u = 386^k \quad \therefore$ OK

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PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 3

TITLE: EXISTING SYSTEM SYSTEM COST

BY: NMB

SHEET: S.11

CHKD:

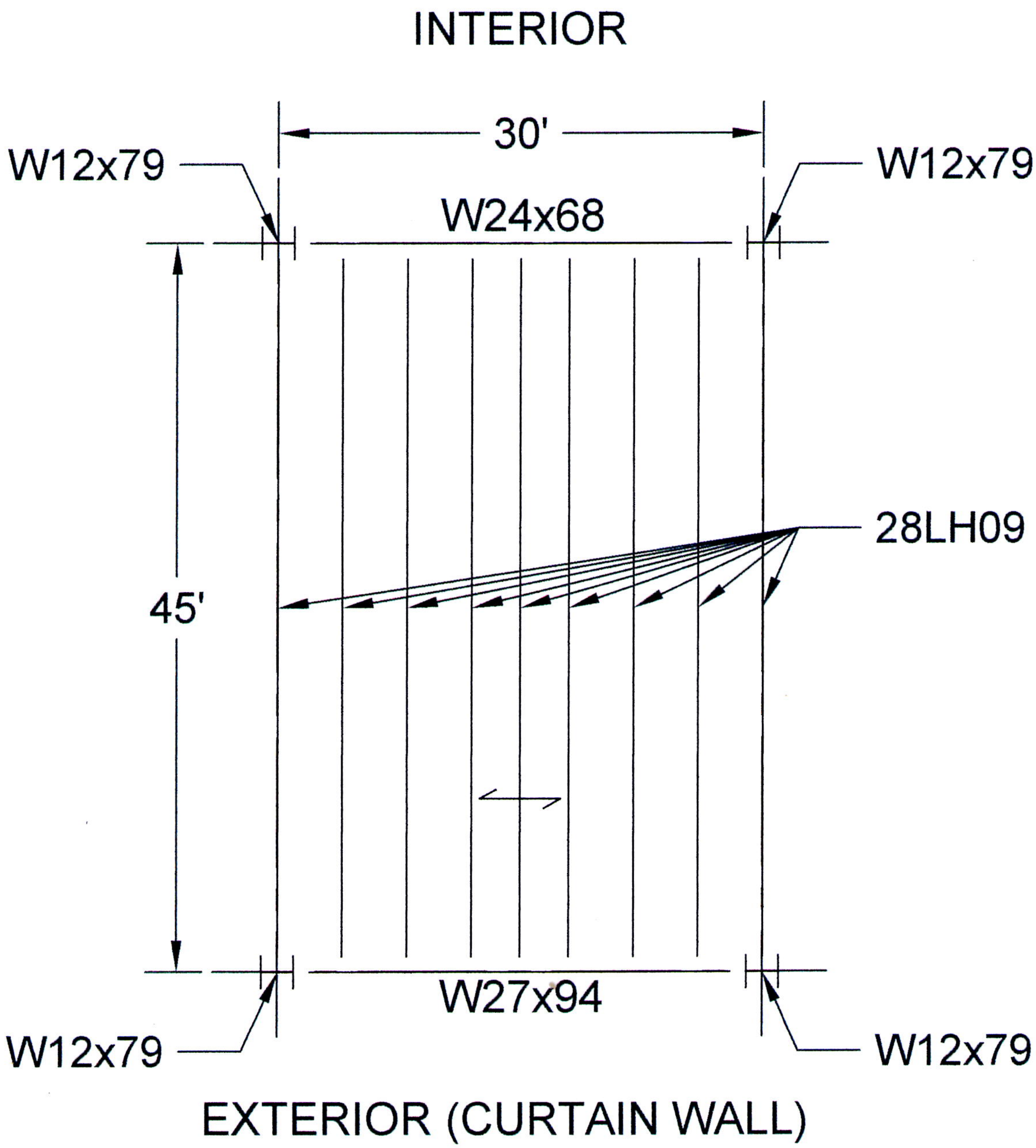
PROJECT NO: TECH 3

DATE:

PAGE:

Alternate System 1: Floor Joist System

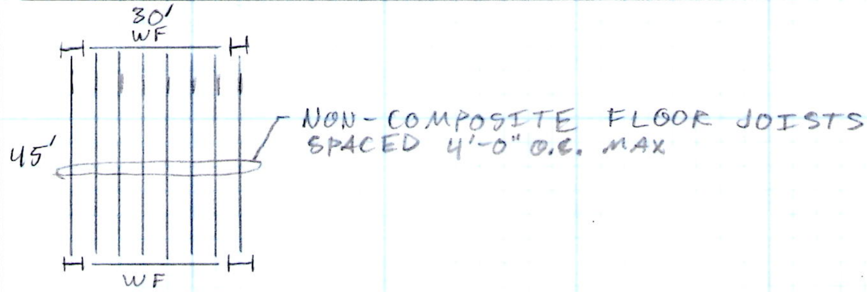
This section presents schematic level member sizes and evaluation of the typical bay of floor joists.



NOTES:
Fy=50 KSI
E=29,000 KSI
SPACING 4'-0" MAX. O.C.

TYPICAL BAY-FLOOR JOIST SYSTEM
SCALE: 3/32"=1'-0"

ALTERNATE SYSTEM: FLOOR JOISTS



- K SERIES JOISTS SPAN UP TO 60' > 45'
- TRY 4'-0" O.C. MAX. SPACING
- FIND SUPERIMPOSED LOAD

  DL = 56 PSF

  LL = 50 PSF

  $1.2D + 1.6L + 1.2W_T = W_{utl}$ WHERE W_T : JOIST WEIGHT

  $1.2(56)(4') + 1.6(50)(4') + 1.2W_T = W_{uTl}$

  $589 \text{ PLF} + 1.2W_T = W_{uTl}$

  $D + L + W_T = W_{Tl}$

  $(56+50)(4') =$ **$424 \text{ PLF} + W_T = W_{Tl}$**

- FROM ECONOMY TABLES [VULCRAFT APP. C]

  AT 45' LARGEST CAPACITY = 583 PLF < 589 PLF

- TRY LONG SPAN JOISTS; LH SERIES
  - FROM LH TABLES: 24LH08

    $W_{nTl} = 625 \text{ PLF} > 589 \text{ PLF} + 1.2(18 \text{ PLF}) = 611 \text{ PLF}$ OK

