DEPARTMENT OF STRUCTURAL ENGINEERING
UNIVERSITY OF CALIFORNIA SAN DIEGO

# Homework 4 Due: 11.59 pm Thursday 25 February 2021 (Electronic Submission)

**Notes:**

- **Please read the guidelines of submission of homework.**
- **A significant portion of the grade will be given to the interpretation of the results and discussion.**
- **The report shall be of professional quality, as expected in a design office, including the structural drawings. Poorly presented reports and drawings will be penalized up to 30%.**
- **Late submission policy:**
  - **No penalty if a Medical or Police excused is presented.**
  - **5% penalty if submitted between 12 am and 12.59 am of 26 February 2021.**
  - **7.5% penalty if submitted between 1 am and 5.59 am of 26 February 2021.**
  - **15% penalty if submitted between 6 am and 11.59 am of 26 February 2021.**
  - **25% if submitted 24h 01 m late, 35% if submitted 48h 01 m late and so forth.**

## Aim

This homework familiarizes students with the strut-and-tie method of analysis and design following ACI 318-19, and with the stringer-panel analysis method and design interpreting various prescriptive requirements in ACI 318-19. Furthermore, the homework requires students to present clear and well-detailed structural drawings and a professionally written report.

## Description

A RC structure in a power plant needs to be designed to resist two possible heavy load cases, see Fig. 1. The structure is composed of two identical deep beams spaced 10'-4". The beams are stepped and are 12'-0" high in the deepest portion and 6' high in the shallowest part. The deepest portion of the beams will be connected to 1'-6" deep top and bottom slabs forming a tube; see Sections 1 and 2 in Fig. 2. These slabs have been designed by a different group in your company and require 2-#6@8" o.c. top and bottom reinforcing mats (i.e., two #6 bars, bundled) ending in vertical 90-degree standard hooks. The structure itself will be supported on 3 ft. diameter columns on grids A and B and acting in the middle between the two deep beams, see Section 1 in Fig. 2. The structure and supporting columns will be cast-in-place. The entire structure will be exposed to weather, and the concrete cover shall meet the ACI 318-19 requirements prescribed in Table 20.5.13.1.

Loads $P_{u,LC1}$ and $P_{u,LC2}$ are to be applied to the structure at points c and d, see Fig. 1, with one- half of the load applied to each beam, see Section 2 in Fig. 2. Loads $P_{u,LC1}$ and $P_{u,LC2}$ will not be acting simultaneously, for which each load will be treated as an independent load case. These two loads will be applied to the structure by bolting proprietary hardware requiring 12 or 8 - 1 ¼" in. diameter A193 Grade B5 bolts into 2 in. thick ASTM A36 embedded steel plates in the pattern shown in Fig. 3. For this design, you may ignore the effects of self-weight.

**Specifications**

$f'_c$ = 5,000 psi, normal weight concrete

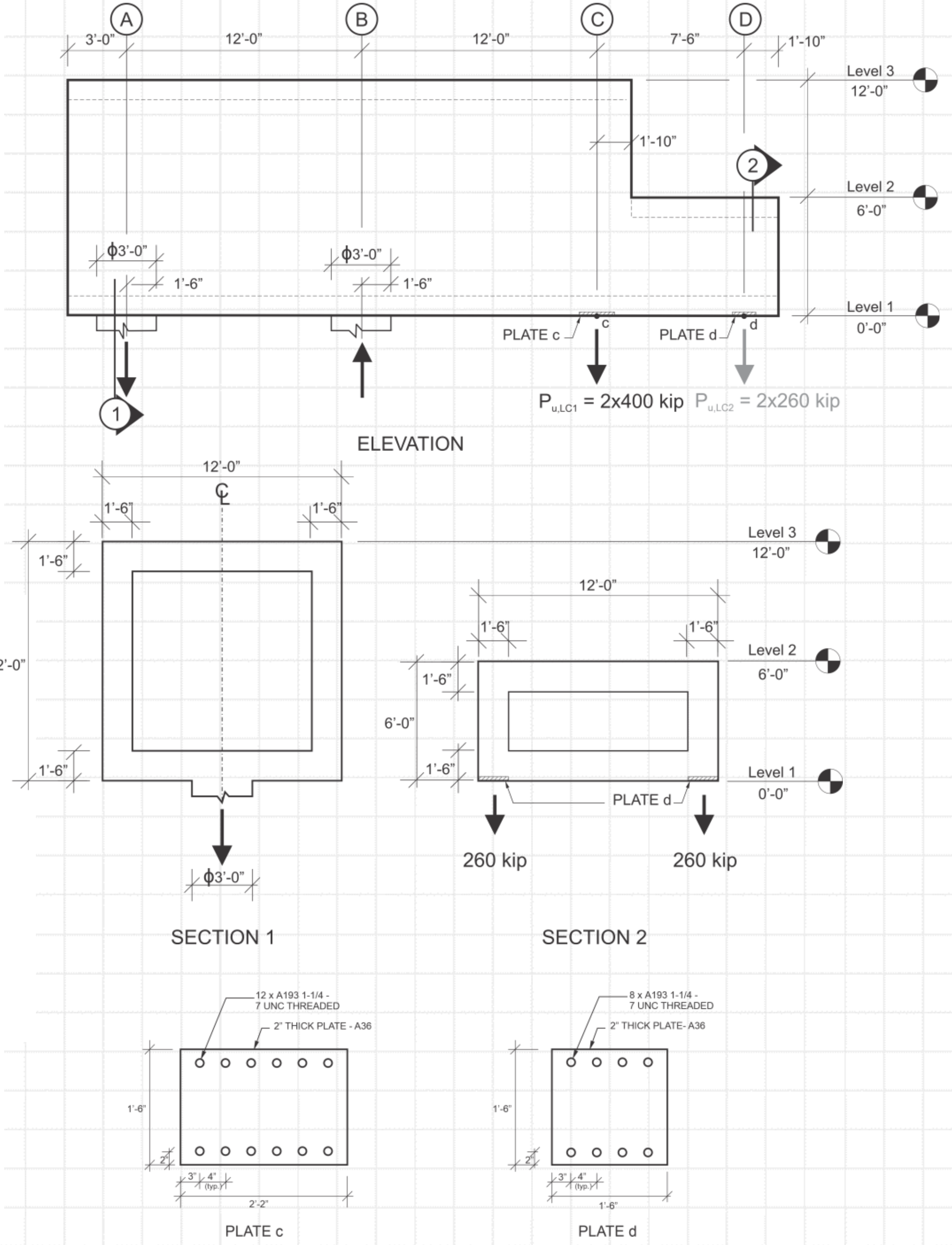
$f_y$ = 60,000 psi, ASTM A706 Grade 60 reinforcement

$F_y$ = 36,000 psi, ASTM A36 steel plate

Minimum cover to any reinforcement: 3 in.

Design strength of a A193 Grade B5 1 ¼" bolt: $\phi F_n$ = 69.0 kip (PCI Design Manual, Design Aid 6.15.6)

Use E80 electrodes to weld ASTM A706 Grade 60 reinforcement (PCI Design Manual, Design Aid 6.15.4)



## Suggested Report Layout

1. **Design Brief**
   Write a brief description of the work so that a third party can read and get a glimpse of the design work – Present in this section Figures 1-3 shown above.

2. **Bibliographic References**
   Cite here the bibliographical references used, use Harvard's format:


   ACI Committee 318, 2019. *Building Code Requirements for Structural Concrete (ACI 318-19)[and] Commentary on Building Code Requirements for Structural Concrete (ACI 318R-19)*, Farmington Hills, MI.

   Precast/Prestressed Concrete Institute, 2010. *PCI design handbook: Precast and prestressed concrete*. 7th Edition. Precast/Prestressed Concrete Institute, Chicago, IL.

   Croft, E. 2019. Revit video tutorial, UCSD Canvas.

   Other… like the coupler webpages


3. **Conceptual Design**
   Write a paragraph and let the reader know how you understand the flow of internal forces (i.e. load-path). Sketches and figures should be included. Though this structure appears to be 2D, it is actually a 3D structure. Describe the strategy to transfer the loads from the deep beams into the columns. It is acceptable to present hand sketches or reference figures presented in the HW4 tutorial.

4. **Statics**
   Ignore the dead load in this homework. Though important, it will require you to carry an extra strut and tie model just for the dead load, with barely any impact in the amount of reinforcement and no impact on detailing. Note that in a professional environment, you shall consider the presence of the dead load. You can use the tutorial sketches to show the applied loads and reactions.

5. **Embed Plate Design**
   PCI Design Manual, Design Aid 6.15.6, and check the number of bolts required by the hardware manufacturer are appropriate for loading plates c and d.

   Determine the number of Gr. 60 reinforcing bars needed to transfer the applied external loads $P_{u,LC1}$ and $P_{u,LC2}$ into the structure (Suggestion, use #9 bars). These bars shall be welded into the 2" thick plate. Show the layouts of the bars to be welded into each plate. Provide bar-plate welding details per PCI Design Manual, Design Aid 6.15.4.

   Welding very long bars normal to a steel plate can be cumbersome. Handling the embed plate with long bars can also be challenging. Suggestion, weld short bars onto the steel plates, and either (i) lap splice these bars in the field (and provide transverse reinforcement in the lap-splice region), or (ii) specify mechanical splices per ACI 318-19 §25.5.7. The Utility Company has accredited the following mechanical splices and corresponding headed ends:

   1. nVent LENTON taper-threaded splicing system https://www.erico.com/category.asp?category=R1433
   2. HRC 400 Series https://www.hrc-usa.com/hrc-400-series/

   (Optional: you may also wish to check the adequacy of the plate thickness)

6. **Strut and Tie Model Analysis and Design**
   Refer to the HW4 Tutorial for structural analysis, which has been done for you. Utilizing the information provide design the following:

   Load Case $P_{u,LC1}$
   - Propose a statically determinate truss model to transfer load $P_{u,LC1}$ onto the supports (Provided to you in this HW)
   - Carry out a structural analysis of the truss model (Provided to you in this HW)
   - Size and check node ℓ acting along grid C. Use Ch 23 of ACI 318-19
   - Size all other nodal zones per ACI 318-14 (Skipped in this HW)
   - Check effective stresses for all struts and nodes. Use Ch 23 of ACI 318-19

   Load Case $P_{u,LC2}$
   - Propose a statically determinate truss model to transfer load $P_{u,LC2}$ onto the supports (Provided to you in this HW)
   - Carry out a structural analysis of the truss model (Provided to you in this HW)
   - Size and check the top node acting along grid C. Use Ch 23 of ACI 318-19
   - Size all other nodal zones per ACI 318-14 (Skipped in this HW)
   - Check effective stresses for all struts and nodes. Use Ch 23 of ACI 318-19

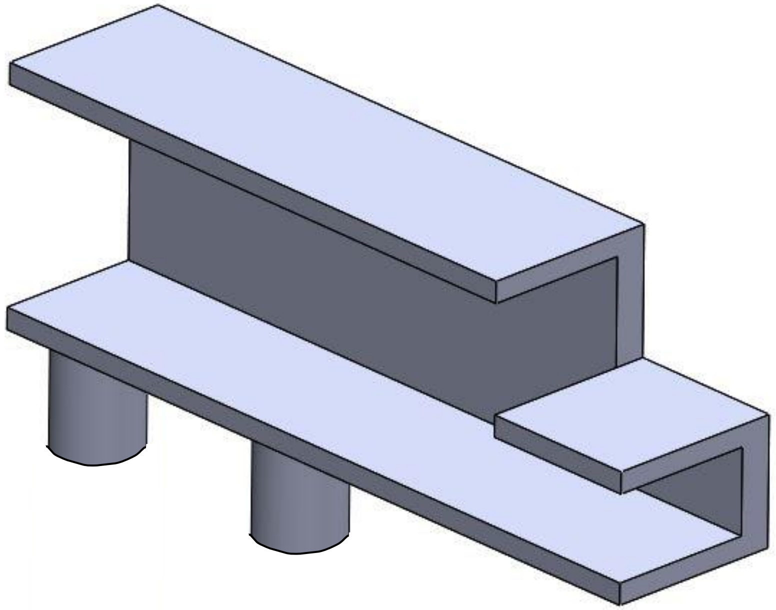
   Ties
   - Overlay the two truss models above and create an envelope of the ties (you should have done this also for the struts and check every strut for the maximum force, but we are skipping this requirement for the struts in the HW)
   - Design all ties per ACI 318-19 and provide adequate development to the reinforcement at each node
   - Detail two curtains of tie reinforcement

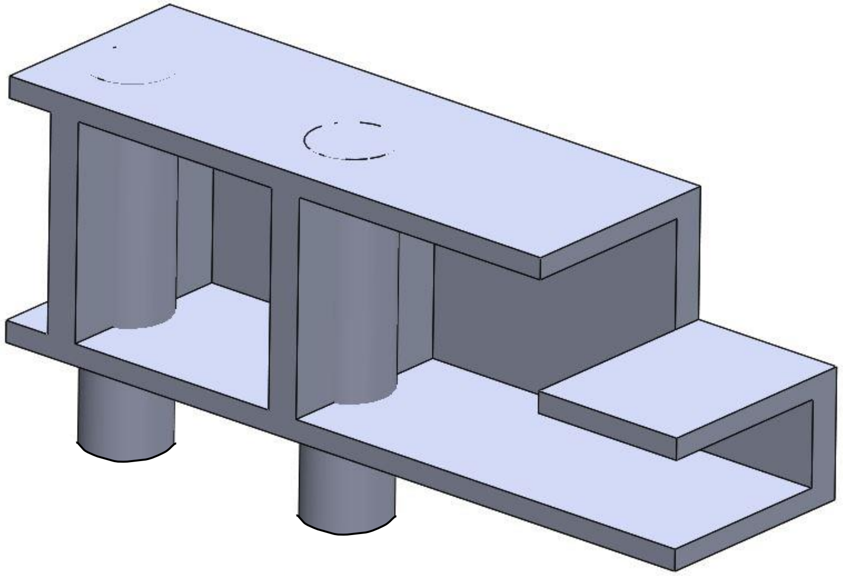
   Diaphragms
   - Propose a truss model to transfer the in-plane beam reactions at grid A (with the largest reactions from the two load cases) onto the columns via a set of vertical diaphragms. Use struts inclined between 25 and 65 degrees from the axis of a tie
   - Carry out a structural analysis of the truss model
   - Design the tie reinforcement in the diaphragm (make the diaphragm between 24 and 30 in. thick) and detail two curtains of tie reinforcement. Note: the column at A should be designed as a tie.
   - Size the struts and nodal zones per ACI 318-19 (Skipped in this HW)
   - Check the minimum diaphragm wall width
   - Using shear friction, perform an alternative design

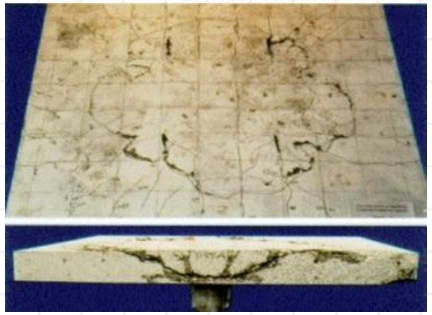
7. **Drawings**
   Develop exquisite drawings. See sample drawings in the HW4 tutorial. Drawings shall include:
   - Elevation (Side) View of Structure
   - Cross-Section of tunnel between Grids B-C
   - Cross Section between Level 1 and Level 2 (bottom segment of beam view)
   - Diaphragm Cross-Section at Grid A
   - Plate Details

8. **Use of Tutorial Truss Models and 3D Renderings**
   Student can use the renderings of the structure and the truss models depicted in the tutorial but they shall cite the tutorial material in the list of references.