    $W = 208 \text{ PLF}$ FOR $L/360$

    $L/240 : 208(1.5) = 312 \text{ PLF} \not> 424 \text{ PLF}$ NG

  - **28LH09**

    $W_{ntl} = 844 \text{ PLF} > 614 \text{ PLF} \ [589 + 1.2(21 \text{ PLF})]$ OK

    $W = 329 \text{ PLF}$ FOR $L/360$

    $L/240 = 329(1.5) = 493 \text{ PLF} > 424 + 21 = 445 \text{ PLF}$ OK

DESIGN GIRDERS

$J = 614 \text{ PLF}(45')/1000 = 27.6^k$

$J_{dist} = (27.6^k)(6)/30' = 5.52 \text{ KLF}$

$W_u = 5.52 \text{ KLF} + 1.2(45')(2 \text{ PSF})/1000 = \mathbf{5.63 \text{ KLF}}$

GIRDER SELF WT ↵

- DESIGN MOMENT

$$M_u = \frac{w l^2}{8} = \frac{(5.63)(30^2)}{8} = \mathbf{663\ 'k}$$

- TRIAL SIZE

W24 x 68: $\phi M_p = 664\ 'k$ [AISC 360-10 TABLE 3-2]

- CHECK DEFLECTIONS

$W_{LL} = 1.69$ KLF (PREVIOUS)

$\Delta_{LL MAX} = L/360 = 30 \cdot 12/360 = 1''$

$$\Delta_{LL} = \frac{5(1.69)(30^4)(1728)}{384(29000)(1830)} = 0.58'' \quad \underline{OK}$$

*ASSUMING W24x68 CONTINUALLY BRACED BY DECKING

$\phi M_n = 664\ 'k > M_u = 663\ 'k$ <u>OK</u>

∴ GIRDER IS **W24 x 68**

DESIGN COLUMN

- LIVE LOAD REDUCTION

$A_T = 1350$ SF $\quad L_o = 50$ PSF $\quad S = 22$ PSF

$K_{LL} = 4 \quad L = 22.7$ PSF

- SAME AS EXISTING

- DEAD LOADS

- NEW FRAMING ALLOWANCE

45'(21 PLF)(8) = 7560#
30'(2)(68 PLF) = 4080#
COLUMNS(PREVIOUS) = 4450#

} 16090#

$$\frac{16090}{(30')(45')} = 12 \text{ PSF}$$

- FLOOR: $D = 56\ PSF + 12\ PSF = 68\ PSF$
- ROOF: $D_r = 25\ PSF + 12\ PSF = 37\ PSF$

• DESIGN LOADS

$DL = (4\ FLRS)(68\ PSF)(1350\ SF) + (37\ PSF)(1350\ SF) = 417^K$

$LL = (4\ FLRS)(22.7)(1350) = 123^K$

$SL = (1)(22)(1350) = 29.7^K$

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

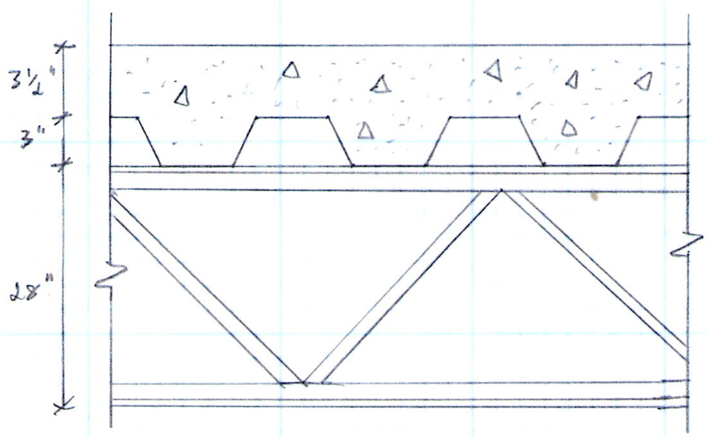
$1.2(417^K) + 1.6(123) + 0.5(29.7) = 712^K = P_u$

• SELECT COLUMN
- UNBRACED LENGTH = 15.3'
- PIN - PIN: K = 1.0
- KL = 15.3' → TABLE 16'
- FROM AISC 360-10 TABLE 4-1:

W12x79: $\phi P_n = 781^K$

$\phi P_n > P_u$ ∴ OK

SYSTEM DEPTH



TOTAL DEPTH: 34.5"

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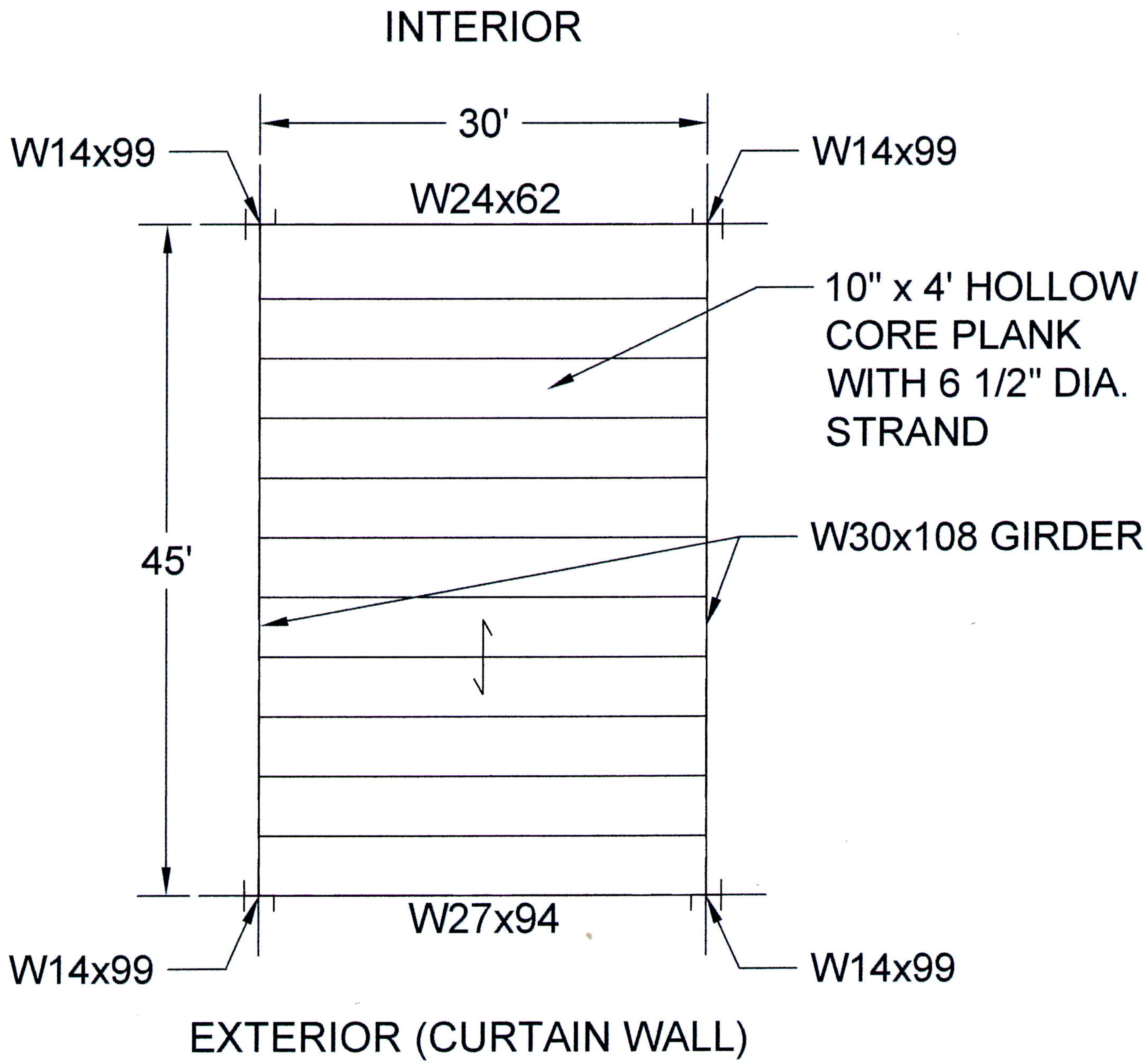
FLOOR JOIST SYSTEM COLUMN DESIGN		TECH 3
	DATE:	PAGE:

Alternate System 2: Hollow Core Plank System

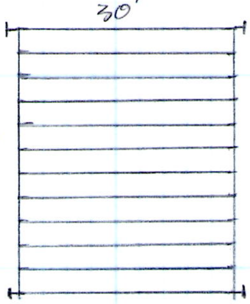
This section presents schematic level member sizes and evaluation of the typical bay of hollow core plank on steel framing.



NOTES:
Fy=50 KSI
Es=29,000 KSI
Ec=4696 KSI
2 HR FIRE RATING

TYPICAL BAY-HOLLOW CORE SYSTEM
SCALE: 3/32"=1'-0"

ALTERNATE SYSTEM: HOLLOW CORE PLANK



SYSTEM REQUIREMENTS
- 2" LEVELING TOPPING
- 2 HOUR FIRE RATING

• SUPERIMPOSED SERVICE LOAD

$1.2(20) + 1.6(50) = 104$ PSF (LRFD FOR COMPARISON)

• FROM NITTERHOUSE

- SPANNING 45': 16"x 4' SSL = 69 PSF < 104 PSF <u>NG</u>
- SPANNING 30':

  10"x4' 6 ½" Ø STRAND SSL = 128 PSF > 104 PSF <u>OK</u>

• DESIGN MOMENT

$W_u = 1.2(20+68+25)(4) + 1.6(50) = 622$ PLF

$$M_u = \frac{w l^2}{8} = \frac{0.622(30^2)}{8} = 70.0 \text{ k·ft}$$

$M_n = 168$ k·ft $\Rightarrow \phi M_n = 0.9(168) = 151$ k·ft

$\phi M_n > M_u$ ∴ <u>OK</u> FOR STRENGTH

• LIVE LOAD DEFLECTION

$$\Delta_{LL} = \frac{5 w_{LL} l^4 (1728)}{384 EI}$$

$E_c = 33 w^{1.5}\sqrt{f'c} = 33(150)^{1.5}\sqrt{6000} = 4696$ KSI

$W_{LL} = 1.6(50)(4')/1000 = 0.320$ KLF

$I_c = 5102 \text{ in}^4$

$$\Delta_{LL} = \frac{5(0.320)(30^4)(1728)}{384(4696)(5102)} = 0.24''$$

$$\Delta_{LL\,MAX} = L/360 = \frac{30 \cdot 12}{360} = 1.0''$$

$\Delta_{LL\,MAX} > \Delta_{LL}$ <u>OK</u>

• TOTAL LOAD DEFLECTION

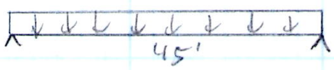
$W_u = 0.662$ KLF

$$\Delta_{TL} = \frac{5(0.662)(30^4)(1728)}{384(4696)(5102)} = \mathbf{0.504''}$$

$$\Delta_{TL\,MAX} = L/240 = 30 \cdot 12/240 = \mathbf{1.5''}$$

$\Delta_{TL\,MAX} > \Delta_{TL}$ OK

DESIGN GIRDERS



$P = (0.662\text{ KLF})(30')(11.25\text{ PLANKS})/45' = 4.97$ KLF

$W_u = 4.97\text{ PLF} + 1.2(2\text{ PSF})(30') = 5.04$ KLF

SELF WT ↲

• DESIGN MOMENT

$$M_u = \frac{wl^2}{8} = \frac{5.04(45^2)}{8} = \mathbf{1276\ 'k}$$

• TRIAL SIZE

W30x108 : $\mathbf{\varnothing M_p = 1300\ 'k}$ [AISC 360-10 TABLE 3-2]

- $W_{LL} = 37.5(30)/1000 = 1.13$ KLF

$$\Delta_{LL} = \frac{5(1.13)(45^4)(1728)}{384(29000)(4470)} = \mathbf{0.804''}$$

$$\Delta_{LL\,MAX} = L/360 = 45 \cdot 12/360 = \mathbf{1.5''}$$

$\Delta_{LL\,MAX} > \Delta_{LL}$ **OK**

$\varnothing M_n > M_u$ **OK** ∴ **GIRDER W30x108**

DESIGN COLUMNS

• LIVE LOAD REDUCTION

$A_T = 1350$ SF　　$L_o = 50$ PSF　　$S = 22$ PSF

$K_{LL} = 4$　　$L = 22.7$ PSF

- SAME AS EXISTING

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HOLLOW CORE SYSTEM GIRDER DESIGN		TECH 3
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• DEAD LOADS

- ASSUME 10 PSF STEEL FRAMING ALLOWANCE
- FLOOR: 20 PSF + 68 PSF + 25 PSF + 10 PSF = 123 PSF
  (↑ SDL, ↑ PLANK, ↑ TOPPING)
- ROOF: D_r = 25 PSF + 10 PSF = 35 PSF