# Conceptual Design







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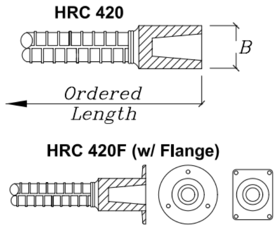
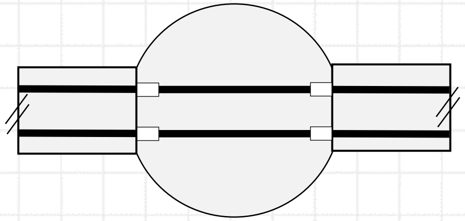


**HRC 400 Series High Performance Mechanical Couplers**

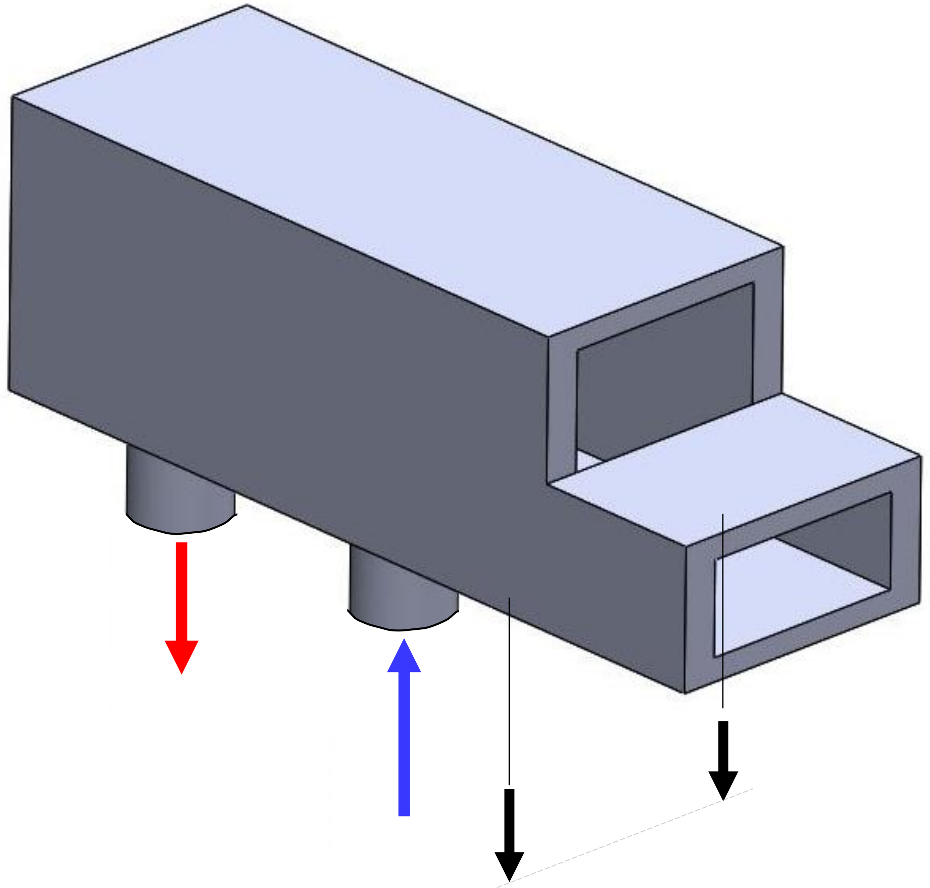
| Bar Size | Dla. (Inch) | Bar Dla. USA [1] | A ( in. ) | B ( in. ) | C ( in. ) | D ( in. ) | E ( in. ) | Fmax ( in. ) | Gmax ( in. ) | Torque* (lb-ft) |
|---|---|---|---|---|---|---|---|---|---|---|
| # 5 | 0.625 | 16mm | 1.000 | 1.000 | 1.625 | NA | NA | NA | 2.625 | 100 |
| # 6 | 0.750 | 19mm | 1.250 | 1.250 | 1.750 | NA | NA | NA | 2.625 | 100 |
| # 7 | 0.875 | 22mm | 1.500 | 1.500 | 2.000 | NA | NA | NA | 3.000 | 100 |
| # 8 | 1.000 | 25mm | 1.625 | 1.625 | 2.250 | NA | NA | NA | 3.500 | 150 |
| # 9 | 1.128 | 29mm | 1.750 | 1.750 | 2.500 | NA | NA | NA | 4.000 | 150 |
| # 10 | 1.270 | 32mm | 1.750 | 2.000 | 2.750 | 2.125 | 6.181 | 13.00 | 5.25 | 200 |
| # 11 | 1.410 | 36mm | 1.750 | 2.125 | 3.125 | 2.250 | 6.693 | 14.00 | 5.25 | 200 |
| # 14 | 1.693 | 43mm | 2.250 | 3.000 | 4.125 | 3.000 | 7.875 | 17.00 | 6.00 | 250 |
| # 18 | 2.257 | 57mm | 3.000 | 3.500 | 5.000 | 3.625 | 9.843 | 21.00 | 7.00 | 250 |

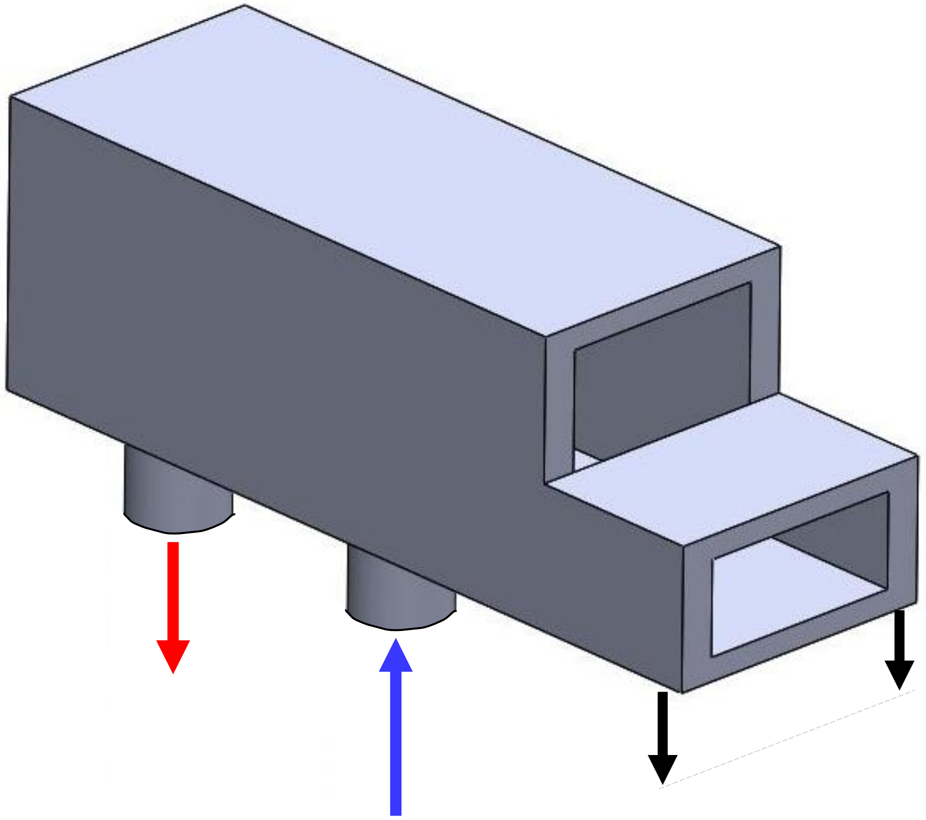
1) Designed to exceed actual stress and strain capacity of grade 60, made according to ASTM A706M specifications or equal.

* – Unless project qualifies and approves hand–tightening.

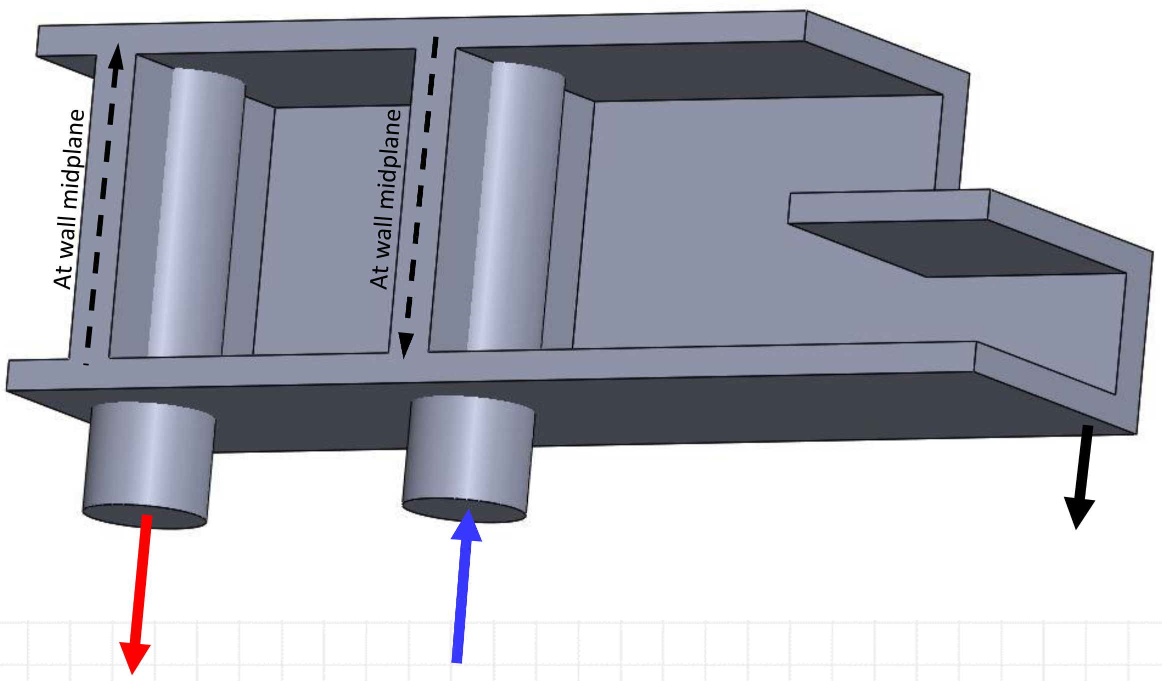
http://www2.hrc-usa.com/downloads/products/HRC400/HRC400-SPEC.pdf

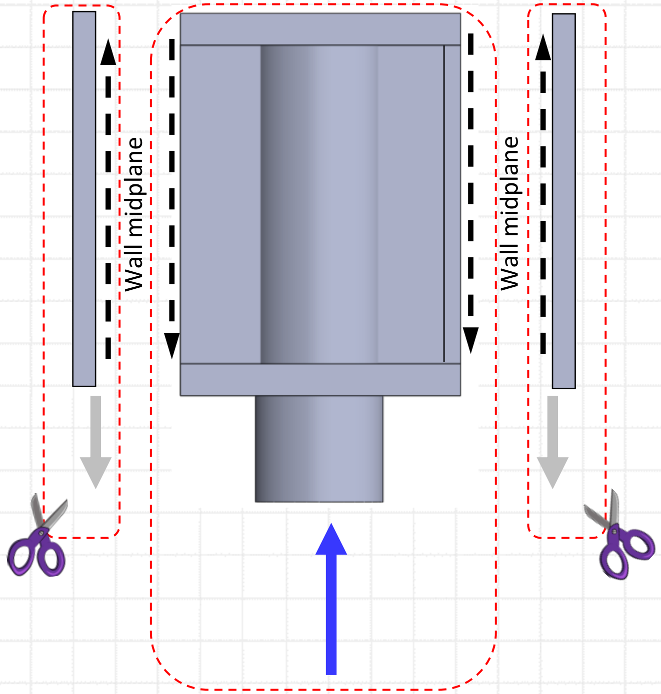
Load Cases and Reactions



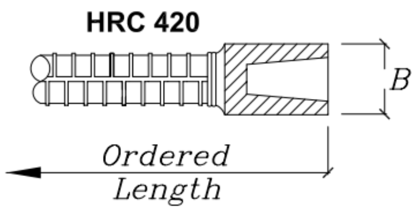


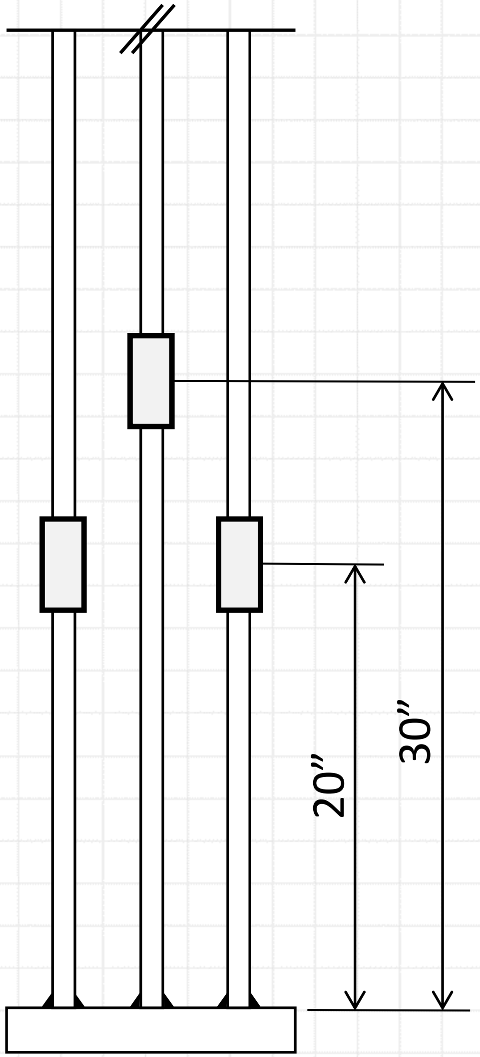
Shear Flow at the Diaphragm Wall Interface



# Embed Plate Design




**HRC 400 Series High Performance Mechanical Couplers**

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|---|---|---|---|---|---|---|---|---|---|---|
| # 5 | 0.625 | 16mm | 1.000 | 1.000 | 1.625 | NA | NA | NA | 2.625 | 100 |
| # 6 | 0.750 | 19mm | 1.250 | 1.250 | 1.750 | NA | NA | NA | 2.625 | 100 |
| # 7 | 0.875 | 22mm | 1.500 | 1.500 | 2.000 | NA | NA | NA | 3.000 | 100 |
| # 8 | 1.000 | 25mm | 1.625 | 1.625 | 2.250 | NA | NA | NA | 3.500 | 150 |
| # 9 | 1.128 | 29mm | 1.750 | 1.750 | 2.500 | NA | NA | NA | 4.000 | 150 |
| # 10 | 1.270 | 32mm | 1.750 | 2.000 | 2.750 | 2.125 | 6.181 | 13.00 | 5.25 | 200 |
| # 11 | 1.410 | 36mm | 1.750 | 2.125 | 3.125 | 2.250 | 6.693 | 14.00 | 5.25 | 200 |
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1) Designed to exceed actual stress and strain capacity of grade 60, made according to ASTM A706M specifications or equal.

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http://www2.hrc-usa.com/downloads/products/HRC400/HRC400-SPEC.pdf

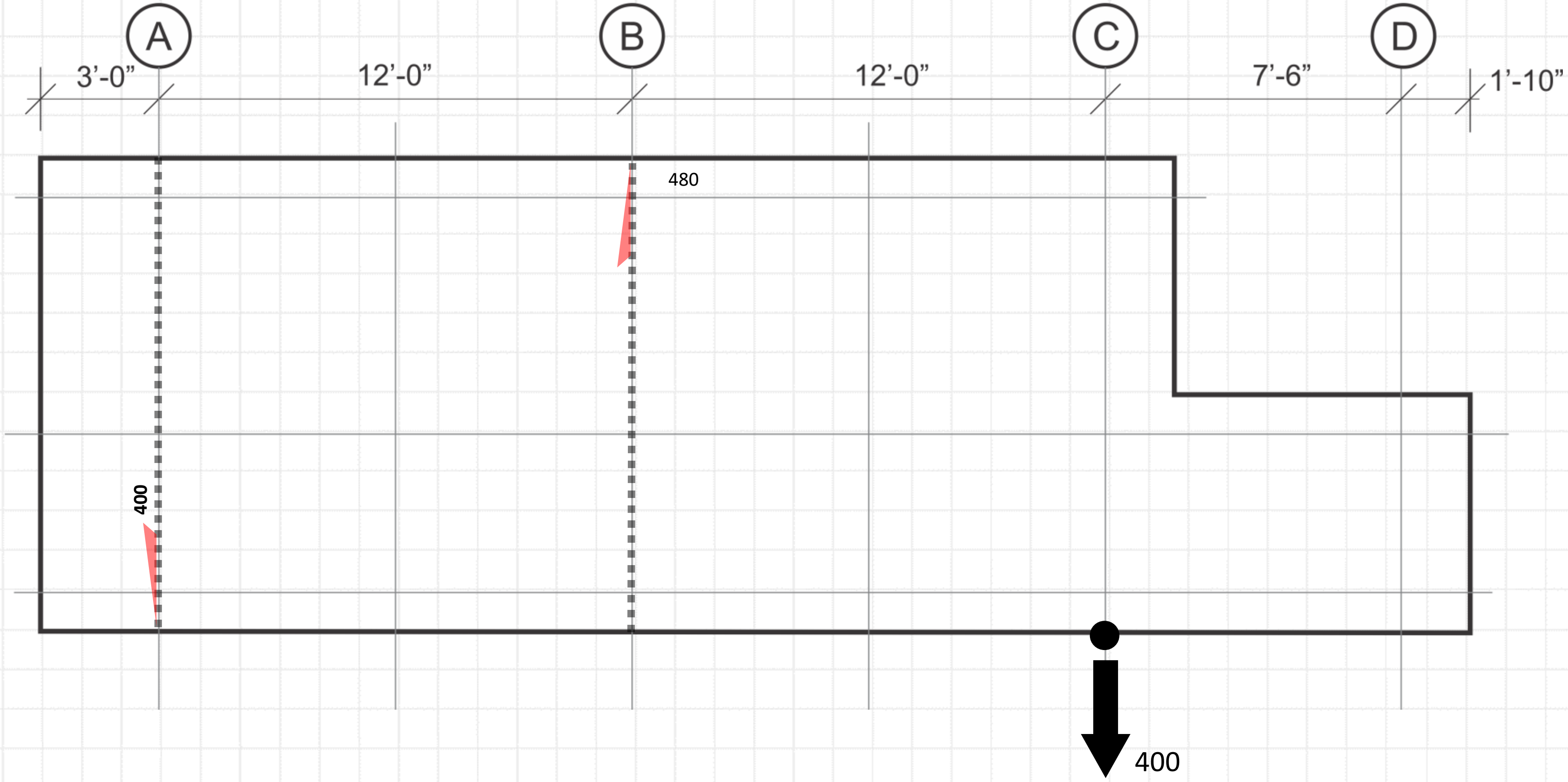
https://texumrebarcoupler.com/wp-content/uploads/2020/04/rebar-coupler-blog.jpg

# Truss Models – Load Case 1

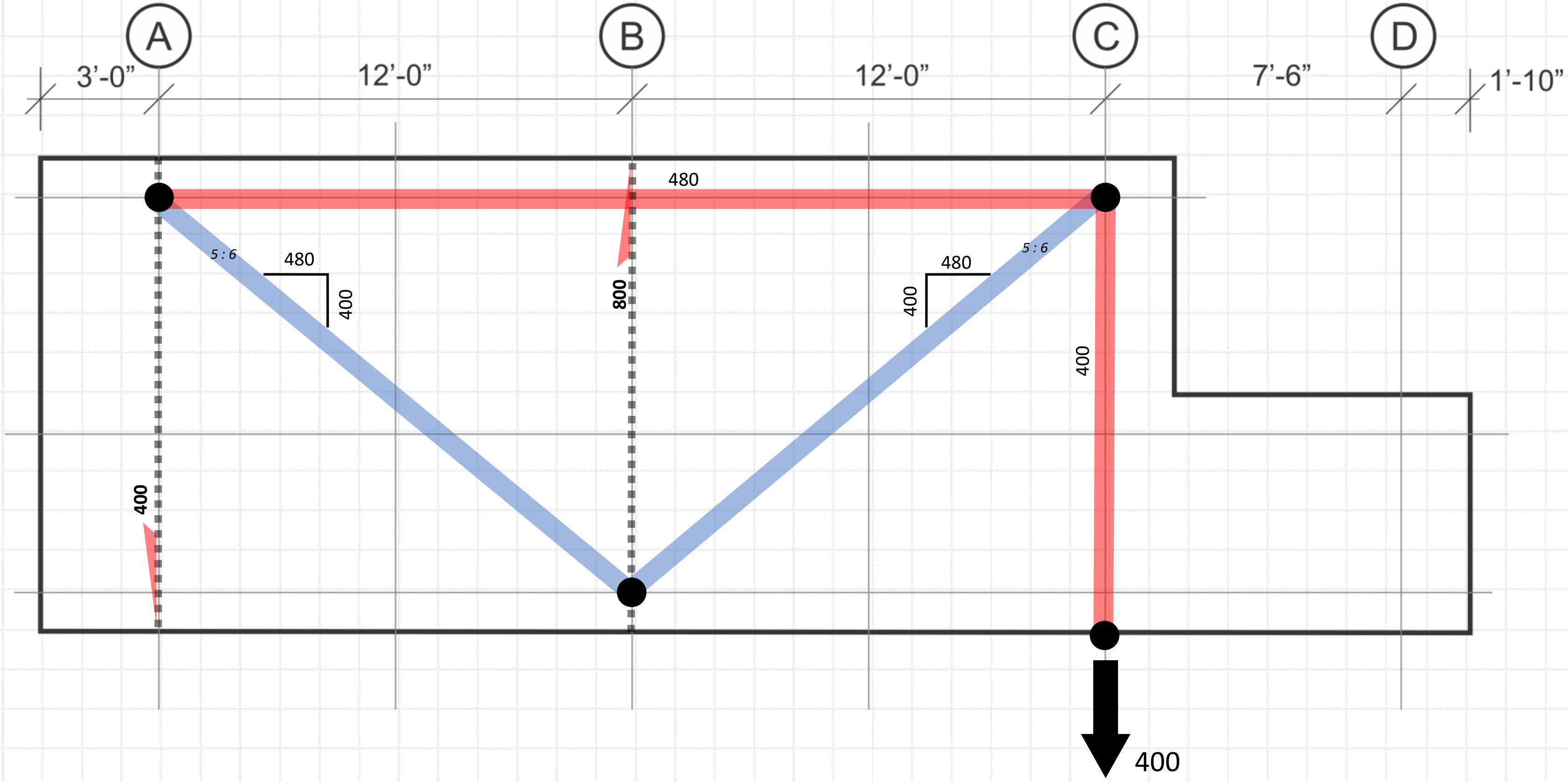
The challenge is to find an internal force flow (load path) using a statically determinate truss that satisfies all boundary conditions:

-truss elements within physical envelope but not at the envelope

-explain the collector



# Truss Models – Load Case 1

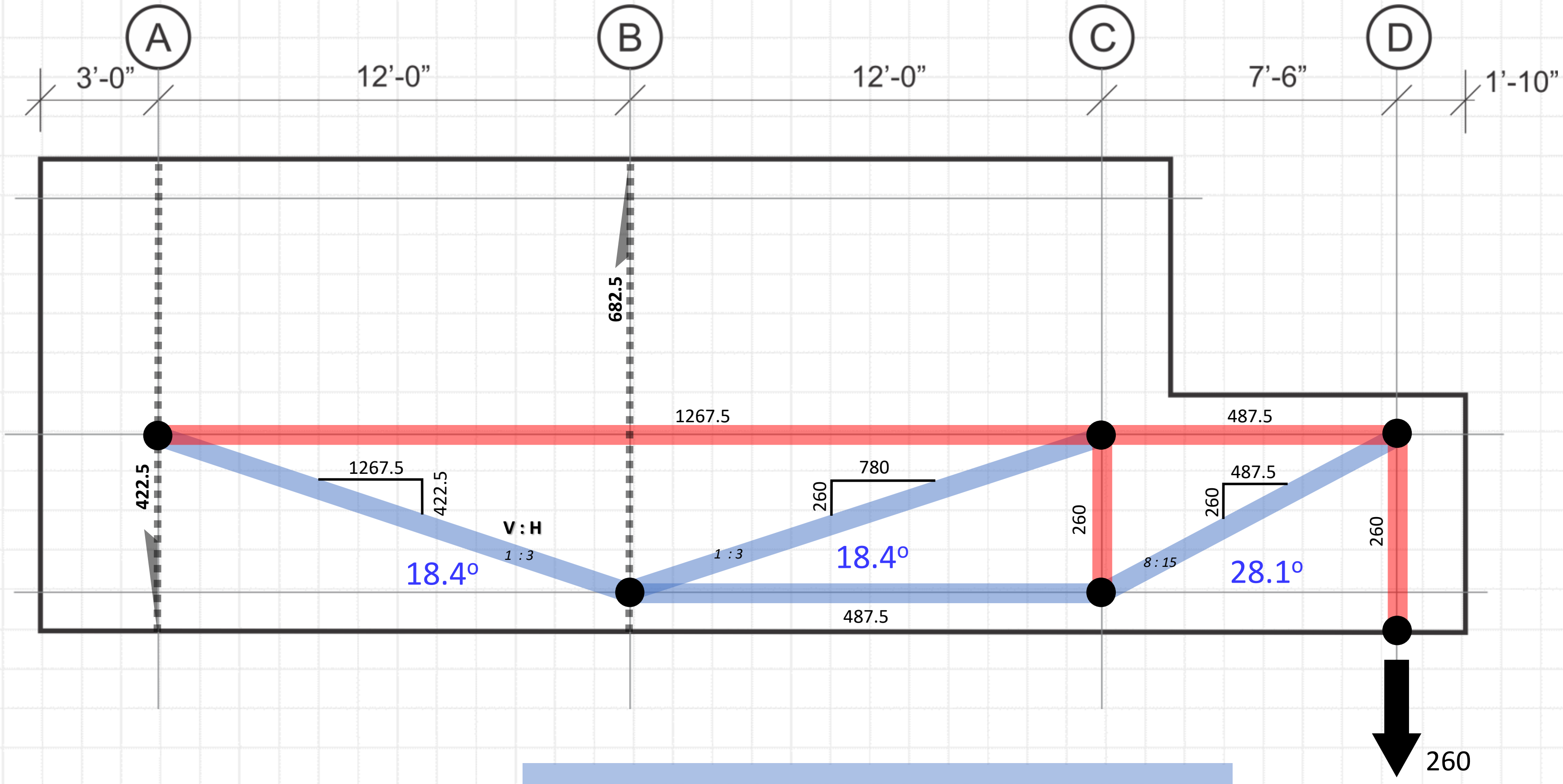



Explain

Strain energy index

SEI= 480x24 = 11,520

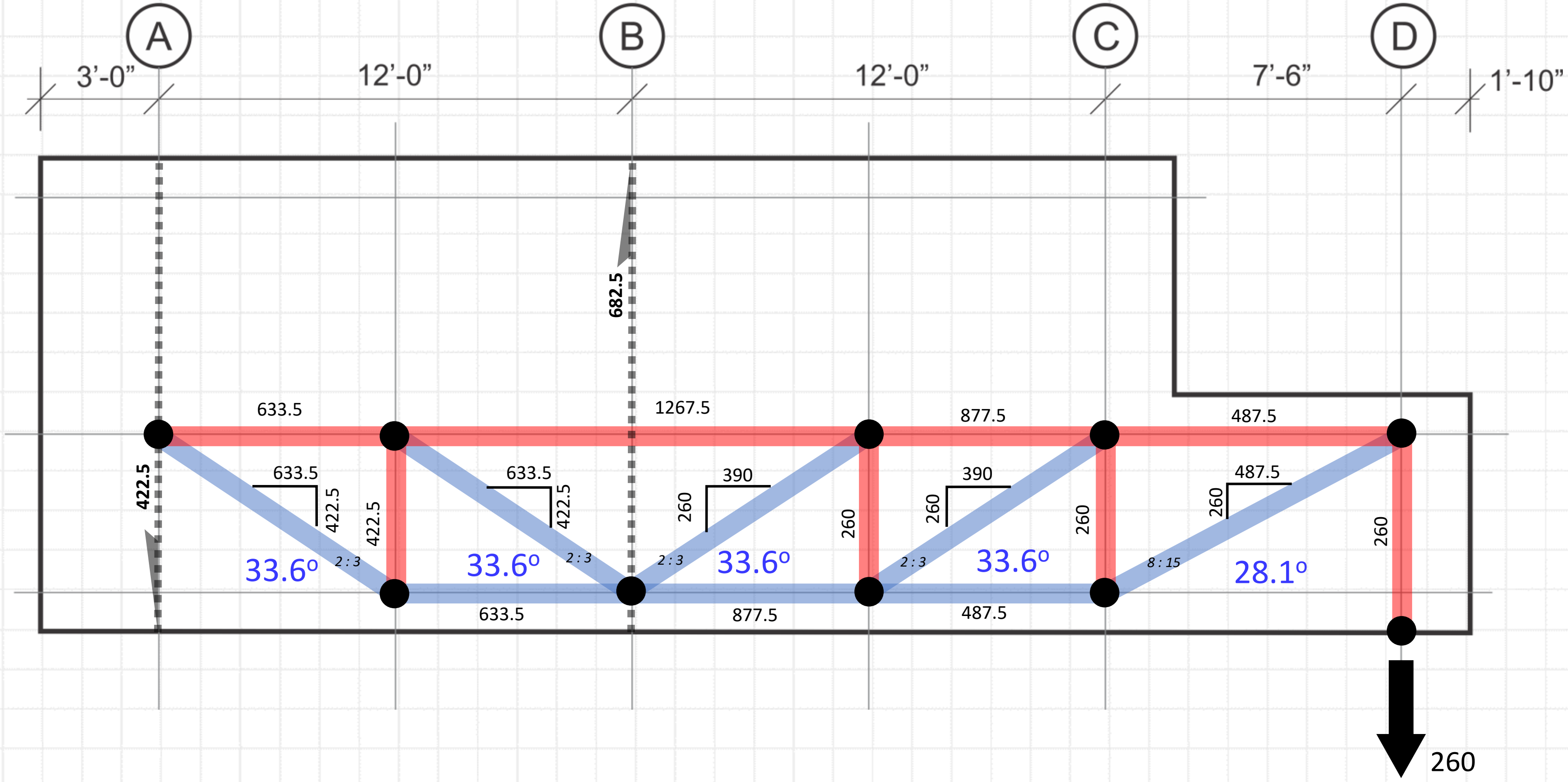
# Truss Models – Load Case 2




SEI= 1267.5x24 + 487.5x7.5 + 260x4 + 260x5 = 36,416

**23.2.7** The angle between the axes of any strut and any tie entering a single node shall be at least 25 degrees.

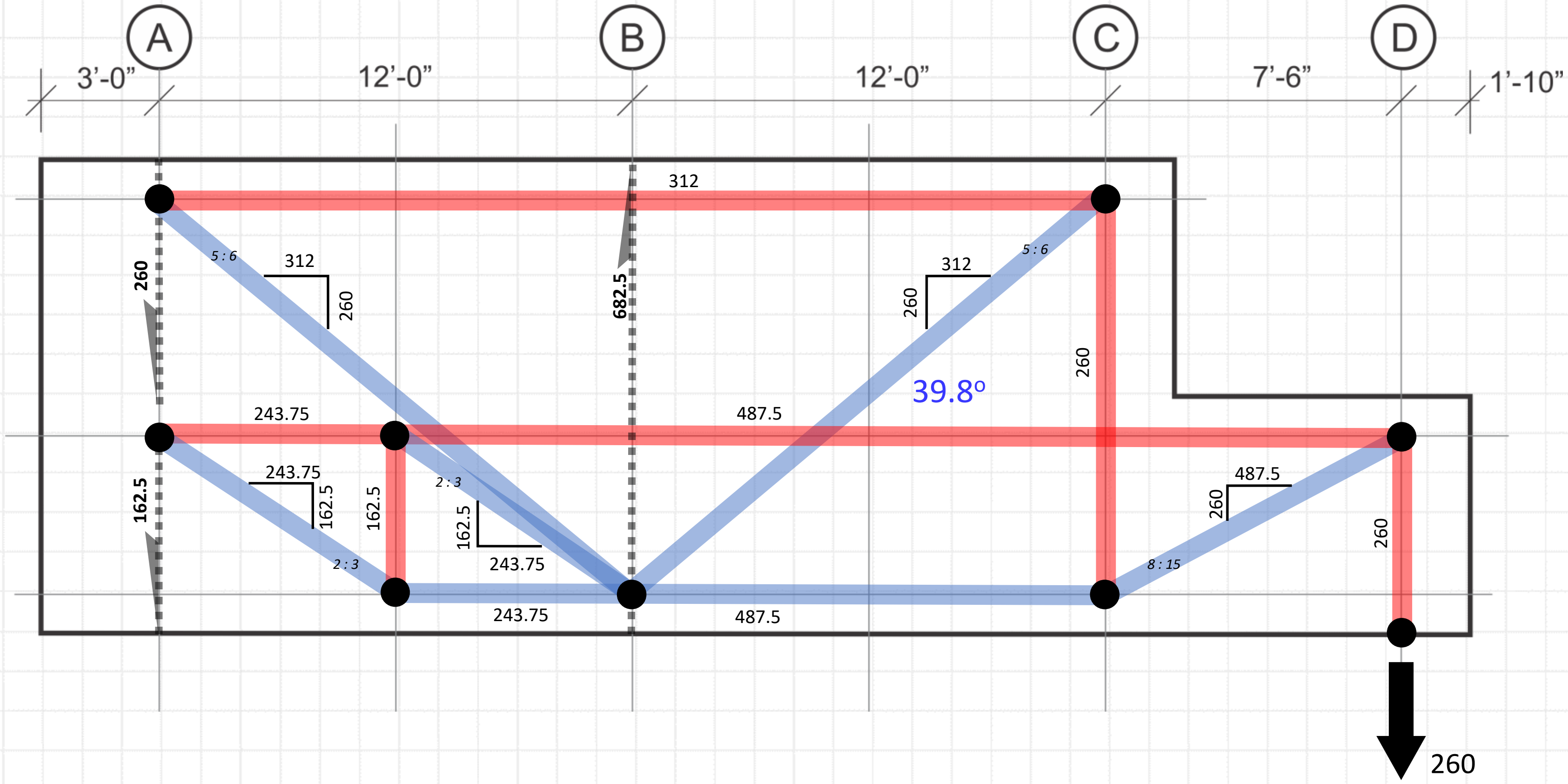
# Truss Models – Load Case 2




Strain energy index

SEI= 633.5x6 + 1267.5x12 + 877.5x6 + 487.5x7.5 + 422.5x4 + 260x4x2 + 260x5 = 33,002

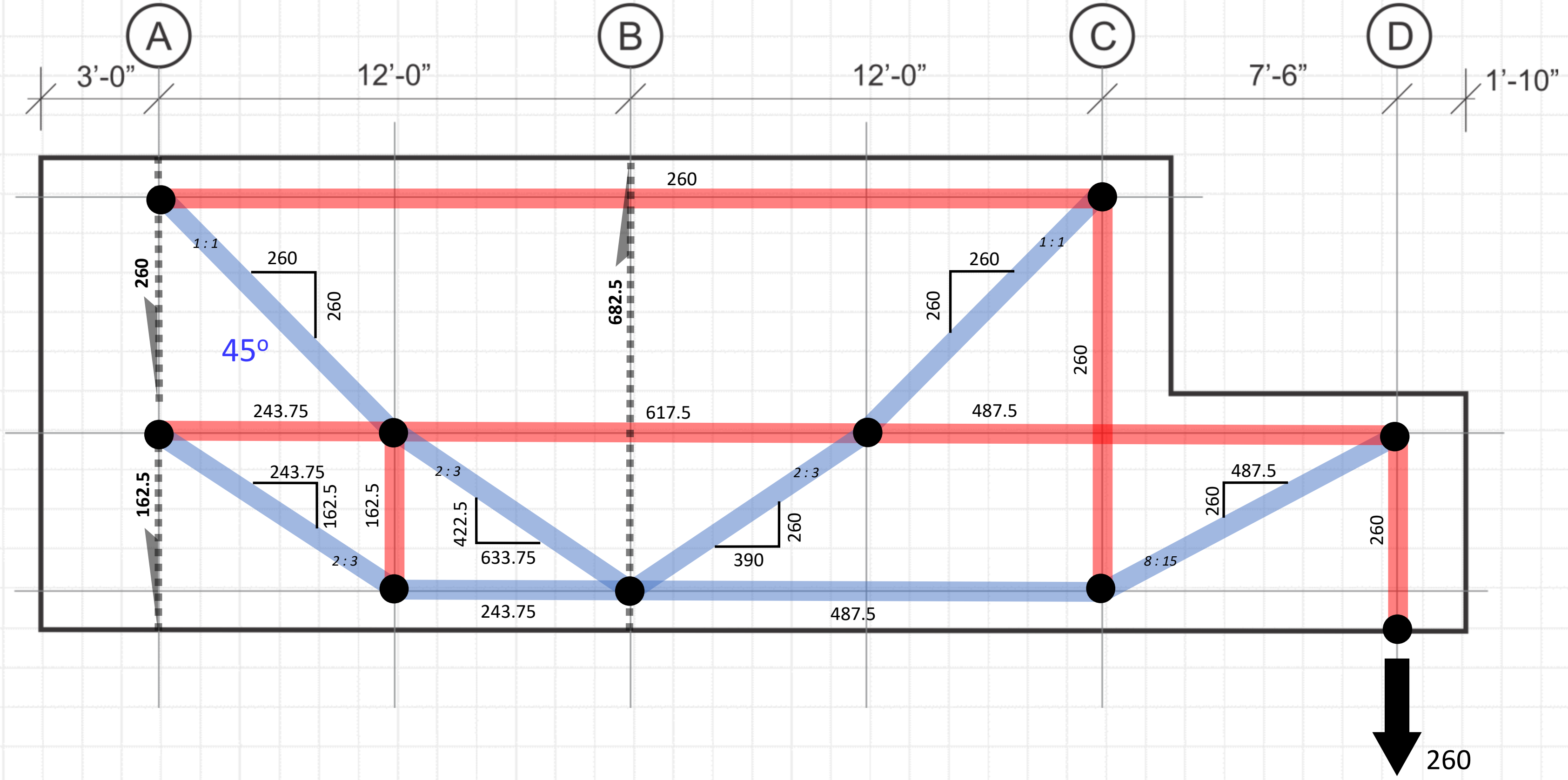
# Truss Models – Load Case 2




Strain energy index

SEI= 312x24 + 243.75x6+ 487.5x25.5 + 162.5x4 + 260x10 + 260x5 = 25,932

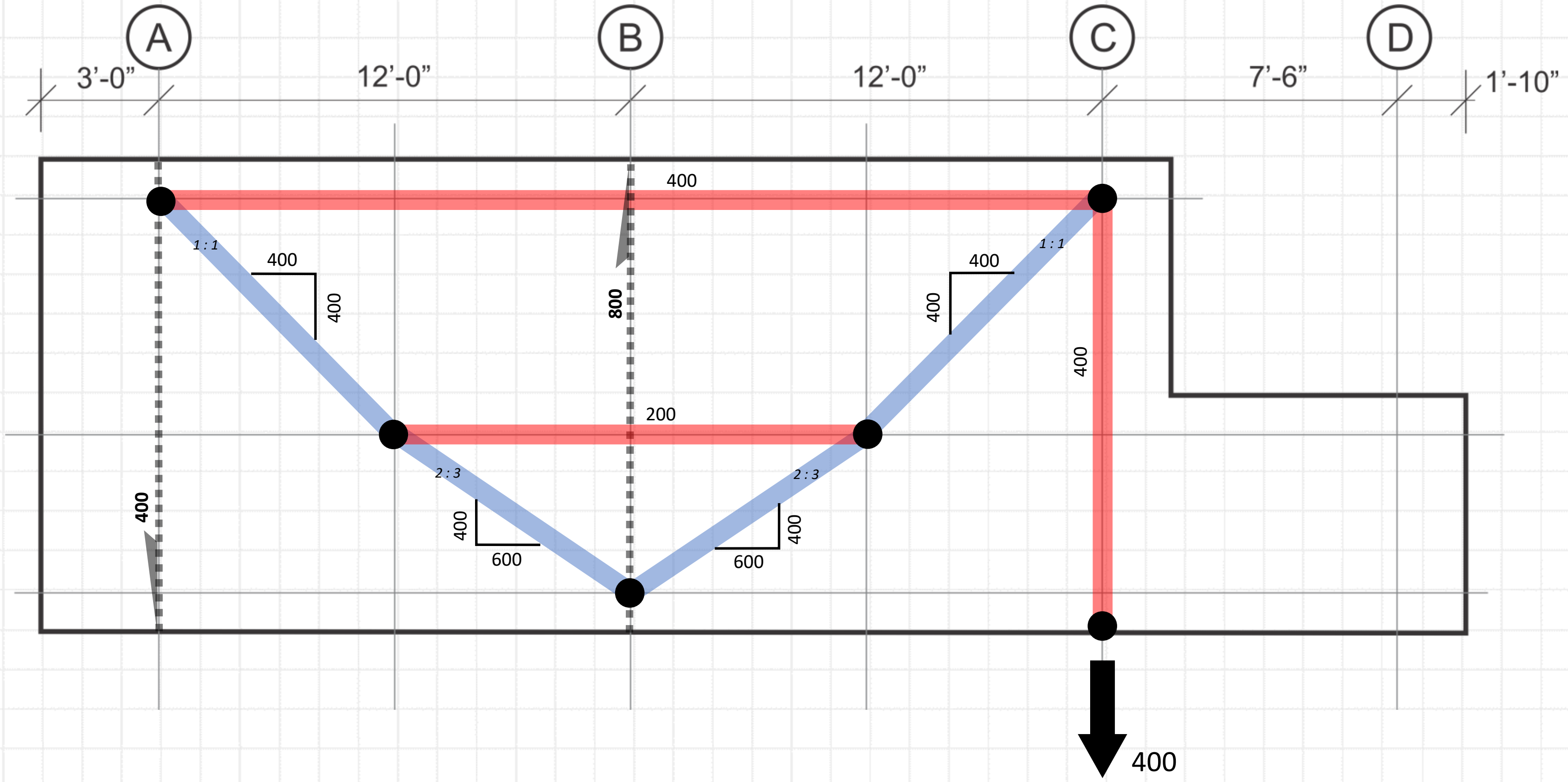
# Truss Models – Load Case 2




Strain energy index

SEI= 260x24 + 243.75x6 + 617.5x12 + 487.5x13.5 + 162.5x4 + 260x10 + 260x5 = 26,244

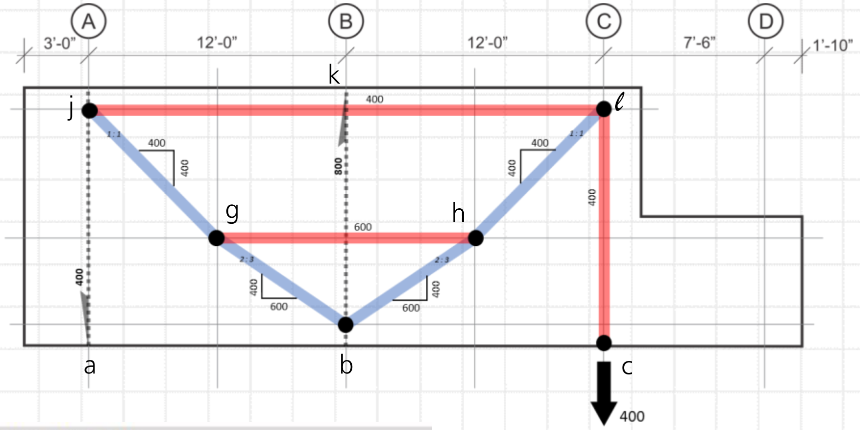
# Truss Models – Revisiting the Truss Model for Load Case 1

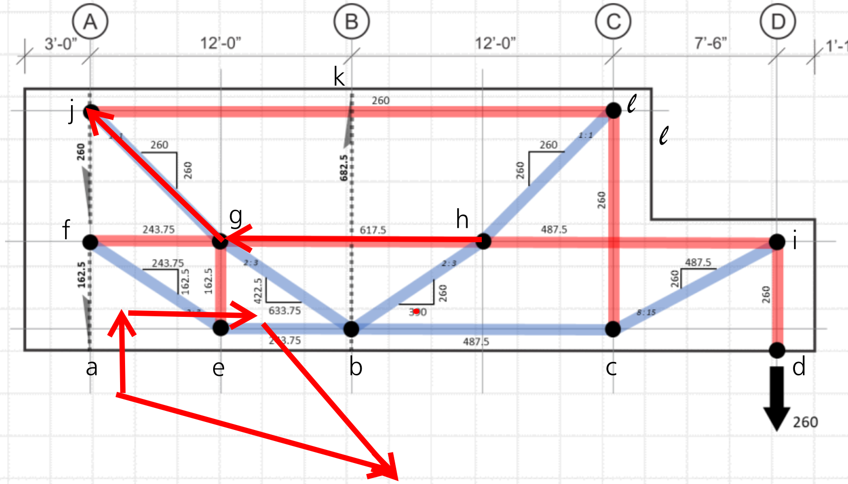



Strain energy index

SEI= 400x24 + 600x12 = 16,800

# Tie Design




**23.3—Design strength**

**23.3.1** For each applicable factored load combination, design strength of each strut, tie, and nodal zone in a strut-and-tie model shall satisfy $\phi S_n \geq U$, including (a) through (c):

(a) Struts: $\phi F_{ns} \geq F_{us}$

(b) Ties: $\phi F_{nt} \geq F_{ut}$

(c) Nodal zones: $\phi F_{nn} \geq F_{us}$

**23.3.2** $\phi$ shall be in accordance with 21.2.

**23.7—Strength of ties**

**23.7.1** Tie reinforcement shall be nonprestressed or prestressed.

**23.7.2** The nominal tensile strength of a tie, $F_{nt}$, shall be calculated by:

$$F_{nt} = A_{ts} f_y + A_{tp} \Delta f_p \qquad (23.7.2)$$

where $A_{tp}$ is zero for nonprestressed members.

| Imperial Bar Size | Nominal Diameter (in) | Nominal Area(in$^2$) |
|---|---|---|
| #5 | 0.625 | 0.31 |
| #6 | 0.750 | 0.44 |
| #7 | 0.875 | 0.6 |
| #8 | 1.000 | 0.79 |
| #9 | 1.128 | 1 |
| #11 | 1.41 | 1.56 |

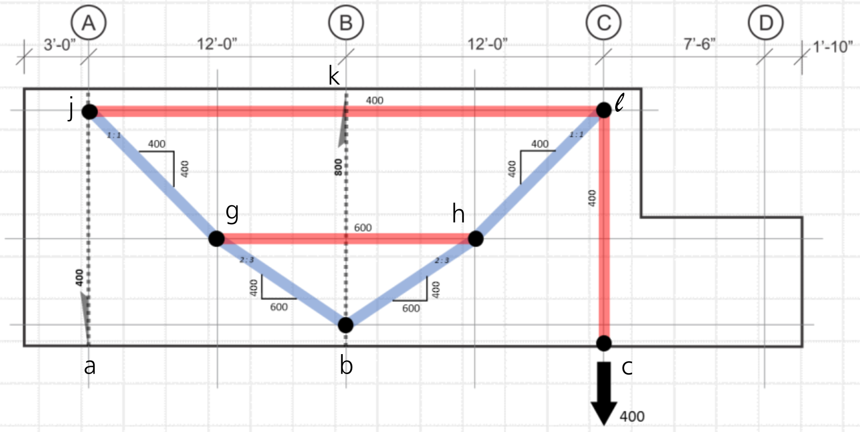
https://www.harrissupplysolutions.com/steel-rebar-sizes-stock.html

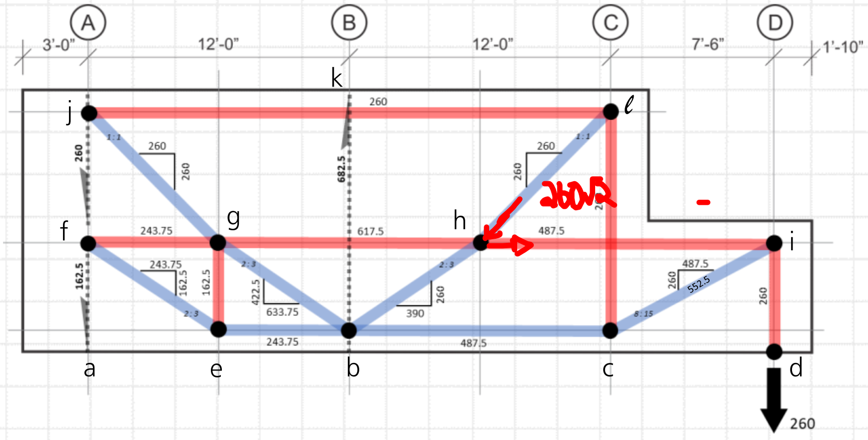
$$\frac{F_{ut}}{\phi f_y}$$

| Member | $T_{u\,LC1}$ (kip) | $T_{u\,LC2}$ (kip) | $F_{ut}$ (kip) | $A_{s,required}$ (in$^2$) | Bar Array | $A_{s,provided}$ (in$^2$) |
|---|---|---|---|---|---|---|
| … | | | | | | |
| *id* | - | 260 | 260 | 5.78 | 6#9 | 6.00 |
| *ih* | - | 487.5 | 487.5 | 10.83 | 8#11 | 12.48 |

⋮

# Strut Design




### 23.3—Design strength

**23.3.1** For each applicable factored load combination, design strength of each strut, tie, and nodal zone in a strut-and-tie model shall satisfy $\phi S_n \geq U$, including (a) through (c):

(a) Struts: $\phi F_{ns} \geq F_{us}$
(b) Ties: $\phi F_{nt} \geq F_{ut}$
(c) Nodal zones: $\phi F_{nn} \geq F_{us}$

**23.3.2** $\phi$ shall be in accordance with 21.2.

### 23.4—Strength of struts

**23.4.1** The nominal compressive strength of a strut, $F_{ns}$, shall be calculated by (a) or (b):

(a) Strut without longitudinal reinforcement

$$F_{ns} = f_{ce} A_{cs} \qquad (23.4.1a)$$

(b) Strut with longitudinal reinforcement

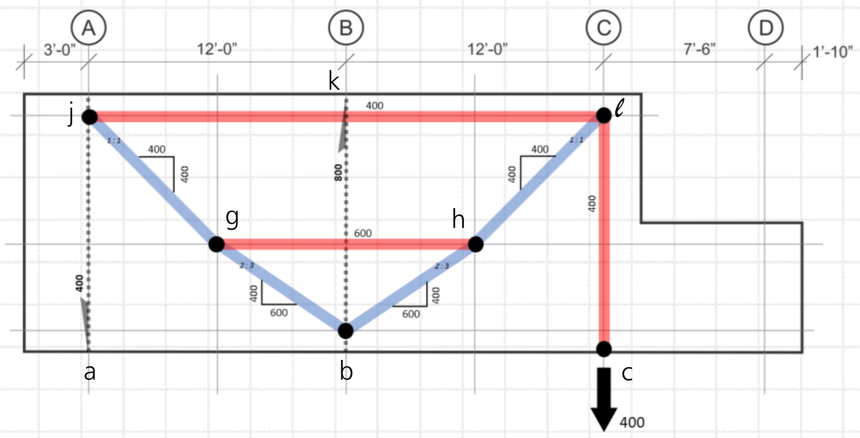
$$F_{ns} = f_{ce} A_{cs} + A_s' f_s' \qquad (23.4.1b)$$

**23.4.3** Effective compressive strength of concrete in a strut, $f_{ce}$, shall be calculated by:

$$f_{ce} = 0.85\, \beta_c \beta_s f_c' \qquad (23.4.3)$$

where $\beta_s$ is in accordance with Table 23.4.3(a) and $\beta_c$ is in accordance with Table 23.4.3(b).

# Nodal Zone Design



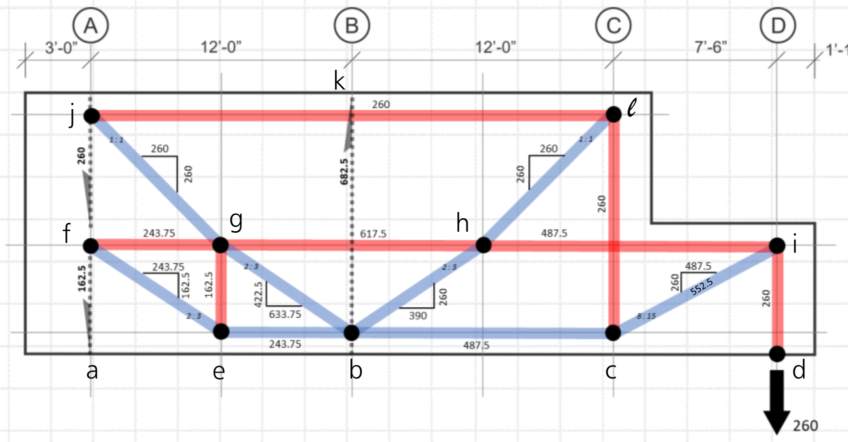




**Table 23.4.3(a)—Strut coefficient $\beta_s$**

| Strut location | Strut type | Criteria | $\beta_s$ | |
|---|---|---|---|---|
| Tension members or tension zones of members | Any | All cases | 0.4 | (a) |
| All other cases | Boundary struts | All cases | 1.0 | (b) |
| | Interior struts | Reinforcement satisfying (a) or (b) of Table 23.5.1 | 0.75 | (c) |
| | | Located in regions satisfying 23.4.4 | 0.75 | (d) |
| | | Beam-column joints | 0.75 | (e) |
| | | All other cases | 0.4 | (f) |

**Table 23.4.3(b)—Strut and node confinement modification factor $\beta_c$**

| Location | $\beta_c$ | | |
|---|---|---|---|
| End of a strut connected to a node that includes a bearing surface<br>Node that includes a bearing surface | Lesser of | $\sqrt{A_2/A_1}$<br>where $A_1$ is defined by the bearing surface | (a) |
| | | 2.0 | (b) |
| Other cases | 1.0 | | (c) |

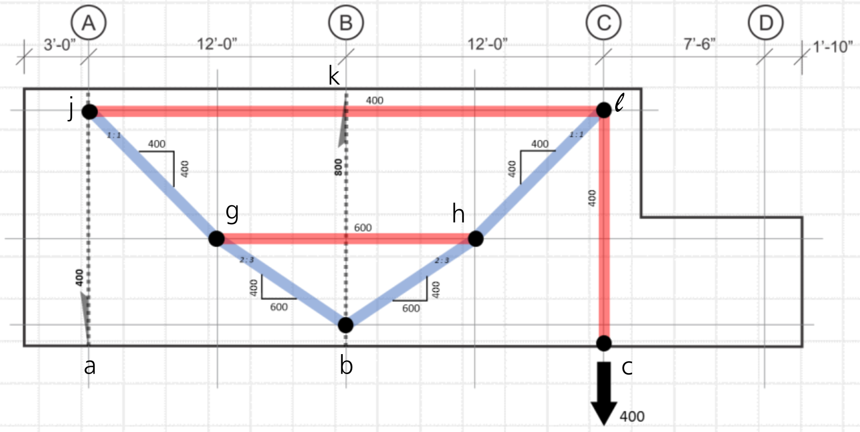
$f'_c = 5{,}000\,psi,\ t = 18"$

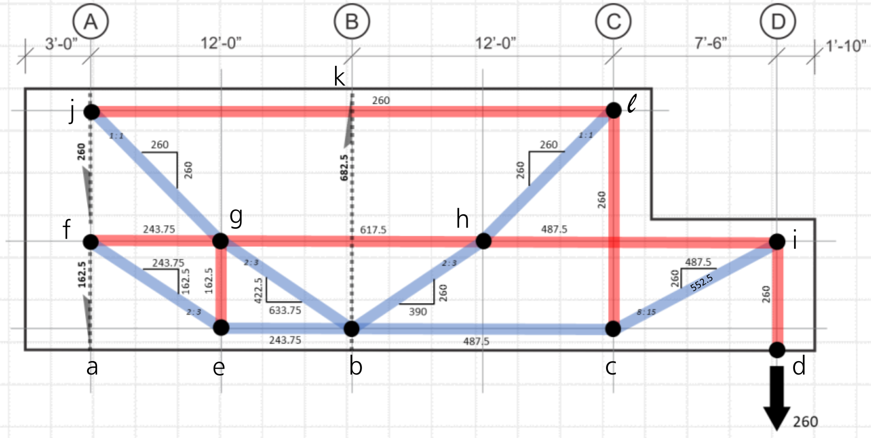
$f_{ce} = 0.85\beta_s\beta_c f'_c$ $\quad w_{\min} = \dfrac{F_{us}}{\phi\, t\, f_{ce}}$

| Member | $C_{u\,LC1}$ (kip) | $C_{u\,LC2}$ (kip) | $F_{us}$ (kip) | Strut type Table 23.4.3(a) | $\beta_s$ | Strut type Table 23.4.3(b) | $\beta_c$ | $f_{ce}$ (psi) | Strut's min. width, $w_{min}$ (in.) |
|---|---|---|---|---|---|---|---|---|---|
| … | | | | | | | | | |
| … | | | | | | | | | |
| ci | - | 552.5 | 552.5 | (c) | 0.75 | (c) | 1.0 | 3,188 | 9.6 |

⋮

# Nodal Zone Design




**23.3—Design strength**

**23.3.1** For each applicable factored load combination, design strength of each strut, tie, and nodal zone in a strut-and-tie model shall satisfy $\phi S_n \geq U$, including (a) through (c):

(a) Struts: $\phi F_{ns} \geq F_{us}$

(b) Ties: $\phi F_{nt} \geq F_{ut}$

(c) Nodal zones: $\phi F_{nn} \geq F_{us}$

**23.3.2** $\phi$ shall be in accordance with 21.2.

**23.9—Strength of nodal zones**

**23.9.1** The nominal compressive strength of a nodal zone, $F_{nn}$, shall be calculated by:

$$F_{nn} = f_{ce} A_{nz} \qquad (23.9.1)$$

where $f_{ce}$ is defined in 23.9.2 or 23.9.3 and $A_{nz}$ is given in 23.9.4 or 23.9.5.