• DESIGN LOADS

$DL = (4\ FLRS)(123)(1350) + (35)(1350) = 711^K$

$LL = (4)(22.7)(1350) = 123^K$

$SL = (1)(22)(1350) = 29.7^K$

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

$1.2(711^K) + 1.6(123^K) + 0.5(29.7^K) = P_u = 1065^K$

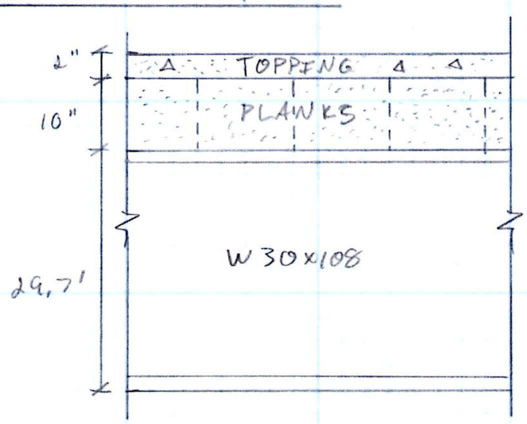
• SELECT COLUMN

- UNBRACED LENGTH = 15.3'
- PIN - PIN: K = 1.0
- KL = 15.3' → TABLE 16'
- FROM AISC 360-10 TABLE 4-1:

  $W14 \times 99: \phi P_n = 1080^K$

  $\phi P_n > P_u$ OK

SYSTEM DEPTH



TOTAL DEPTH: 42" = 3'-6"

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TITLE	HOLLOW CORE SYSTEM COLUMN DESIGN
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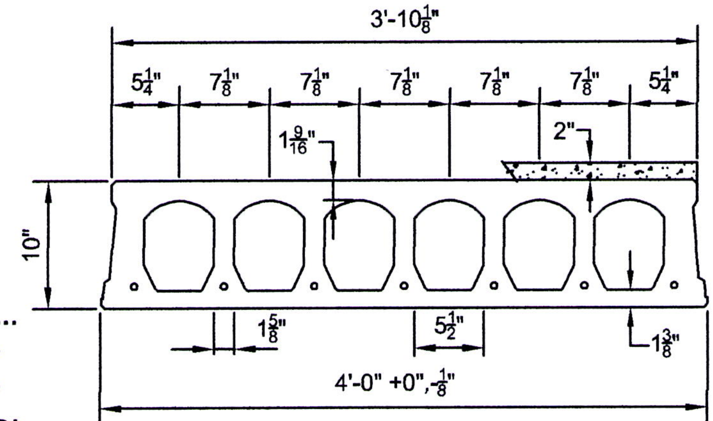
Prestressed Concrete
10"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2}	Precast b_w = 13.13 in.
I_c = 5102 in.^{4}	Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in.	Topping S_{tct} = 1242 in.^{3}
Y_{tcp} = 3.81 in.	Precast S_{tcp} = 1340 in.^{3}
Y_{tcp} = 5.81 in.	Precast Wt. = 272 PLF
	Precast Wt. = 68.00 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 191.7 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

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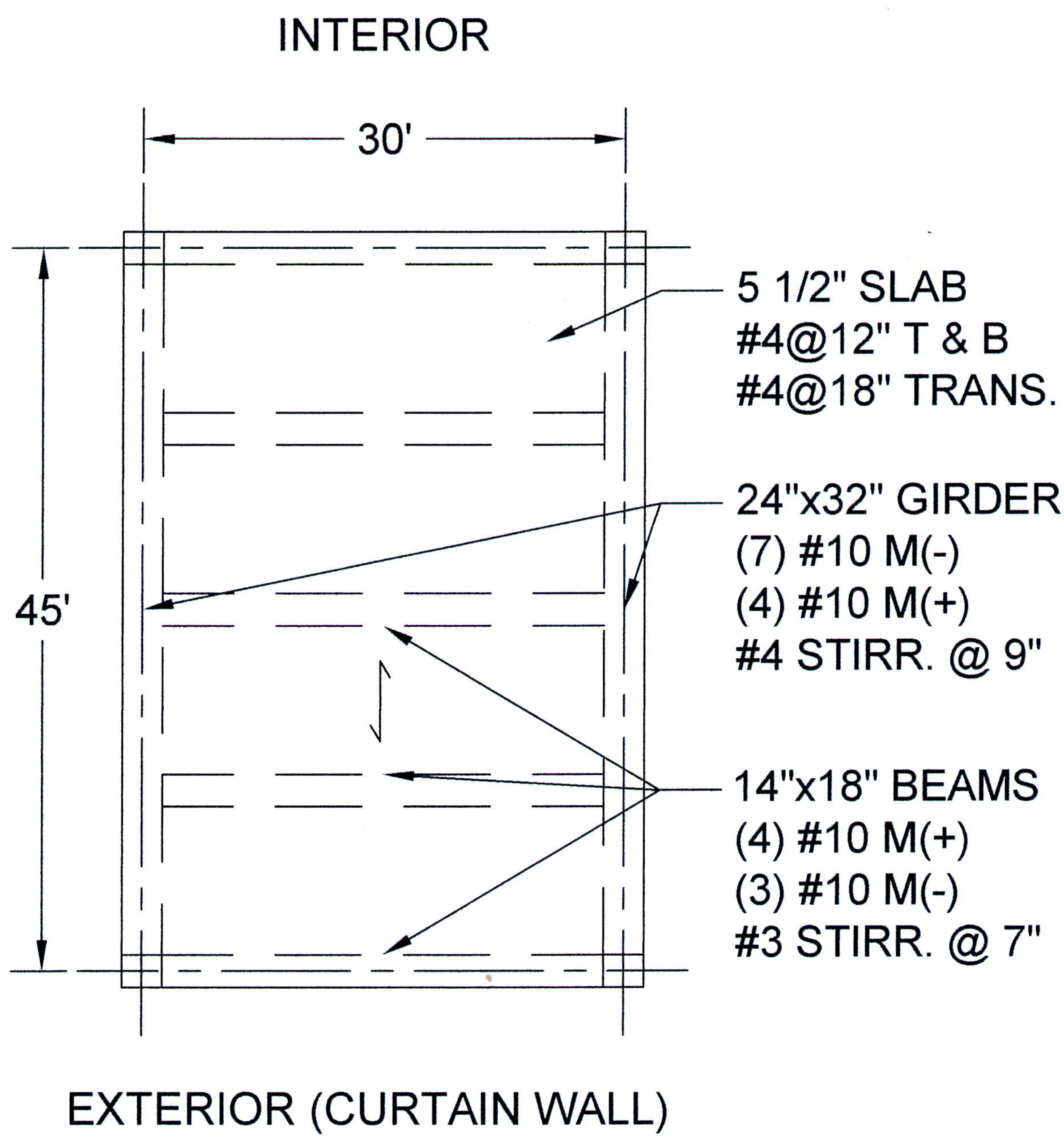
This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

Alternate System 3: One-Way Slab System

This section presents schematic level member sizes and evaluation of the typical bay of concrete framing in a one-way slab system.

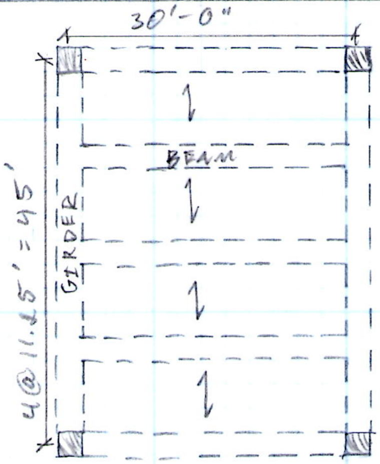


NOTES:
Fy=50 KSI
Ey=29,000 KSI
f'c=4000 PSI NORMAL WEIGHT
Ec=3605 KSI
2 HOUR FIRE RATING

TYPICAL BAY-ONE-WAY SLAB SYSTEM
SCALE: 3/32"=1'-0"

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

ALTERNATE SYSTEM: ONE WAY SLAB



$f'_c = 4000$ PSI

$F_y = 60$ KSI

NORMAL WEIGHT CONCRETE (150 PCF)

$E = 57000\sqrt{f'_c} = 57000\sqrt{4000}/1000 = 3605$ KSI

ASSUME 12" WIDE BEAMS (CONSERVATIVE)

DESIGN SLAB

- MINIMUM THICKNESS
  - FIRE RATING
    - 2 HOUR RATING → **5" MIN.** THICKNESS [ACI 216.1-07]
  - STRENGTH AND SERVICEABILITY
    - EXT. BAY: $h = l/24 = 11.25 \cdot 12/24 = 5.625" \rightarrow 5.75"$
    - INT. BAY: $h = l/28 = 11.25 \cdot 12/28 = 4.82" \rightarrow 5"$

    ∴ TRY **h = 5.75"** [ACI 318-11 TABLE 9.5(a)]

- DETERMINE d
  - ASSUME #4 REIN., $d_b = 0.5"$
  - CLEAR COVER = 3/4"
  - $d = h - \text{CLR. COVER} - d_b/2$

    $d = 5.75" - 0.75" - 0.25" = \mathbf{4.75"}$

- DETERMINE LOADS
  - $w_d = 150\text{ PCF}(5.75/12) + 20\text{ PSF SDL} = 91.9$ PSF
  - $w_L = 50$ PSF
  - ROOF:

    $w_d = 150\text{ PCF}(5.75/12) + 25\text{ PSF SDL} = 96.9$ PSF

    $w_{Lr} = 20$ PSF

    $S = 22$ PSF
  - $w_u = 1.4\, w_d$

    $= 1.4(91.9\text{ PSF}) = 129$ PSF

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PROJECT	RGA GLOBAL HQ TECHNICAL REPORT 3
TITLE	ONE-WAY SLAB SYSTEM SLAB DESIGN
BY:	NMB
SHEET:	A3.3
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- $W_u = 1.2W_d + 1.6W_L$
  $= 1.2(91.9 \text{ PSF}) + 1.6(50 \text{ PSF}) = 190 \text{ PSF}$ CONTROLS

- DESIGN MOMENT
  - ASSUME TENSION CONTROLLED SECTION ⇒ $\phi = 0.9$
  - ABLE TO USE MOMENT COEFFICIENT METHOD?
    - 4 SPANS > 2 SPANS ✓
    - SPANS EQUAL ✓
    - UNIFORM LOADS ✓
    - $W_L = 50 \text{ PSF} < 3W_d = 276 \text{ PSF}$ ✓
    - MEMBERS ARE PRISMATIC ✓

    ∴ METHOD OK

  - LARGEST DESIGN MOMENT

$l_n = 11.25' - 1 = 10.25'$

$$M_u^- = \frac{-W_u l_n^2}{10} = \frac{-(190)(10.25^2)}{10(1000)} = -2.0 \text{ ft}\cdot\text{K/ft WIDTH}$$

$$M_u^+ = \frac{W_u l_n^2}{16} = \frac{(190)(10.25^2)}{16(1000)} = +1.248 \text{ ft}\cdot\text{K/ft WIDTH}$$

- REQUIRED REINFORCEMENT
  - ASSUME $jd \approx 0.95d$

$$A_s \geq \frac{M_u}{\phi f_y(d - a/2)} \approx \frac{M_u}{\phi f_y jd} = \frac{(2.0 \text{ ft}\cdot\text{K/ft})(12)}{0.9(60)(0.95)(4.75)} = 0.098 \text{ in}^2/\text{ft}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.098(60)}{0.85(4)(12'')} = 0.144''$$

PER FOOT WIDTH

$$A_s \geq \frac{M_u}{\phi f_y(d - a/2)} = \frac{(2\cdot 12)}{0.9(60)(4.75 - 0.144/2)} = 0.095 \text{ in}^2/\text{ft}$$

$$\rho = \frac{A_s}{bd} = \frac{0.095}{12(4.75)} = 0.00167 \quad \underline{OK}$$

- SHEAR CHECK

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(190)(10.25)}{2} = 1120 \text{ \#/FT}$$

$\phi V_c = \phi 2\lambda\sqrt{f'_c}\, b_w d$

$\phi = 0.75$ [ACI 318-11 §9.3.2.3]

$\lambda = 1.0$ FOR NORMAL WT CONC.

$\phi V_c = 0.75(2)\sqrt{4000}(12)(4.75) = 5407 \text{ \#/FT}$

$\phi V_c > V_u$ <u>OK</u>

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PROJECT	BY:	SHEET:
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TITLE	CHKD:	PROJECT NO:
ONE-WAY SLAB SYSTEM SLAB DESIGN		TECH 3
	DATE:	PAGE:

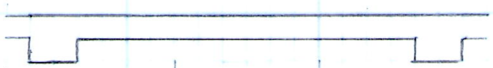
• DESIGN REINFORCEMENT

$-A_{s\,MIN} = \frac{3\sqrt{f'c}}{f_y} b_w d = \frac{3\sqrt{4000}}{60\,000}(12)(4.75) = 0.180\ in^2/ft$

$-A_{s\,MAX} = \frac{200\,b_w d}{f_y} = \frac{200(12)(4.75)}{60\,000} = 0.190\ in^2/ft$

– TEMPERATURE AND SHRINKAGE [ACI 318-11 § 7.6.5]