**23.9.2** The effective compressive strength of concrete at a face of a nodal zone, $f_{ce}$, shall be calculated by:

$$f_{ce} = 0.85 \beta_c \beta_n f_c' \qquad (23.9.2)$$

where $\beta_n$ shall be in accordance with Table 23.9.2 and $\beta_c$ is in accordance with Table 23.4.3(b).

**Table 23.4.3(b)—Strut and node confinement modification factor $\beta_c$**

| Location | $\beta_c$ | | |
|---|---|---|---|
| End of a strut connected to a node that includes a bearing surface<br>Node that includes a bearing surface | Lesser of | $\sqrt{A_2/A_1}$<br>where $A_1$ is defined by the bearing surface | (a) |
| | | 2.0 | (b) |
| Other cases | | 1.0 | (c) |

**Table 23.9.2—Nodal zone coefficient $\beta_n$**

| Configuration of nodal zone | $\beta_n$ | |
|---|---|---|
| Nodal zone bounded by struts, bearing areas, or both | 1.0 | (a) |
| Nodal zone anchoring one tie | 0.80 | (b) |
| Nodal zone anchoring two or more ties | 0.60 | (c) |

$f_c' = 5{,}000\,psi,\ t = 18"$

$f_{ce} = 0.85 \beta_c \beta_n f_c'$

| Node | Node type Table 23.4.3(b) | $\beta_c$ | Node type Table 23.9.2 | $\beta_n$ | $f_{ce}$ (psi) |
|---|---|---|---|---|---|
| ... | | | | | |
| ... | | | | | |
| i | (c) | 1.0 | (c) | 0.6 | 2,550 |

•
•
•

# Design & Detailing of Nodal Zone i – Step 1: Sizing – Trial 1

## Strategy,

$$f_{ce} = \min\left( f_{ce,nodal\,zone},\ f_{ce,strut1}, \ldots, f_{ce,strut\,n} \right)$$

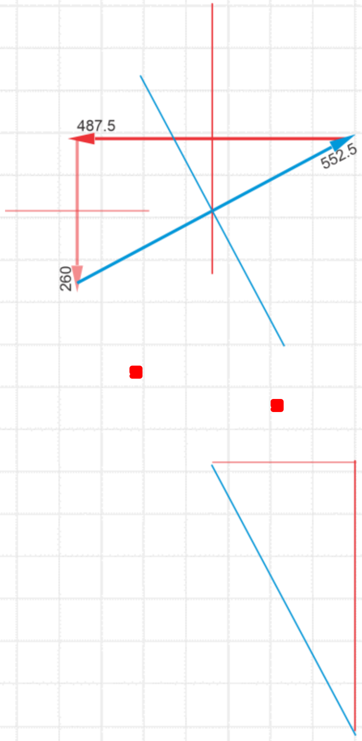
for Nodal Zone i,

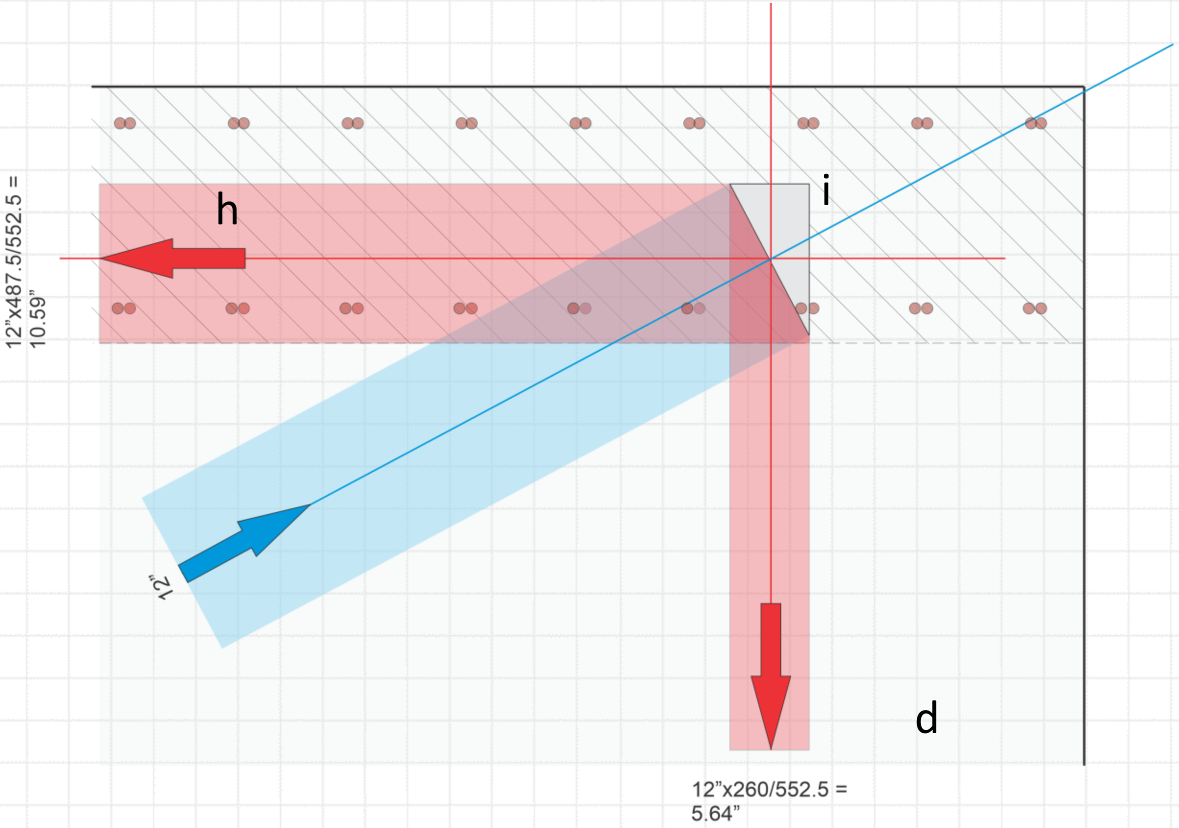
$$f_{ce} = \min\left(2,550,\ 3,188\right) = 2,550\ psi$$

Trial 1: strut ci width,

$$w_{s,ci} = \frac{552.5kip}{2.55ksi \times 18in} = 12in$$

Force polygon and joint construction

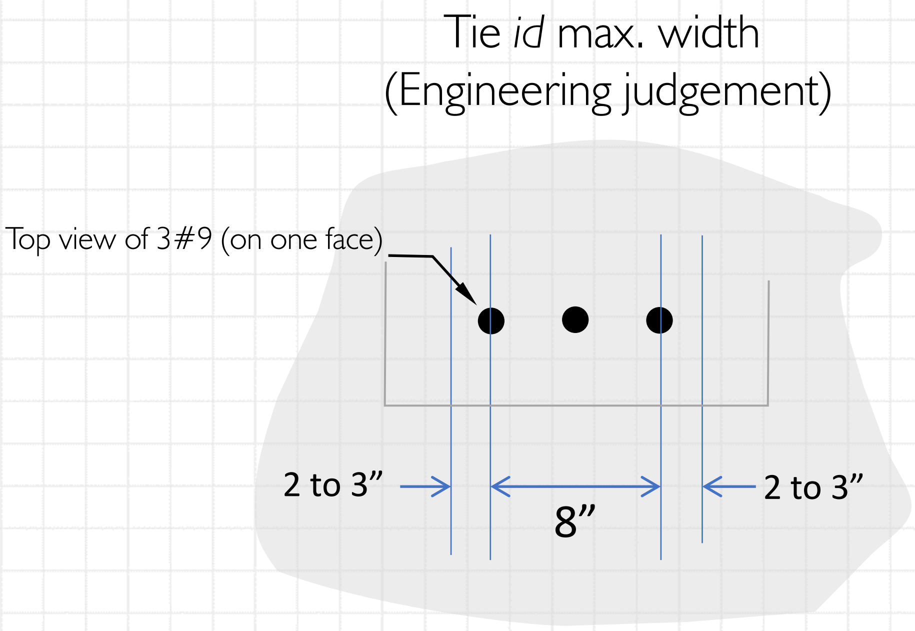


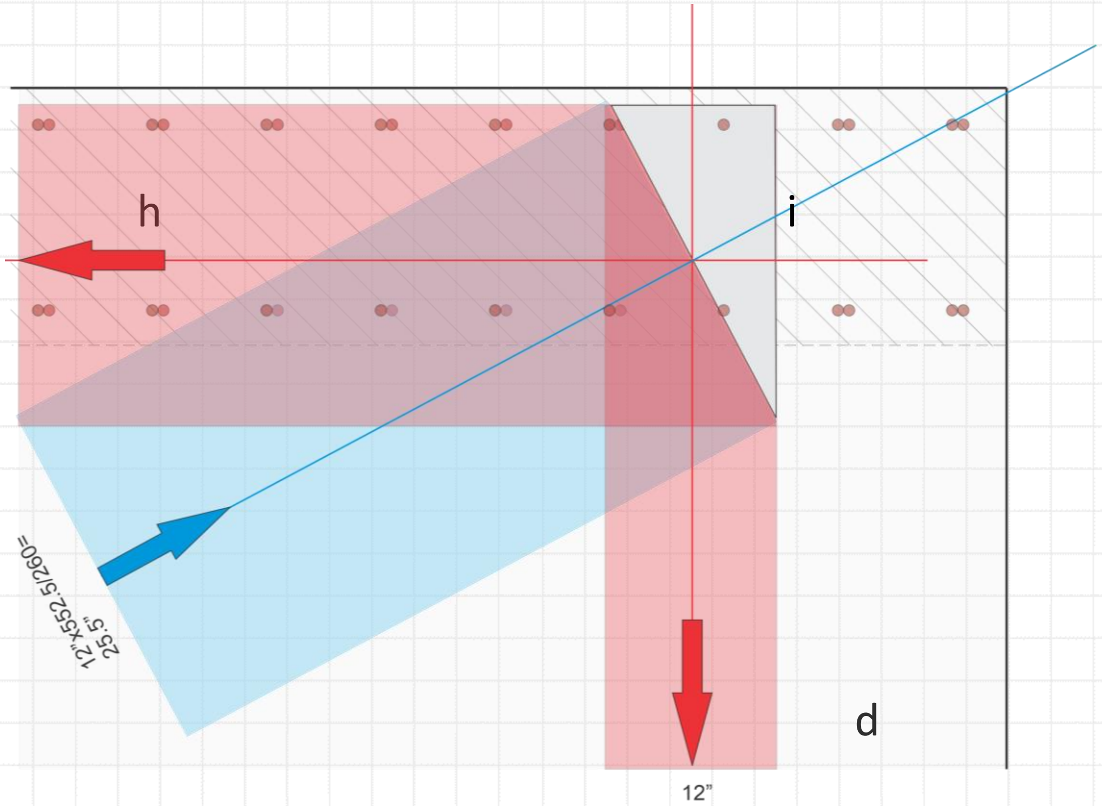




-Trial fails, we can't have the 6#9 bars within a 5.64" wide tie *id*

-The extreme #9 bar array are spaced 8" o.c., hence, a 10-12" tie width is deemed suitable. Do a second try starting with a 10" wide tie.

# Design & Detailing of Nodal Zone i – Step 1: Sizing – Trial 2




-By choosing 12” for the vertical strut *di*, we increased the size of the joint and of all other ties and struts. The joint and strut stresses decreased as a result

-We need to examine every other tie and ensure the width is appropriate to accommodate the tie’s reinforcement. Further trials may be needed in some cases. Here, the horizontal tie *ih* is 18.75” wide and can easily accommodate the 8#11 bars placed in four layers of 2#11

# Design & Detailing of Nodal Zone i – Step 1: Sizing – Nodal Zone Strength Check

**23.3—Design strength**

**23.3.1** For each applicable factored load combination, design strength of each strut, tie, and nodal zone in a strut-and-tie model shall satisfy $\phi S_n \geq U$, including (a) through (c):

(a) Struts: $\phi F_{ns} \geq F_{us}$
(b) Ties: $\phi F_{nt} \geq F_{ut}$
(c) Nodal zones: $\phi F_{nn} \geq F_{us}$

**23.3.2** $\phi$ shall be in accordance with 21.2.

**23.9—Strength of nodal zones**

**23.9.1** The nominal compressive strength of a nodal zone, $F_{nn}$, shall be calculated by:

$$F_{nn} = f_{ce} A_{nz} \tag{23.9.1}$$

where $f_{ce}$ is defined in 23.9.2 or 23.9.3 and $A_{nz}$ is given in 23.9.4 or 23.9.5.

**23.9.2** The effective compressive strength of concrete at a face of a nodal zone, $f_{ce}$, shall be calculated by:

$$f_{ce} = 0.85\beta_c\beta_n f_c' \tag{23.9.2}$$

where $\beta_n$ shall be in accordance with Table 23.9.2 and $\beta_c$ is in accordance with Table 23.4.3(b).

**Table 23.4.3(b)—Strut and node confinement modification factor $\beta_c$**

| Location | $\beta_c$ | | |
|---|---|---|---|
| End of a strut connected to a node that includes a bearing surface<br>Node that includes a bearing surface | Lesser of | $\sqrt{A_2/A_1}$<br>where $A_1$ is defined by the bearing surface | (a) |
| | | 2.0 | (b) |
| Other cases | 1.0 | | (c) |

**Table 23.9.2—Nodal zone coefficient $\beta_n$**

| Configuration of nodal zone | $\beta_n$ | |
|---|---|---|
| Nodal zone bounded by struts, bearing areas, or both | 1.0 | (a) |
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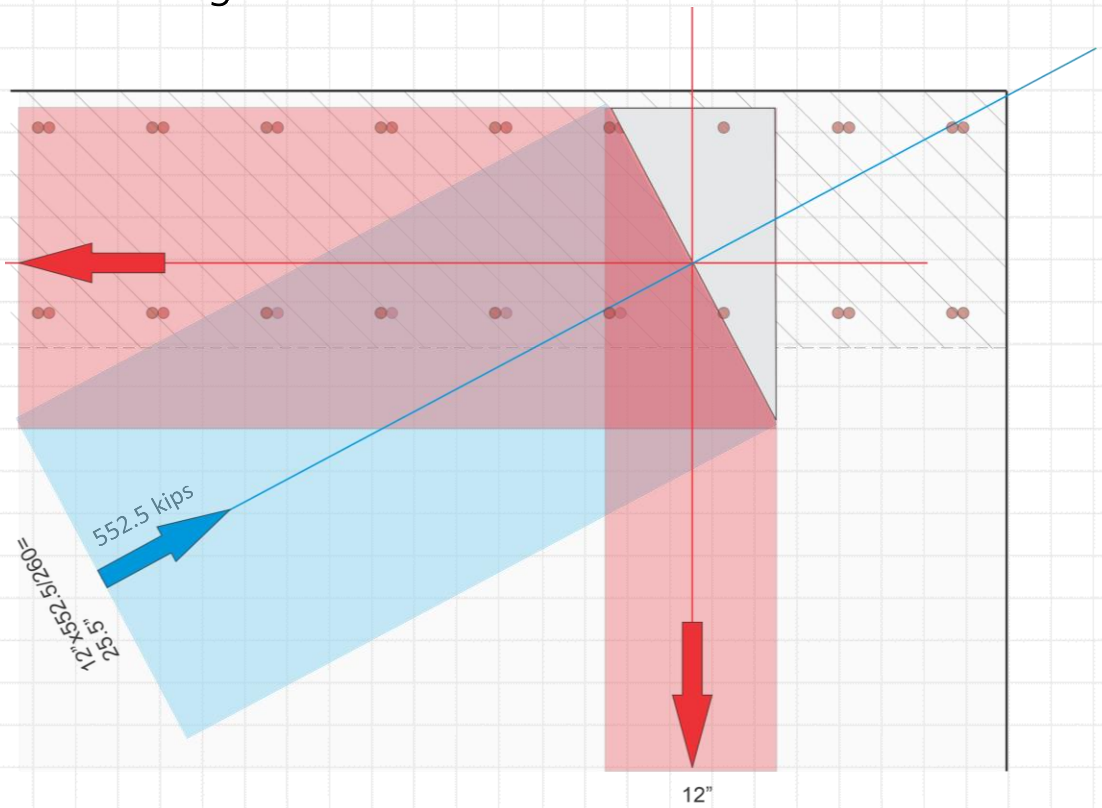



$f_c' = 5{,}000\,psi,\ t = 18"$

$A_{nz} = 25.5" \times 18" = 459 in^2$

Effective concrete compressive strength,

$f_{ce} = 0.85\beta_c\beta_n f_c' = 0.85 \times 1.0 \times 0.6 \times 5{,}000 = 2{,}550\,psi$

$\phi F_{nn} = 0.75 \times 2{,}550\,psi \times 459 in^2 = 878 kips$

$\phi F_{nn} \geq F_{us} = 552.5 kips$

# Design & Detailing of Nodal Zone i – Step 3: Bar Development

Vertical bars - tie *id*

**25.4.4.2** Development length $\ell_{dt}$ for headed deformed bars in tension shall be the longest of (a) through (c):

(a) $\left(\dfrac{f_y \psi_e \psi_p \psi_o \psi_c}{75\sqrt{f'_c}}\right) d_b^{1.5}$ with $\psi_e$, $\psi_p$, $\psi_o$, and $\psi_c$, given in 25.4.4.3

(b) $8d_b$

(c) 6 in.

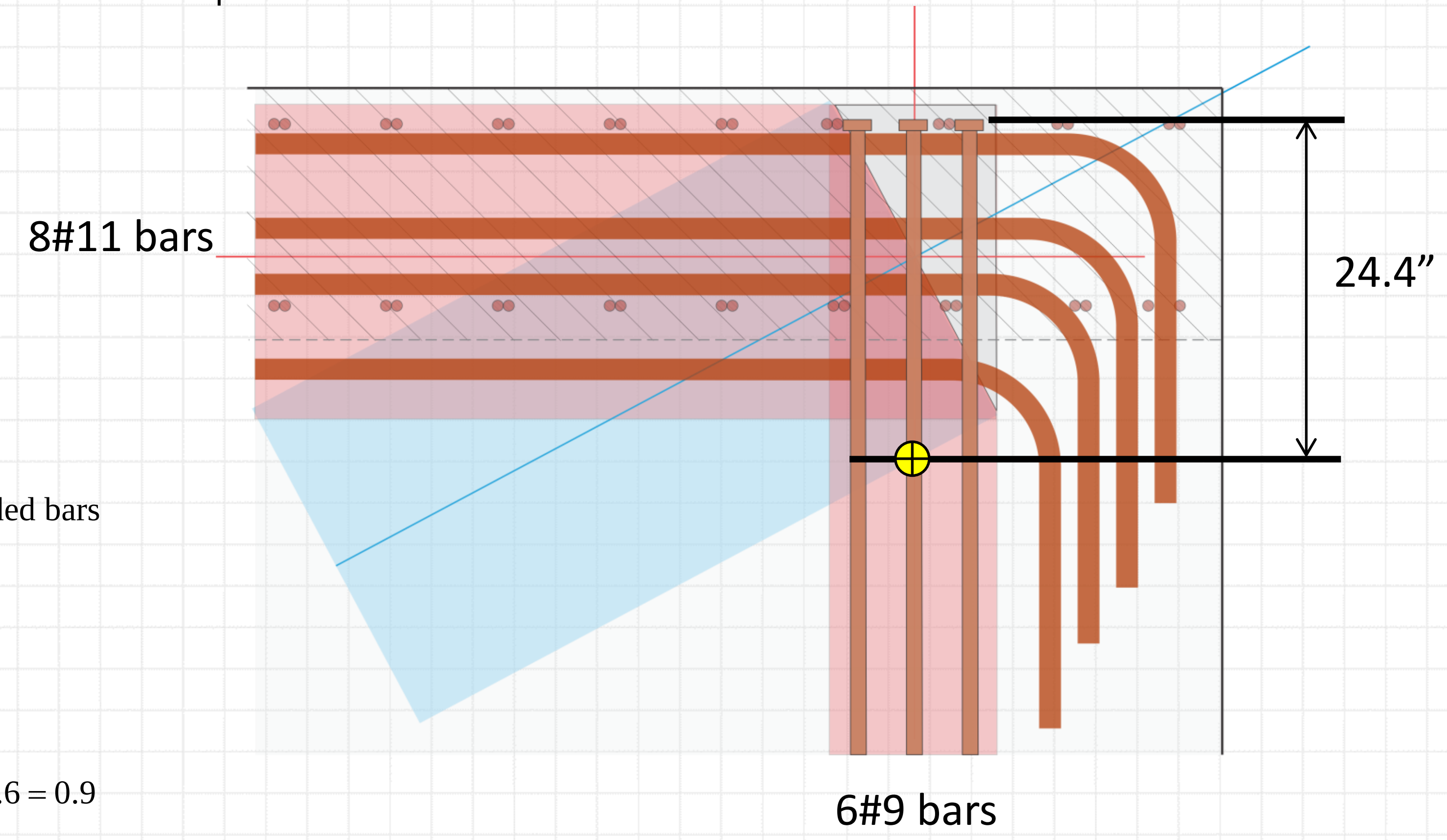
**Table 25.4.4.3—Modification factors for development of headed bars in tension**

| Modification factor | Condition | Value of factor |
|---|---|---|
| Epoxy $\psi_e$ | Epoxy-coated or zinc and epoxy dual-coated reinforcement | 1.2 |
| | Uncoated or zinc-coated (galvanized) reinforcement | 1.0 |
| Parallel tie reinforcement $\psi_p$ | For No. 11 and smaller bars with $A_{tt} \geq 0.3A_{hs}$ or $s$[1] $\geq 6d_b$[2,3] | 1.0 |
| | Other | 1.6 |
| Location $\psi_o$ | For headed bars: (1) Terminating inside column core with side cover to bar $\geq$ 2.5 in.; or (2) With side cover to bar $\geq 6d_b$ | 1.0 |
| | Other | 1.25 |
| Concrete strength $\psi_c$ | For $f'_c$ < 6000 psi | $f'_c/15{,}000 + 0.6$ |
| | For $f'_c \geq$ 6000 psi | 1.0 |

[1] $s$ is minimum center-to-center spacing of headed bars.

[2] $d_b$ is nominal diameter of headed bar.

[3] Refer to 25.4.4.5.




Vertical #8 headed bars

Required

$\psi_e = 1.0$

$\psi_p = 1.6$

$\psi_o = 1.25$

$$\psi_c = \frac{5,000}{15,000} + 0.6 = 0.9$$

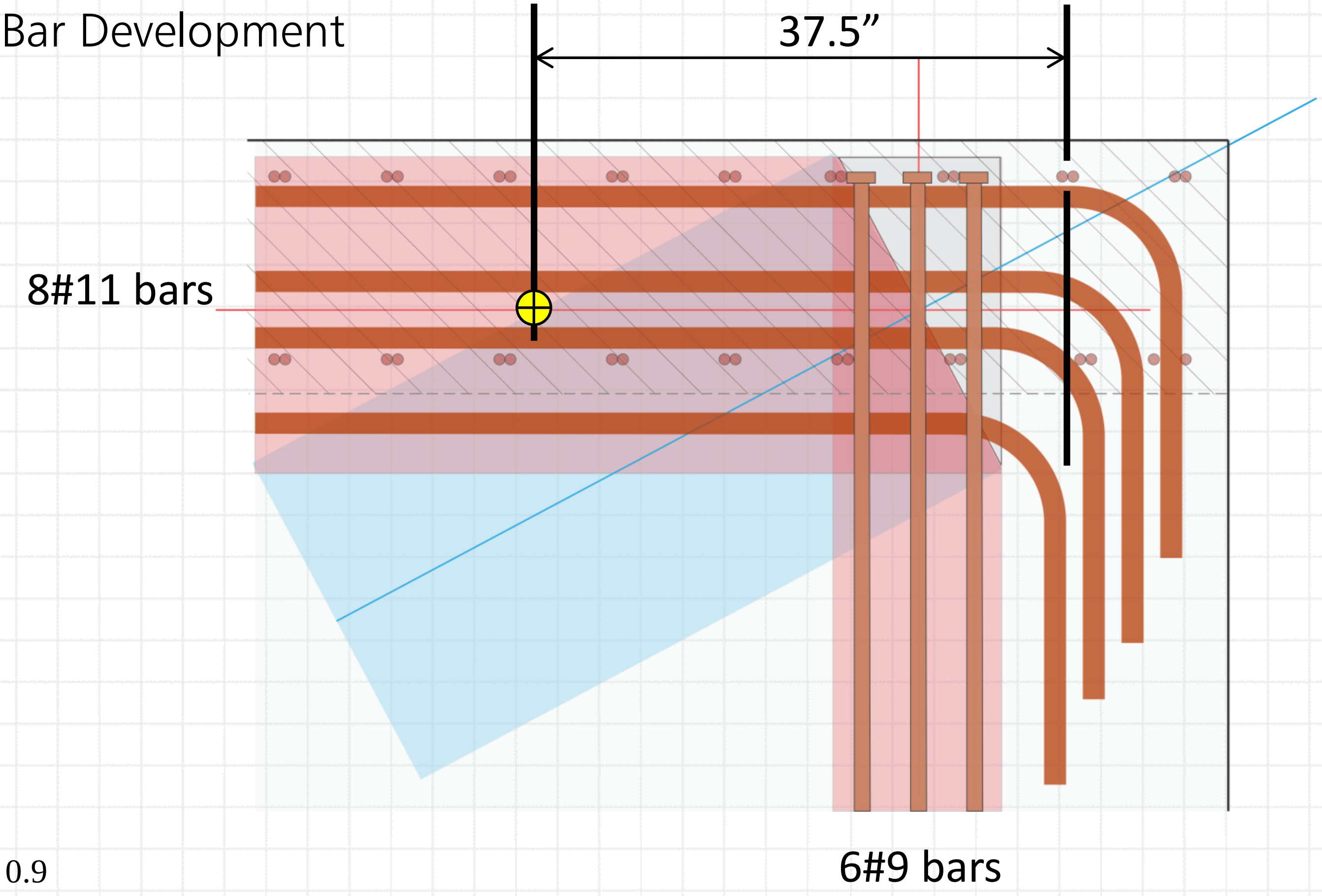
$$\ell_{dt,req} = \max\left(8\times 1in,\ 6in,\ \frac{60,000\times 1.0\times 1.6\times 1.25\times 0.9}{75\sqrt{5,000}}(1.125)^{1.5}\ in\right) =$$

$$\ell_{dt,req} = \max(8in,\ 6in,\ 20.4in) = 24.3in$$

Provided

$\ell_{dt,pro} = 24.4" > \ell_{dt,req}$

# Design & Detailing of Nodal Zone i – Step 4: Bar Development




## Horizontal bars - tie *ih*

**25.4.3** *Development of standard hooks in tension*

**25.4.3.1** Development length $\ell_{dh}$ for deformed bars in tension terminating in a standard hook shall be the greater of (a) through (c):

(a) $\left(\dfrac{f_y \psi_e \psi_r \psi_o \psi_c}{55\lambda\sqrt{f_c'}}\right) d_b^{1.5}$ with $\psi_e$, $\psi_r$, $\psi_o$, $\psi_c$, and $\lambda$ given in 25.4.3.2

(b) **8**$d_b$

(c) 6 in.

**25.4.3.2** For the calculation of $\ell_{dh}$, modification factors $\psi_e$, $\psi_r$, $\psi_o$, $\psi_c$, and $\lambda$ shall be in accordance with Table 25.4.3.2. At discontinuous ends of members, 25.4.3.4 shall apply.

**Table 25.4.3.2—Modification factors for development of hooked bars in tension**

| Modification factor | Condition | Value of factor |
|---|---|---|
| Lightweight $\lambda$ | Lightweight concrete | 0.75 |
| | Normalweight concrete | 1.0 |
| Epoxy $\psi_e$ | Epoxy-coated or zinc and epoxy dual-coated reinforcement | 1.2 |
| | Uncoated or zinc-coated (galvanized) reinforcement | 1.0 |
| Confining reinforcement $\psi_r$ | For No. 11 and smaller bars with $A_{th} \geq 0.4A_{hs}$ or $s^{[1]} \geq 6d_b^{[2]}$ | 1.0 |
| | Other | 1.6 |
| Location $\psi_o$ | For No. 11 and smaller diameter hooked bars: (1) Terminating inside column core with side cover normal to plane of hook ≥ 2.5 in., or (2) With side cover normal to plane of hook ≥ $6d_b$ | 1.0 |
| | Other | 1.25 |
| Concrete strength $\psi_c$ | For $f_c' < 6000$ psi | $f_c'/15{,}000 + 0.6$ |
| | For $f_c' \geq 6000$ psi | 1.0 |

[1] $s$ is minimum center-to-center spacing of hooked bars.
[2] $d_b$ is nominal diameter of hooked bar.

Horizontal #11 bars

Required

$$\lambda = 1.0$$

$$\psi_e = 1.0$$

$$\psi_p = 1.6$$

$$\psi_o = 1.25$$

$$\psi_c = \frac{5{,}000}{15{,}000} + 0.6 = 0.9$$

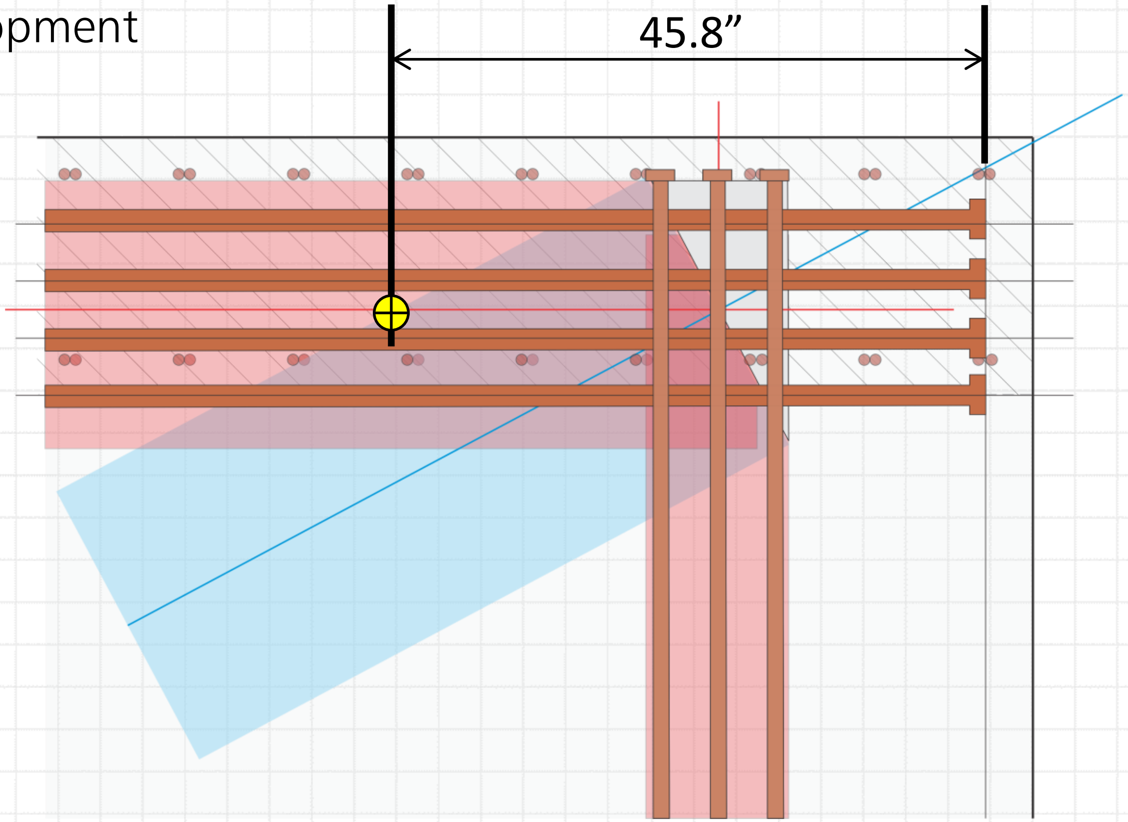
$$\ell_{dt,req} = \max\left(8\times 1in,\ 6in,\ \frac{60{,}000\times 1.0\times 1.6\times 1.25\times 0.9}{55\sqrt{5{,}000}}(1.375)^{1.5}\ in\right) =$$

$$\ell_{dt,req} = \max\left(8in,\ 6in,\ 44.8in\right) = 44.8in$$

Provided

$\ell_{dt,pro} = 37.5" \leq \ell_{dt,req} \rightarrow$ **Use headed reinforcement** ☹

# Design & Detailing of Nodal Zone i – Step 4: Bar Development




## Horizontal bars tie *ih*

**25.4.4.2** Development length $\ell_{dt}$ for headed deformed bars in tension shall be the longest of (a) through (c):

(a) $\left(\dfrac{f_y \psi_e \psi_p \psi_o \psi_c}{75\sqrt{f_c'}}\right) d_b^{1.5}$ with $\psi_e$, $\psi_p$, $\psi_o$, and $\psi_c$, given in 25.4.4.3

(b) $8d_b$

(c) 6 in.

**Table 25.4.4.3—Modification factors for development of headed bars in tension**

| Modification factor | Condition | Value of factor |
|---|---|---|
| Epoxy $\psi_e$ | Epoxy-coated or zinc and epoxy dual-coated reinforcement | 1.2 |
| | Uncoated or zinc-coated (galvanized) reinforcement | 1.0 |
| Parallel tie reinforcement $\psi_p$ | For No. 11 and smaller bars with $A_{tt} \geq 0.3A_{hs}$ or $s$[1] $\geq 6d_b$[2,3] | 1.0 |
| | Other | 1.6 |
| Location $\psi_o$ | For headed bars: (1) Terminating inside column core with side cover to bar $\geq$ 2.5 in.; or (2) With side cover to bar $\geq 6d_b$ | 1.0 |
| | Other | 1.25 |
| Concrete strength $\psi_c$ | For $f_c' < 6000$ psi | $f_c'/15{,}000 + 0.6$ |
| | For $f_c' \geq 6000$ psi | 1.0 |

[1] $s$ is minimum center-to-center spacing of headed bars.

[2] $d_b$ is nominal diameter of headed bar.

[3] Refer to 25.4.4.5.

Horizontal #11 headed bars

Required

$\psi_e = 1.0$

$\psi_p = 1.6$

$\psi_o = 1.25$

$$\psi_c = \frac{5{,}000}{15{,}000} + 0.6 = 0.9$$

$$\ell_{dt,req} = \max\left(8 \times 1in,\ 6in,\ \frac{60{,}000 \times 1.0 \times 1.6 \times 1.25 \times 0.9}{75\sqrt{5{,}000}}(1.375)^{1.5}\ in\right) =$$

$$\ell_{dt,req} = \max\left(8in,\ 6in,\ 32.8in\right) = 32.8in$$

Provided

$\ell_{dt,pro} = 45.8" > \ell_{dt,req}$

**25.2—Minimum spacing of reinforcement**

**25.2.1** For parallel nonprestressed reinforcement in a horizontal layer, clear spacing shall be at least the greatest of 1 in., $d_b$, and $(4/3)d_{agg}$.