$A_{s\,MIN} = 0.0018\,bh = 0.0018(12)(5.75) = 0.124\ in^2/ft$



l_n (ft)	10.25	10.25	10.25
$W_u l_n^2$	20.0	20.0	20.0
M (COEFF.)	$-1/11$	$+1/16$	$-1/11$
M_u	-1.82	1.25	-1.82
A_s req'd	0.090	0.062	0.090
$A_{s\,MIN}$	0.180	0.180	0.180
REBAR	#4 @ 12"	#4 @ 12"	#4 @ 12"
$A_{s\,FINAL}$	0.20	0.20	0.20

• SPACING

$S_{MAX} = \text{MIN} \begin{cases} 3h = 3(5.75) = 17.25" \Rightarrow 17" \\ 18" \end{cases}$ [ACI 318-11 § 7.6.5]

– CRACK CONTROL [ACI 318-11 § 10.6.4]

$S_{MAX} = 15\left(\frac{40\,000}{2/3 f_y}\right) - 2.5(\text{CLR. COVER})$

$= 15\left(\frac{40\,000}{2/3 \cdot 60\,000}\right) - 2.5(0.75) = 13.125" \Rightarrow 13"$

$S_{MAX} = 12\left(\frac{40\,000}{2/3 \cdot 60\,000}\right) = 12"$ CONTROLS

• TEMPERATURE AND SHRINKAGE (TRANSVERSE)

$A_s = \text{MAX} \begin{cases} \rho_{min}\,bd = 0.0014(12)(4.75) = 0.0798\ in^2/ft \\ \rho_{min}\,bd = 0.0018(12)(4.75) = 0.102\ in^2/ft \end{cases}$

$S_{MAX} = \text{MIN} \begin{cases} 5h = 5(5.75) \\ 18" \quad \text{CONTROLS} \end{cases}$ ∴ #4 @ 18"

5.75" SLAB WITH: #4 @ 12" O.C. TOP & BOTTOM (FLEXURE)
#4 @ 18" O.C. TRANSVERSE

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PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 3

TITLE: ONE-WAY SLAB SYSTEM SLAB DESIGN

BY: NMB

SHEET: A3.5

CHKD:

PROJECT NO: TECH 3

DATE:

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DESIGN INTERIOR BEAMS

TRY b = 14"

• FIND h

$$h = \ell/21 = 30 \cdot 12/21 = 17.1" \Rightarrow 18"$$ [ACI 318-11 TABLE 9.5(a)]

• DETERMINE LOADS

$W_{BM} = 150\ PCF\ (14/12)\left(\frac{18-5.75"}{12}\right) = 179$ PLF

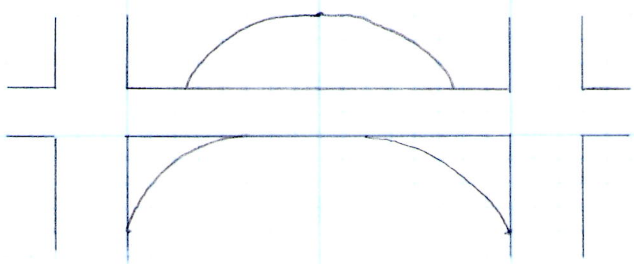
$W_{SLAB} = 150\ (5.75"/12)(11.25') = 809$ PLF

$W_{SDL} = 20\ PSF\ (11.25) = 225$ PLF

$W_{LL} = 50\ PSF\ (11.25) = 563$ PLF

$W_u = 1.2\,W_D + 1.6\,W_L$

$= 1.2(0.179 + 0.809 + 0.225) + 1.6(0.563) = 2.36$ KLF



• DESIGN MOMENTS

$$M_u^+ = \frac{w\ell^2}{8} = \frac{2.36(30^2)}{8} = 266\ \text{k·ft}$$

$$M_u^- = \frac{-w\ell^2}{11} = \frac{-2.36(30^2)}{11} = -193\ \text{k·ft}$$

• REINFORCEMENT

- TRY #10's
- $d = h - CLR - d_b/2 = 18 - 1.5 - 1.27/2 = 15.9" \Rightarrow 15.75"$
- $A_s = \frac{M_u}{4d} = \frac{266}{4(15.75)} = 4.2\ in^2$

  (4) #10 $\Rightarrow A_s = 4(1.27) = 5.08\ in^2 > 4.2\ in^2$ OK
- $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{5.08(60)}{0.85(4)(14)} = 6.40"$
- $c = a/\beta_1 = 6.40/0.85 = 7.53"$ $\beta_1 = 0.85$ FOR 4000 PSI
- $\varepsilon_s = \varepsilon_u\left(\frac{d-c}{c}\right) = 0.003\left(\frac{15.75-7.53}{7.53}\right) = 0.00327 > 0.002$ OK

- $\varepsilon_t > 0.005 \quad \therefore \phi = 0.9$

- MIDSPAN MOMENT

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

$= 5.08(60)(15.75 - 6.4/2)/12 = 3825 \text{ k·in} = 319 \text{ k·ft}$

$\phi M_n = 287 \text{ ft·K}$

$\phi M_n = 287 \text{ Ft·K} > M_u^+ = 266 \text{ ft·K}$

- CHECK $A_{s\,MIN}$

$$A_{s\,MIN} = MAX \begin{cases} \dfrac{3\sqrt{f'c}}{f_y} b_w d = \dfrac{3\sqrt{4000}}{60000}(14)(15.75) = 6.97 \text{ in}^2 \\ \dfrac{200 b_w d}{f_y} = \dfrac{200(14)(15.75)}{60000} = 0.735 \text{ in}^2 \text{ CONTROLS} \end{cases}$$

$A_{s,PROV} > A_{s\,MIN}$ OK

- MOMENT AT SUPPORTS

$M_u^- = -193 \text{ k·ft}$

$A_s = \dfrac{M_u}{4d} = \dfrac{193}{4(15.75)} = 3.06 \text{ in}^2$

- TRY (3) #10 => $A_{s\,PROV} = 3.81 \text{ in}^2$

$a = \dfrac{3.81(60)}{0.85(4)(14)} = 4.80''$

$c = a/\beta_1 = 4.80/0.85 = 5.65''$

$\varepsilon_s = 0.003\left(\dfrac{15.75 - 5.65}{5.65}\right) = 0.00536 > 0.005$ OK