**25.2.2** For parallel nonprestressed reinforcement placed in two or more horizontal layers, reinforcement in the upper layers shall be placed directly above reinforcement in the bottom layer with a clear spacing between layers of at least 1 in.

**25.2.3** For longitudinal reinforcement in columns, pedestals, struts, and boundary elements in walls, clear spacing between bars shall be at least the greatest of 1.5 in., $1.5d_b$, and $(4/3)d_{agg}$.

# Distributed Reinforcement for Crack Control

**23.5—Minimum distributed reinforcement**

**23.5.1** In D-regions designed using the strut-and-tie method, minimum distributed reinforcement shall be provided across the axes of interior struts in accordance with Table 23.5.1.

**Table 23.5.1—Minimum distributed reinforcement**

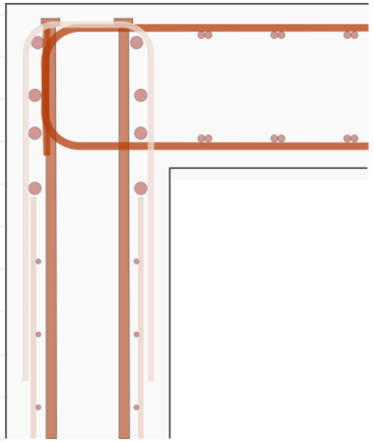
| Lateral restraint of strut | Reinforcement configuration | Minimum distributed reinforcement ratio | |
|---|---|---|---|
| Not restrained | Orthogonal grid | 0.0025 in each direction | (a) |
| | Reinforcement in one direction crossing strut at angle $\alpha_1$ | $\frac{0.0025}{\sin^2\alpha_1}$ | (b) |
| Restrained | Distributed reinforcement not required | | (c) |

**23.5.2** Distributed reinforcement required by 23.5.1 shall satisfy (a) and (b):

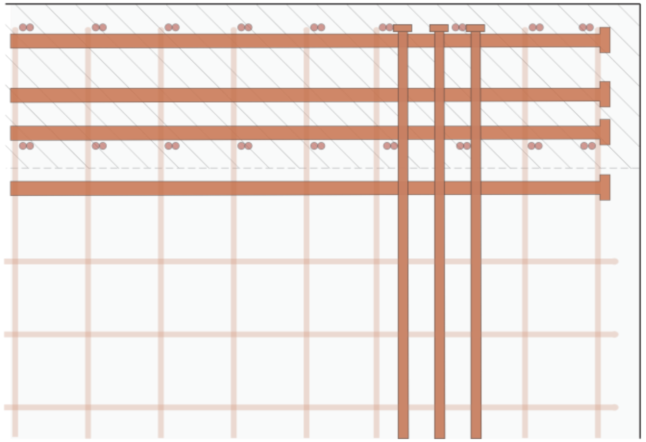
(a) Spacing shall not exceed 12 in.
(b) Angle $\alpha_1$ shall not be less than 40 degrees.

Distributed Reinforcement Around Nodal Zone i

Cross-section



Elevation





t = 18″ s = 8″ reinforcement in two directions on both faces,

Try #4@8" o.c. on both faces, vertical and horizontal,

$$\text{reinforcement ratio} = \frac{2\times 0.2\text{in}^2}{8"\times 18"} = 0.0028 \geq 0.0025$$

# Embed Plate Design

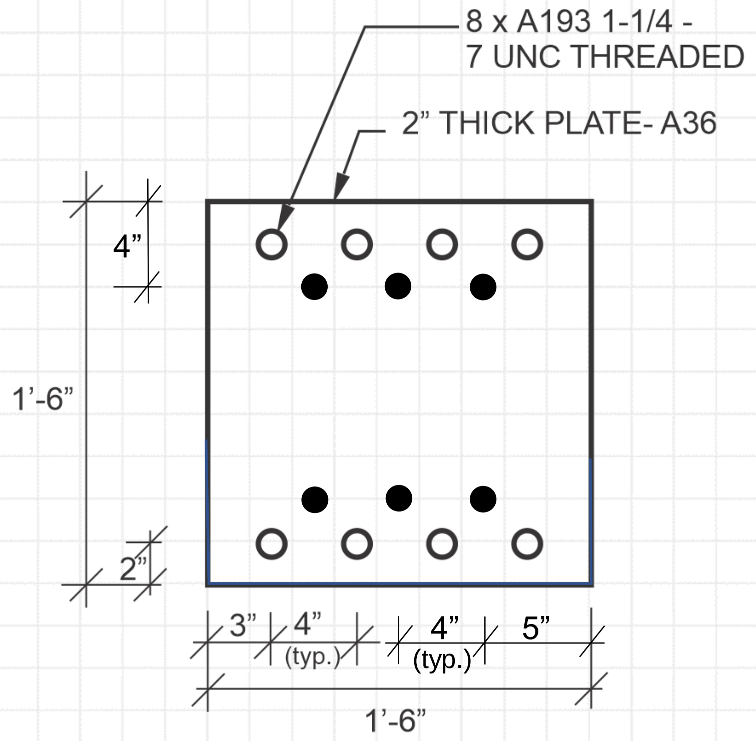



PLATE d

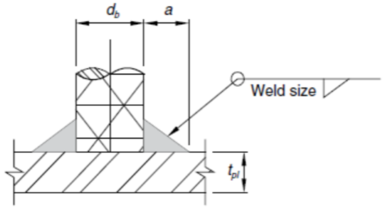
**Design Aid 6.15.4 Size of Fillet Weld Required to Develop Full Strength of Bar. Butt Weld.**

**BAR PERPENDICULAR TO PLATE, WELDED ONE SIDE**

$\ell_w = \pi\left(d_b + \frac{a}{2}\right)$

Plate = $F_y$ = 36 ksi

Plate area = $\pi(d_b + 2_a)t_{pl}$




| Bar size, # | E70 electrode | | E80 electrode[a] | | E90 electrode[a] | |
|---|---|---|---|---|---|---|
| | Weld size,[b] in. | Minimum plate thickness,[c] in. | Weld size,[b] in. | Minimum plate thickness,[c] in. | Weld size,[b] in. | Minimum plate thickness[c] $t_{pl}$, in. |
| Grade 60 bar[d] | | | | | | |
| 3 | | | 3/16 | 1/4 | 3/16 | 1/4 |
| 4 | | | 1/4 | 1/4 | 1/4 | 1/4 |
| 5 | | | 5/16 | 1/4 | 5/16 | 5/16 |
| 6 | | | 3/8 | 5/16 | 3/8 | 5/16 |
| 7 | | | 7/16 | 3/8 | 3/8 | 3/8 |
| 8 | | | 1/2 | 7/16 | 7/16 | 7/16 |
| 9 | | | 9/16 | 1/2 | 1/2 | 1/2 |
| 10 | | | 5/8 | 1/2 | 9/16 | 9/16 |
| 11 | | | 11/16 | 9/16 | 5/8 | 5/8 |

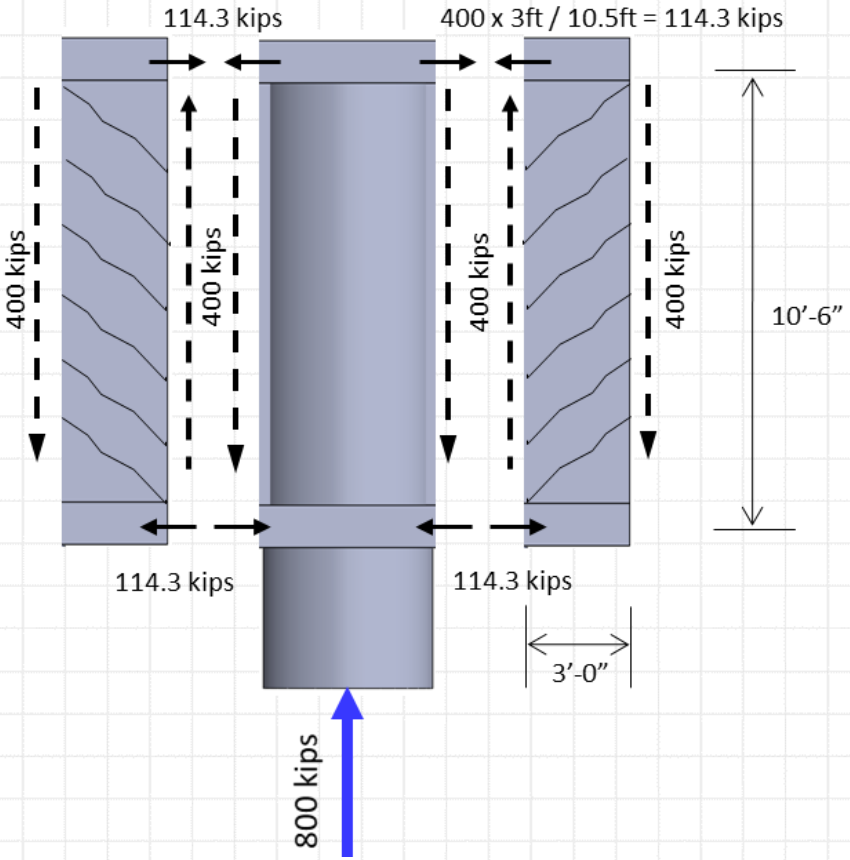
a. Refer to AWS D1.1 Table 3.1 – Prequalified Base Metal – Filler Material Combinations for Matching Strength and AWS D1.4 Table 5.1 Matching Filler Metal Requirements. Use E80 Electrodes for ASTM A706 rebar; use E90 electrodes for ASTM A615 rebar.
b. A minimum of 3/16 in. weld size is suggested.
c. Theoretical thickness for shear stress on base metal = 0.9(0.6)(36) ksi. A more practical thickness might be taken as 1/2 $d_b$ as used with headed studs. A minimum of 1/4 in. plate thickness is suggested.
d. E70 electrodes are not permitted for grade 60 reinforcement.

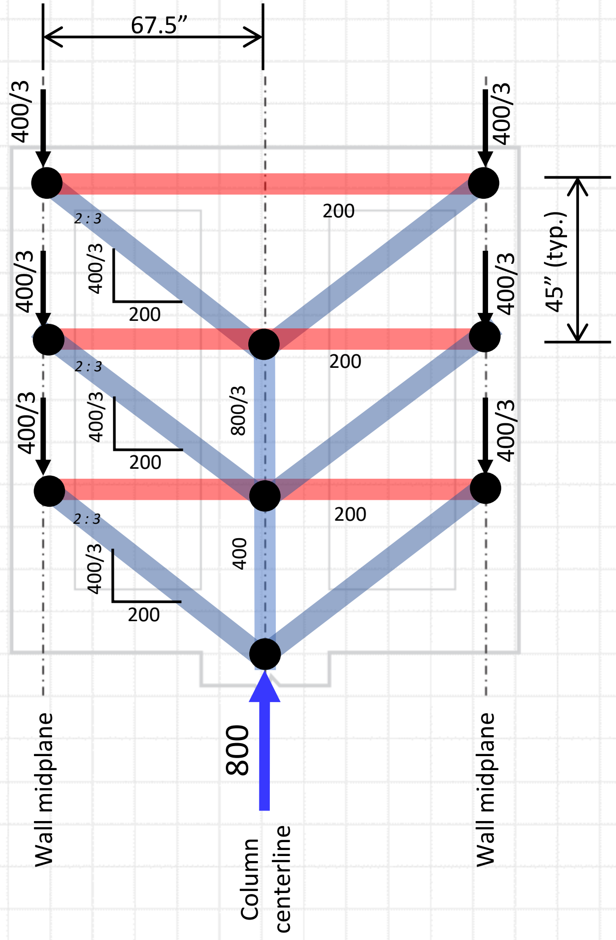
**Design Aid 6.15.6 Strength of Bolts and Threaded Fasteners[a]**

| Bolt diameter, in. | Nominal area $A$, in.$^2$ | ASTM A193 Grade B5 $F_u$ = 100 ksi | | | |
|---|---|---|---|---|---|
| | | Tension | | Shear | |
| | | Design | Service | Design | Service |
| 1/2 | 0.196 | 11.0 | 6.5 | 5.9 | 3.3 |
| 5/8 | 0.307 | 17.3 | 10.1 | 9.2 | 5.2 |
| 3/4 | 0.442 | 24.9 | 14.6 | 13.3 | 7.5 |
| 7/8 | 0.601 | 33.8 | 19.8 | 18.0 | 10.2 |
| 1 | 0.785 | 44.2 | 25.9 | 23.6 | 13.3 |
| 1 1/4 | 1.227 | 69.0 | 40.5 | 36.8 | 20.9 |
| 1 1/2 | 1.767 | 99.4 | 58.3 | 53.0 | 30.0 |
| 2 | 3.142 | 176.7 | 103.7 | 94.3 | 53.4 |

# Diaphragm Design – Strut and Tie Approach

Very oblong member with a construction joint at the column face (roughened)




Parallel Stress Field: “Accordion Effect”

$$A_{s,req} = \frac{200}{0.75 \times 60} = 4.44 in^2 \text{ per tie}$$

or

$$A_{s,req} = 3 \times 4.44 in^2 = 13.3 in^2 \text{ total}$$

Try #5@8" o.c. both faces $\rightarrow A_{s,pro} = \frac{2 \times 0.31}{8"} \times (3 \times 45") = 10.5 in^2 < A_{s,req}$

Try #4@4" o.c. both faces $\rightarrow A_{s,pro} = \frac{2 \times 0.2}{4"} \times (3 \times 45") = 13.5 in^2 > A_{s,req}$

The diaphragm shall have a minimum width to enable the transfer of shear through the roughened (1/4" amplitude) diaphragm - column interface,

$\phi V_n \geq V_u \quad \phi = 0.75$

$V_u = 400 kips$

$V_n = \min\left(0.2 f_c',\ (480 + 0.08 f_c'),\ 1600\right) A_c$

$\rightarrow V_n = \min\left(0.2 \times 5{,}000,\ (480 + 0.08 \times 5{,}000),\ 1600\right) A_c$

$V_n = 880 psi \times A_c$

$\therefore A_c \geq \frac{V_u}{\phi \times 880 psi} = \frac{400{,}000 lbs}{0.75 \times 880 psi} = 606 in^2$

but

$A_c = (10.5') \times w_{dia} = 126 in \times w_{dia}$

$\therefore w_{dia} \geq \frac{A_c}{126 in} = \frac{606 in^2}{126 in} = 4.8 in$, a very small width is required

Use 8" minimum as the width $w_{dia}$ to ease construction

**22.9.4.4** The value of $V_n$ across the assumed shear plane shall not exceed the limits in Table 22.9.4.4. Where concretes of different strengths are cast against each other, the lesser value of $f_c'$ shall be used in Table 22.9.4.4.

**Table 22.9.4.4—Maximum $V_n$ across the assumed shear plane**

| Condition | Maximum $V_n$ | | |
|---|---|---|---|
| Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in. | Least of (a), (b), and (c) | $0.2 f_c' A_c$ | (a) |
| | | $(480 + 0.08 f_c') A_c$ | (b) |
| | | $1600 A_c$ | (c) |
| Other cases | Lesser of (d) and (e) | $0.2 f_c' A_c$ | (d) |
| | | $800 A_c$ | (e) |

Check the reinforcement ratio of horizontal reinforcement provided,

$$\frac{2 \times 0.2 in^2}{4" \times 8"} = 0.0125$$

We need to provide a minimum reinforcement ratio of 0.0025 in the vertical direction,

Try #4 at 8" o.c. on both faces, $\frac{2 \times 0.2 in^2}{8" \times 8"} = 0.0063 \geq 0.0025$, excessive

Try #3 at 8" o.c. on both faces, $\frac{2 \times 0.1 in^2}{8" \times 8"} = 0.0031 \geq 0.0025$

# Diaphragm Design – Shear Friction Approach

**22.9—Shear friction**

**22.9.1** *General*

**22.9.2** *Required strength*

**22.9.2.1** Factored forces across the assumed shear plane shall be calculated in accordance with the factored load combinations defined in Chapter 5 and analysis procedures defined in Chapter 6.

**22.9.3** *Design strength*

**22.9.3.1** Design shear strength across the assumed shear plane shall satisfy:

$$\phi V_n \geq V_u \qquad (22.9.3.1)$$

for each applicable factored load combination.

**22.9.4** *Nominal shear strength*

**22.9.4.1** Value of $V_n$ across the assumed shear plane shall be calculated in accordance with 22.9.4.2 or 22.9.4.3. $V_n$ shall not exceed the value calculated in accordance with 22.9.4.4.

**22.9.4.2** If shear-friction reinforcement is perpendicular to the shear plane, nominal shear strength across the assumed shear plane shall be calculated by:

$$V_n = \mu A_{vf} f_y \qquad (22.9.4.2)$$

where $A_{vf}$ is the area of reinforcement crossing the assumed shear plane to resist shear, and $\mu$ is the coefficient of friction in accordance with Table 22.9.4.2.

**Table 22.9.4.2—Coefficients of friction**

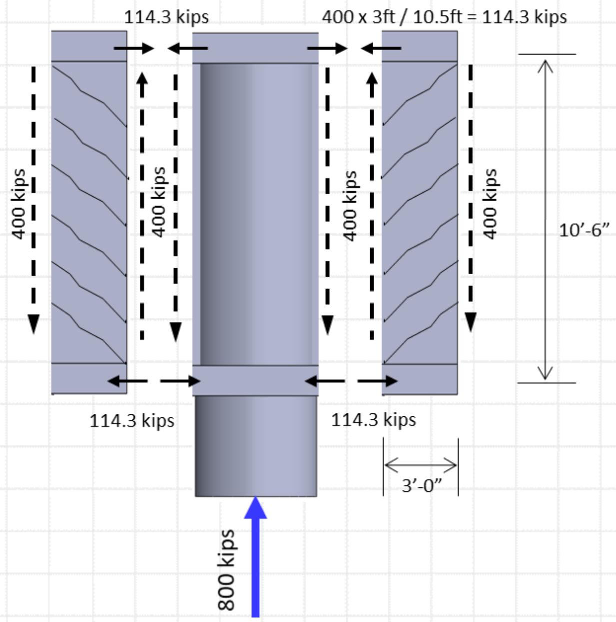
| Contact surface condition | Coefficient of friction $\mu$[1] | |
|---|---|---|
| Concrete placed monolithically | $1.4\lambda$ | (a) |
| Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in. | $1.0\lambda$ | (b) |
| Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened | $0.6\lambda$ | (c) |
| Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires. | $0.7\lambda$ | (d) |

[1]$\lambda$ = 1.0 for normalweight concrete. For lightweight concrete, $\lambda$ is calculated as given in 19.2.4, but shall not exceed 0.85.

**22.9.4.4** The value of $V_n$ across the assumed shear plane shall not exceed the limits in Table 22.9.4.4. Where concretes of different strengths are cast against each other, the lesser value of $f_c'$ shall be used in Table 22.9.4.4.

**Table 22.9.4.4—Maximum $V_n$ across the assumed shear plane**

| Condition | Maximum $V_n$ | | |
|---|---|---|---|
| Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in. | Least of (a), (b), and (c) | $0.2f_c'A_c$ | (a) |
| | | $(480 + 0.08f_c')A_c$ | (b) |
| | | $1600A_c$ | (c) |
| Other cases | Lesser of (d) and (e) | $0.2f_c'A_c$ | (d) |
| | | $800A_c$ | (e) |




$$V_u = 400kips$$

$$\phi V_n = \phi\ \mu\ A_{vt,req}\ f_y = 0.75\times(1.0\lambda)\times A_{vt,req}\times 60ksi$$

$$\phi V_n = 45ksi\ A_{vt}$$

$$\phi V_n \geq V_u \quad \rightarrow \quad 45ksi\ A_{vt,req} \geq 400kips$$

Solving for $A_{vt}$,

$$A_{vt,req} \geq \frac{400kips}{45ksi} = 8.89in^2 \text{ over the entire interface}$$

Try #4@8" on both faces, $A_{vt,pro} = 2\times 0.2\dfrac{(10\times 12+6)in}{8in} = 6.3\text{in}^2$, $A_{vt,pro} < A_{vt,req}$

Try #4@4" on both faces, $A_{vt,pro} = 2\times 0.2\dfrac{(10\times 12+6)in}{4in} = 12.6\text{in}^2$, $A_{vt,pro} > A_{vt,req}$

Minimum wall width calculations are the same as in previous slide. Hence, t = 8"

DRAWINGS

NORTH

B.O.L.D. Engineers

Bhat, Ojo, Lin & Dinh
Structural Engineers

9500 Gilmano Way
Suite 5201
San Diego, CA 92093

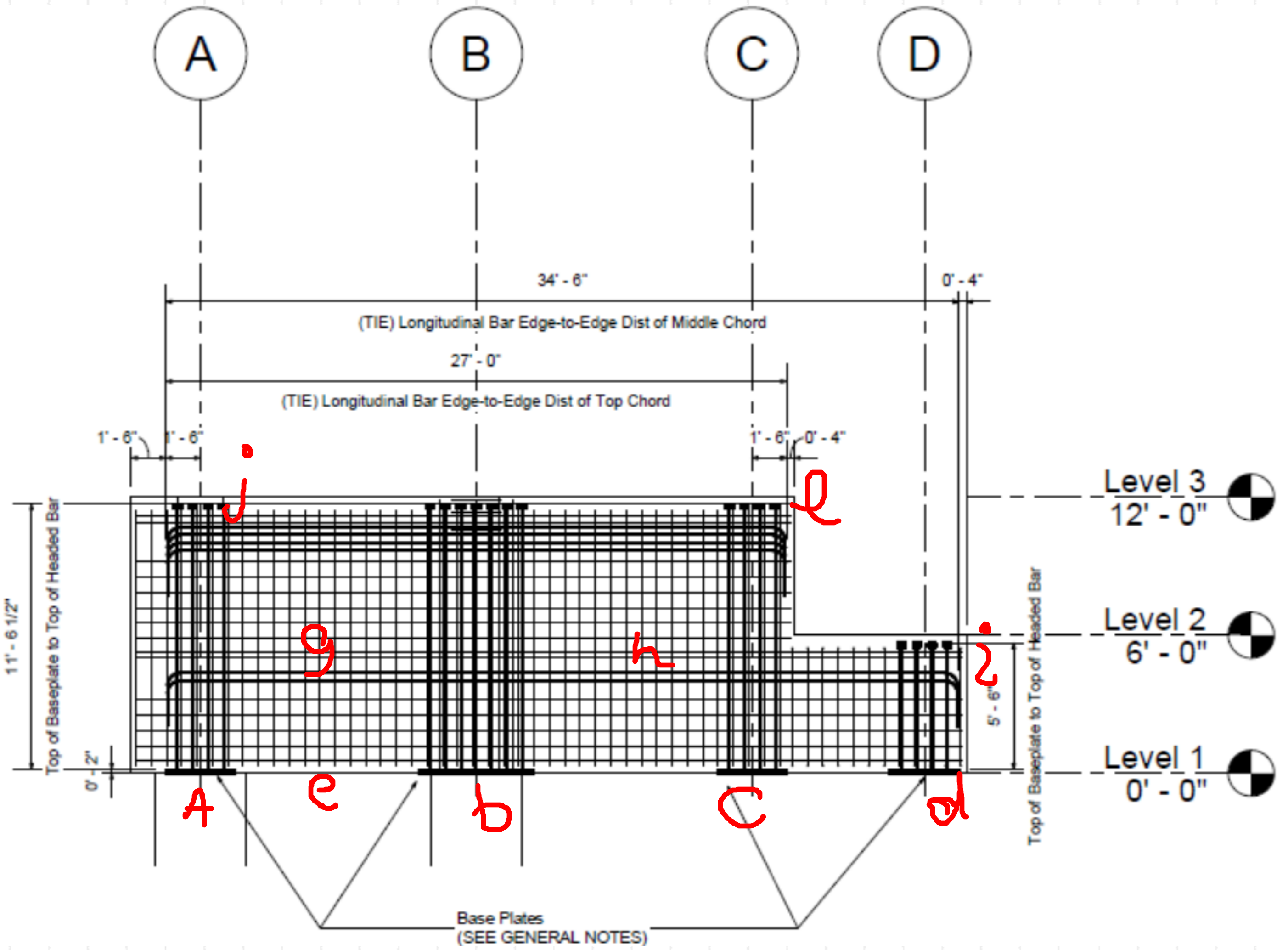
Owner

Project Name

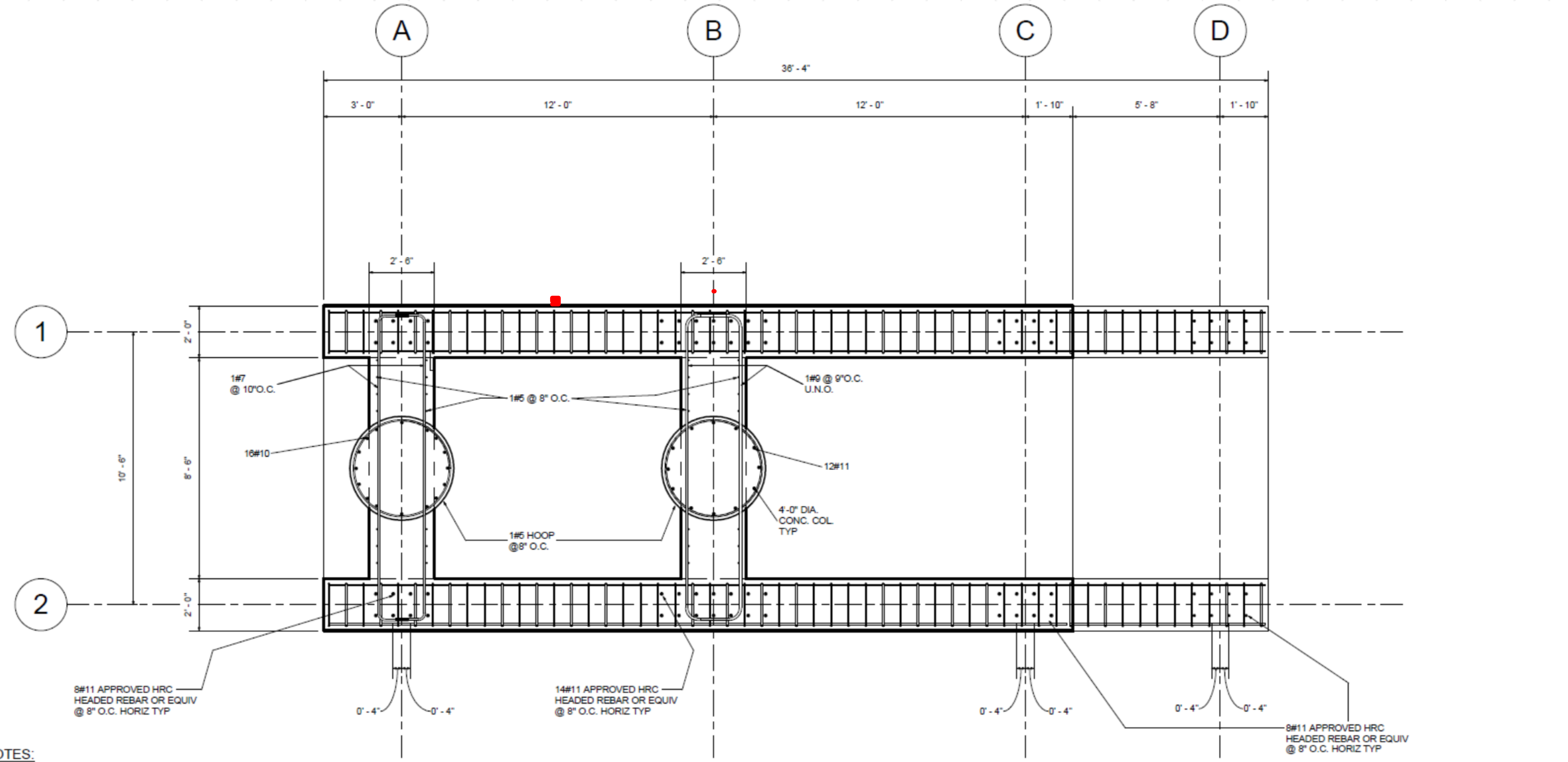
| No. | Description | Date |
|---|---|---|
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |

**Plan View**

| | | |
|---|---|---|
| Date | Issue Date | **ST.101** |
| Drawn by | Author | |
| Checked by | Checker | Scale 1/4" = 1'-0" |

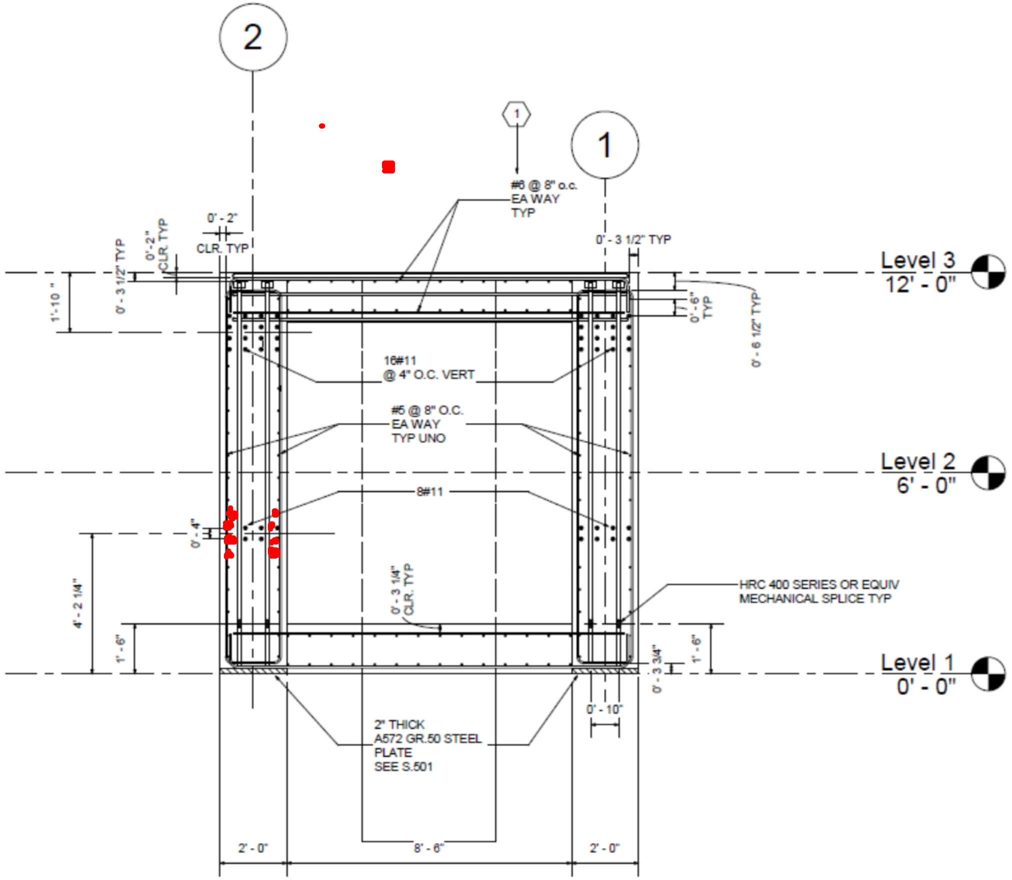


A
B
C
D
34' - 6"
0' - 4"
(TIE) Longitudinal Bar Edge-to-Edge Dist of Middle Chord
27' - 0"
(TIE) Longitudinal Bar Edge-to-Edge Dist of Top Chord
1' - 6"
1' - 6"
1' - 6"
0' - 4"
Level 3
12' - 0"
Level 2
6' - 0"
Level 1
0' - 0"
11' - 6 1/2"
Top of Baseplate to Top of Headed Bar
0' - 2"
5' - 6"
Top of Baseplate to Top of Headed Bar
Base Plates
(SEE GENERAL NOTES)
j
l
g
h
i
A
e
b
C
d




GENERAL NOTES:

1) Rebar Layout for Slabs not shown. See S.301
2) Horizontal Ties not shown. See S.201 & S.301
3) Standard 90 deg Hooks



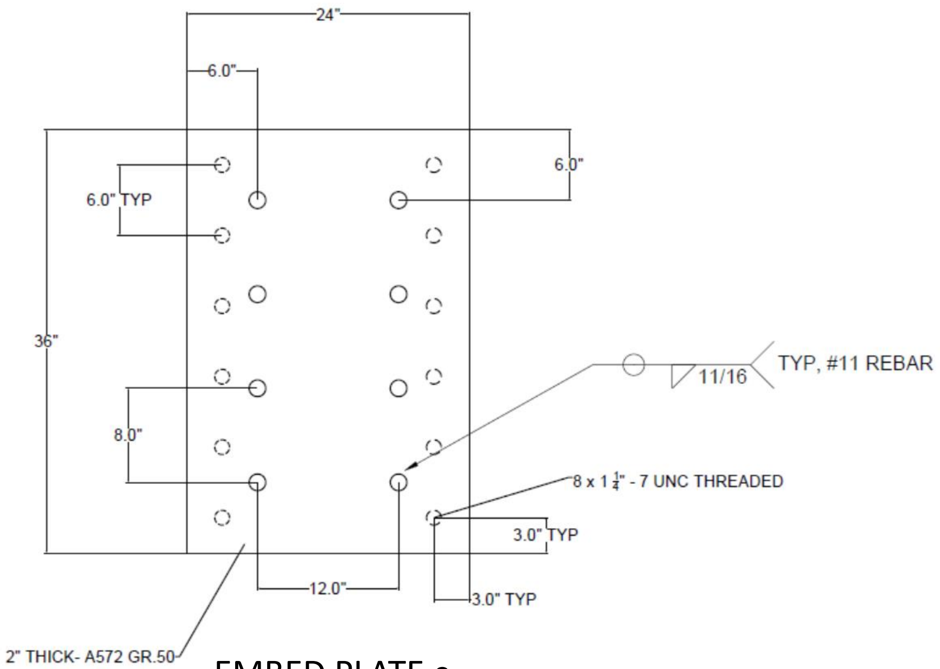

GENERAL NOTES:

1) Standard 90 deg Hooks

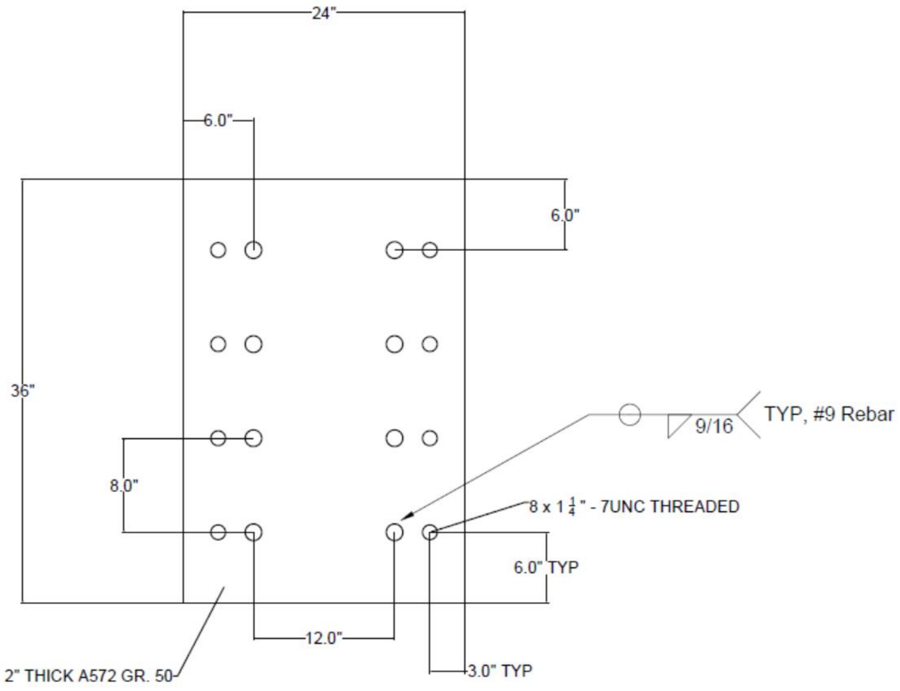
KEYNOTES:

1 Horizontal Reinf. Cutoff @ 3.5" Clear Cover
Transverse Reinf (Into Sheet) Cutoff @ 2" Clear Cover

1 Section 1
1/4" = 1'-0"




EMBED PLATE c
(Total 1)




EMBED PLATE d
(Total 1)