$M_n = 3.81(60)(15.75 - 4.8/2) = 3052 \text{ k·in} = 254 \text{ k·ft}$

$\phi M_n = 229 \text{ ft·K}$

$\phi M_n > M_u$ OK

- PASSES $A_{s\,MIN}$ BY INSPECTION

- SHEAR CHECK

$l_n = 30' - (14''/12) = 28.83'$

$V_u = \dfrac{w_u l_n}{2} = \dfrac{2.36(28.83)}{2} = 34.0^K$

$\phi V_n = \phi(V_c + V_s)$

$\phi = 0.75$

$V_c = 2\lambda\sqrt{f'c}\, b_w d = 2(1)\sqrt{4000}(14)(15.75) = 27.9^k$

$V_s = A_v f_{yt}(d/s)$

$V_s = \frac{V_u}{\phi} - V_c = \frac{34}{0.75} - 27.9 = 17.4^k < 2(27.9)^k$ OK

$S_{MAX} = \underset{MIN}{} \left| \begin{array}{l} d/2 = 15.75/2 = 7.9 \Rightarrow 7'' \text{ CONTROLS} \\ 24'' \end{array} \right.$

$A_{vMIN} = \underset{MAX}{} \left| \begin{array}{l} 0.75\sqrt{f'c}\, b_w s/f_{yt} = 0.75\sqrt{4000}(14)(7)/60000 = 0.077 in^2 \\ 50 b_w s/f_{yt} = 50(14)(7)/6000 = 0.0817 in^2 \end{array} \right.$

$A_{v PROV} = \#3$ [ ] $= 0.22\ in^2 > 0.077 in^2$

$s = A_v f_{yt}(d/V_s) = 0.22(60)(15.75/17.4) = 11.9'' \Rightarrow 11'' > S_{MAX}$

$\therefore s = 7''$

$V_s = (0.22)(60)(15.75/7) = 29.7^K$

$\phi V_n = 0.75(27.9^K + 29.7^K) = \mathbf{43.2^K}$

$\phi V_n > V_u$ #3 [ ] @ 7" O.C.

- CHECK DEFLECTION

$\Delta_{LLMAX} = \frac{l}{360} = \frac{30 \cdot 12}{360} = 1.0''$

$\Delta_{TLMAX} = l/240 = 30 \cdot 12/240 = 1.5''$

- USING SIMPLIFIED DEFLECTION CHECK TO GET COMPARISONS AND SIZED BEAM FROM ACI 318 TABLE 9.5(a).

$I = \frac{bh^3}{12} = \frac{14(18^3)}{12} = 6804\ in^4$

$W_{LL} = 563$ PLF

$\Delta_{LL} = \frac{5 w_{LL} l^4 (1728)}{384 EI} = \frac{5(0.563)(30^4)(1728)}{384(3605)(6804)} = 0.42'' < 1''$ OK

$W_{TL} = 1.78$ KLF

$\Delta_{TL} = \frac{5(1.78)(30^4)(1728)}{384(3605)(6804)} = 1.32'' < 1.5''$ OK

DESIGN GIRDER

$45' = l$

- DETERMINE LOADS

$W_{LL} = 50 \text{ PSF } (30') = 1.5 \text{ KLF}$

$P = w_d l = 1.2 \text{ KLF } (30') = 36^k$

$W_{DL} = 3P/l = 3 \cdot 36/45 = 2.4 \text{ KLF}$

- ASSUME SELF WEIGHT = 10-15% OF DL

$W_u = 1.2\, w_{DL}(1.15) + 1.6\, W_L$

$= 1.2(2.4)(1.15) + 1.6(1.5) = 5.71 \text{ KLF}$

$M_u = \frac{wl^2}{12} = \frac{5.71 \cdot 45^2}{12} = 964 \text{ ft} \cdot \text{K}$

- ESTIMATE GIRDER SIZE

$20 M_u = bd^2$

- ASSUME $b = 3/4\, d$

$20 M_u = 0.75 d^3$

$\sqrt[3]{26.67 M_u} = d$

$d = (26.67 \cdot 964)^{1/3} = 29.5'' \Rightarrow h = 32''$

$b = 0.75 \cdot 29.5 = 22.1 \Rightarrow 23'' \Rightarrow b = 24''$

$W_{SW} = 150 \text{ PCF } (24/12)\left(\frac{32 - 5.75}{12}\right)/1000 = 0.656 \text{ KLF}$

$W_u = 1.2(2.4 + 0.656) + 1.6(1.5) = 6.07 \text{ KLF}$

- DESIGN MOMENTS

$M_u^+ = \frac{wl^2}{24} = \frac{6.07(45^2)}{24} = 512 \text{ ft} \cdot \text{K}$

$M_u^- = \frac{-wl^2}{12} = \frac{-6.07(45^2)}{12} = 1024 \text{ ft} \cdot \text{K}$

- NEGATIVE REINFORCEMENT

$A_s = \frac{M_u^-}{4d} = \frac{1024}{4(29.5)} = 8.67 \text{ in}^2$

TRY (7) #10 $\Rightarrow A_{s\,PROV} = (1.27 \text{ in}^2)(7) = 8.89 \text{ in}^2$

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PROJECT	BY:	SHEET:
RGA GLOBAL HQ TECHNICAL REPORT 3	NMB	A3.9
	CHKD:	PROJECT NO: TECH 3
TITLE: ONE-WAY SLAB SYSTEM GIRDER DESIGN	DATE:	PAGE:

$$a = \frac{8.89(60)}{0.85(4)(24)} = 6.54''$$

$$c = a/\beta_1 = 6.54/0.85 = 7.69''$$

$$\varepsilon_s = 0.003\left(\frac{29.5 - 7.69}{7.69}\right) = 0.0085 > 0.002 \text{ OK}$$

$\phi = 0.9$

$$\phi M_n = 0.9(8.67)(60)(29.5 - 6.54/2)/12 = 1024 \text{ k.ft}$$

$\phi M_n = M_u$

- POSITIVE REINFORCEMENT

$$A_s = \frac{512}{4(29.5)} = 4.34 \text{ in}^2$$

$(4)\ \#10 \Rightarrow A_{s\,PROV} = 1.27(4) = 5.08 \text{ in}^2$

$$a = \frac{5.08(60)}{0.85(4)(24)} = 3.74''$$

$$c = 3.74/0.85 = 4.4''$$

$$\varepsilon_s = 0.003\left(\frac{29.5 - 4.4}{4.4}\right) = 0.017 > 0.003 \quad \phi = 0.90$$

$$\phi M_n = 0.9(8.89)(60)(29.5 - 3.74/2)/12 = 1105 \text{ ft.k}$$

$\phi M_n = 1105 \text{ ft.k} > M_u = 512 \text{ ft.k}$

- SHEAR CHECK

$$l_n = 45' - 14/12 = 43.83'$$

$$V_u = \frac{5.71(43.83)}{2} = 125^k$$

$$V_c = 2(1)\sqrt{4000}(24)(29.5) = 89.6^k$$

$$V_s = 125/0.75 - 89.6 = 57.5^k < 2(89.6) \quad \text{OK}$$

$s_{MAX} = \text{MIN} \begin{cases} 29.5/2 = 14.75'' \Rightarrow 14'' \\ 24'' \end{cases}$

$A_{v\,MIN} = \text{MAX} \begin{cases} 0.75\sqrt{4000}(24)(14)/60000 = 0.266 \text{ in}^2 \quad \text{CONTROLS} \\ 50(24)(14)/60000 = 0.28 \text{ in}^2 \end{cases}$

$A_{v\,PROV} = \#4\ [\,] = 0.40 \text{ in}^2 > 0.266 \text{ in}^2$

$s = 0.4(60)\left(\frac{29.5}{57.5}\right) = 12.3" \Rightarrow 12"$ CONTROLS

$V_s = 0.4(60)(29.5/12) = 59$

$\phi V_n = 0.75(89.6 + 59) = 111^k < 125^k$ NG

- DECREASE STIRRUP SPACING

$125/0.75 - 89.6 = 77.1 = V_s$

$$\frac{77.1}{0.4(60)} = \frac{29.5}{s} \Rightarrow s = 9.83" \Rightarrow 9"$$

$V_s = 0.4(60)(29.5/9) = 78.67$

$\phi V_n = 0.75(89.6 + 78.67) = 126^k > 125^k$

$\phi V_n > V_u$ OK #4 [ ] @ 9" o.c.

- CHECK DEFLECTION

$\Delta_{LL\,MAX} = \ell/360 = 45 \cdot 12/360 = 1.5"$

$\Delta_{TL\,MAX} = \ell/240 = 45 \cdot 12/240 = 2.25"$

$$I = \frac{24(32^3)}{12} = 65536 \text{ in}^4$$

$w_{LL} = 1.5$ KLF

$$\Delta_{LL} = \frac{(5)(1.5)(45^4)(1728)}{384(3605)(65536)} = 0.586" < 1.5" \text{ OK}$$

$w_{TL} = 4.56$ KLF

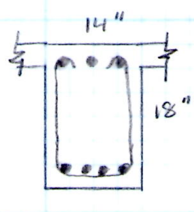
$$\Delta_{TL} = \frac{(5)(4.56)(45^4)(1728)}{384(3605)(65536)} = 1.78" < 2.25" \text{ OK}$$

SYSTEM SUMMARY

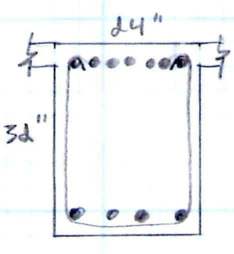
5.75" SLAB

#4 @ 12" o.c. TOP AND BOTTOM FLEXURE

#4 @ 18" o.c. TRANSVERSE



(4) #10 M+
(3) #10 M−
#3 [ ] @ 7"



(7) #10 M−
(4) #10 M+
#4 [ ] @ 9"

SYSTEM DEPTH

$h = 32"$

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PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 3

TITLE: ONE-WAY SLAB SYSTEM GIRDER DESIGN

BY: NMB

SHEET: A3.11

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PROJECT NO: TECH 3

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System Comparisons

This section presents calculations to support the comparison of the considered systems.

SYSTEM COMPARISONS

- SYSTEM WEIGHTS
  - EXISTING:
    - CONC: 33.8 PSF
    - DECK: 2.14 PSF
    - FRAMING: 13 PSF
    - 49 PSF
  - JOIST SYSTEM:
    - CONC: 33.8 PSF
    - DECK: 2.14 PSF
    - JOISTS (21 PLF)(45')(8)/1350 SF = 5.6 PSF
    - W24x68: (68 PLF)(30')/1350 = 1.51 PSF
    - W27x94: (94 PLF)(30')/1350 = 2.1 PSF
    - W12x79: (79 PLF)(14')(4)/1350 = 3.3 PSF
    - TOTAL = 48.5 PSF
  - HOLLOW CORE SYSTEM:
    - PLANK = 68 PSF
    - TOPPING = 25 PSF
    - W24x62: (62 PLF)(30')/1350 = 1.38 PSF
    - W27x94: 2.1 PSF
    - W30x108: (108 PLF)(45')(2)/1350 = 7.2 PSF
    - W14x99: (99 PLF)(14')(4)/1350 = 4.11 PSF
    - TOTAL: 107.8 PSF
  - ONE-WAY SLAB:
    - SLAB: 150 PCF (5.75/12) = 71.9 PSF
    - BEAMS: 150 PCF (14/12)(18/12)(5)(30')/1350 = 29.2 PSF
    - GIRDERS: 150 PCF (24/12)(32/12)(2)(45')/1350 = 53.3 PSF
    - COL. ALLOW: 150 PCF (18/12)(2)(4)(14')/1350 = 18.7 PSF
    - TOTAL: 101.2 PSF

EXISTING SYSTEM-Bare Cost					
Item	**Item #**	**Material**	**Install.**	**Total**	**Bay Size Adjust.**
Columns	B1010 208 9200	14.05	0.36	14.42	N/A
Composite Beams, Deck & Slab	B 1010 256 8000	18.60	7.65	26.25	25.31
			Cost=	**39.73**	**per. SF**

FLOOR JOIST SYSTEM-Bare Cost					
Item	**Item #**	**Material**	**Install.**	**Total**	**Bay Size Adjust.**
Steel Joists, Beams & Slab on Columns	B 1010 250 9750	18.70	7.20	25.90	28.54
Steel Column	B 1010 250 9800	6.04	1.80	7.84	8.64
			Cost=	**37.18**	**per. SF**

HOLLOW CORE PLANK SYSTEM-Bare Cost					
Item	**Item #**	**Material**	**Install.**	**Total**	**Bay Size Adjust.**
W Shape Beams & Girders	B1010 241 9970	19.55	6.80	26.35	29.04
Precast Plank with 2" Topping	B1010 230 3600	8.45	4.84	13.25	N/A
Steel Column	B1010 208 9200	14.05	0.36	14.42	N/A
			Cost=	**56.70**	**per. SF**

ONE-WAY SLAB SYSTEM-Bare Cost					
Item	**Item #**	**Material**	**Install.**	**Total**	**Bay Size Adjust.**
Cast in Place Slabs, One-Way	B1010 217 5700	4.14	8.40	12.54	N/A
Concrete Framing	(Local)	3.50	7.73	11.23	N/A
Concrete Column	B1010 203 9900	2.20	5.26	7.46	N/A
			Cost=	**31.23**	**per. SF**