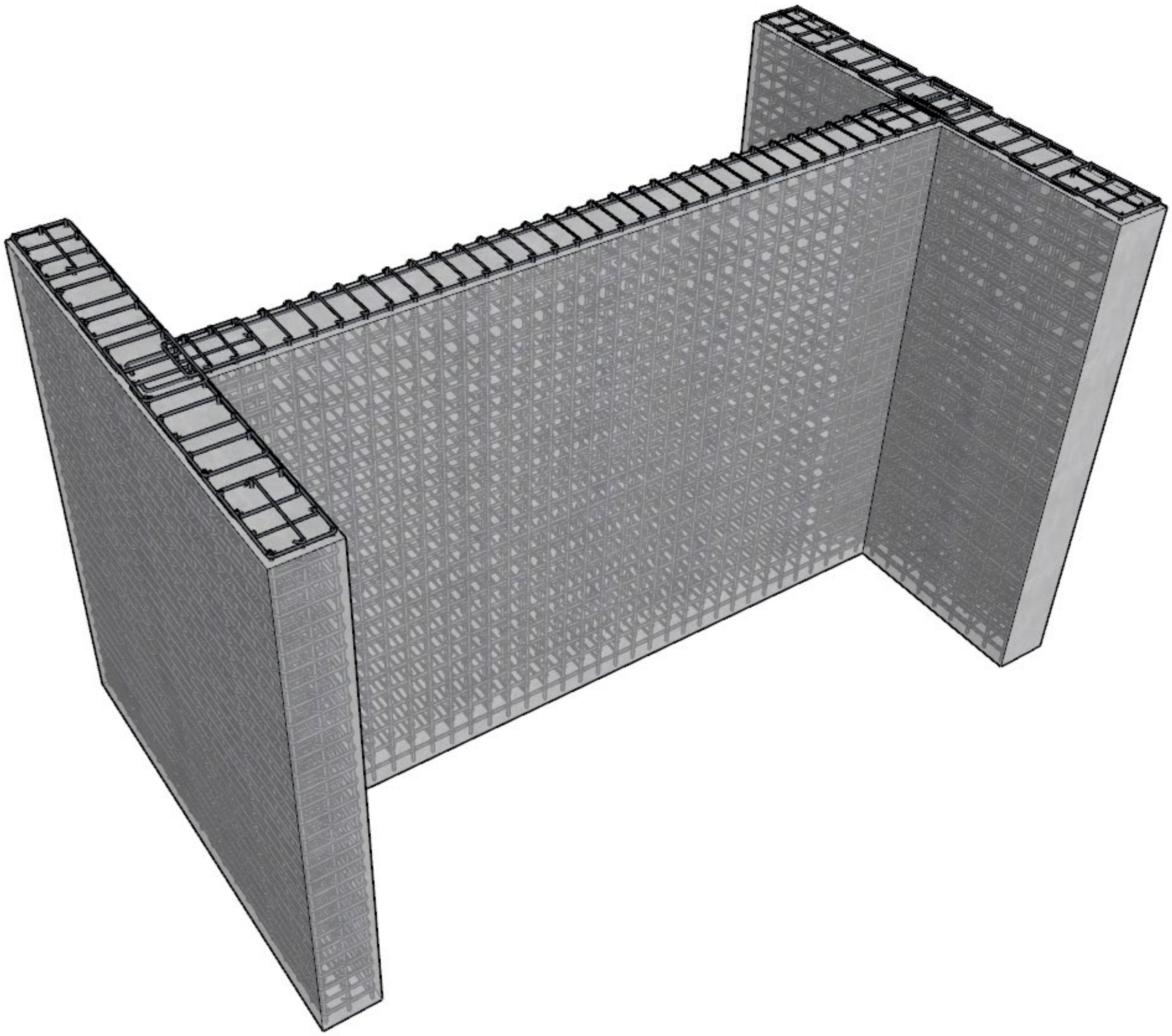
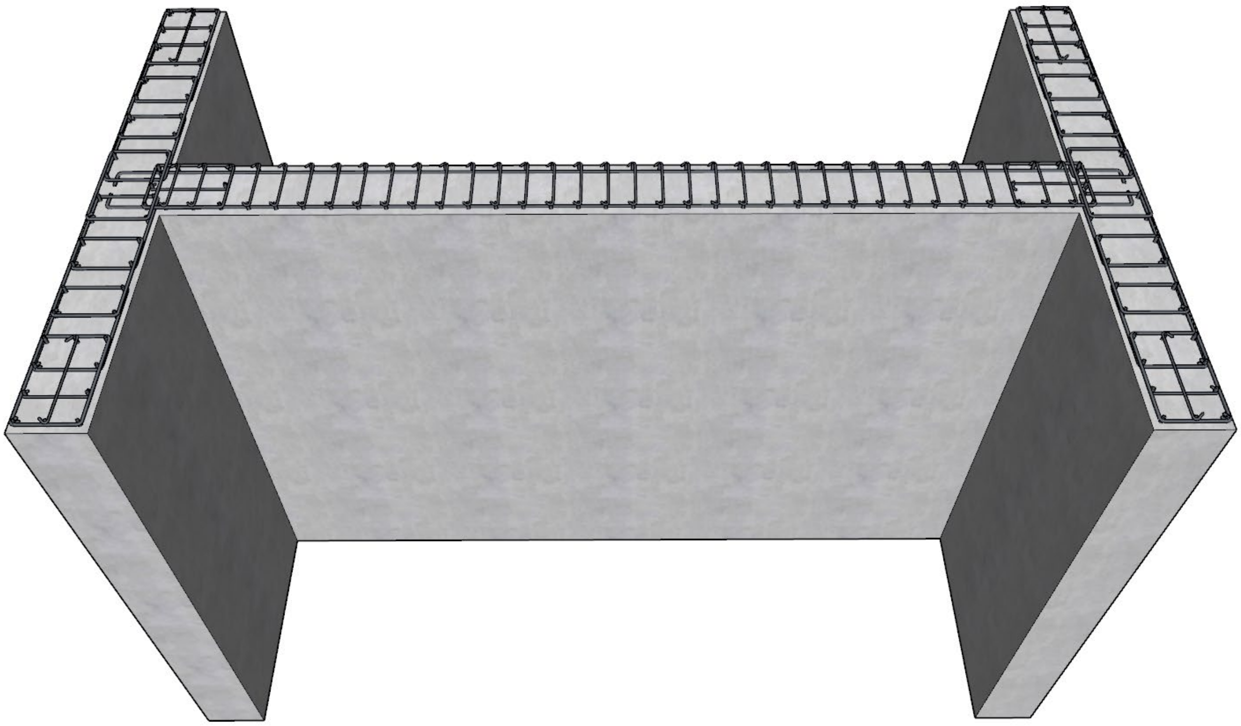
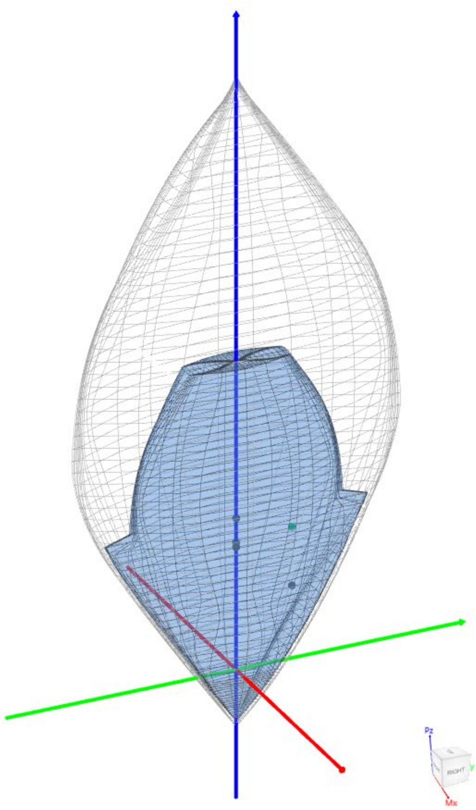
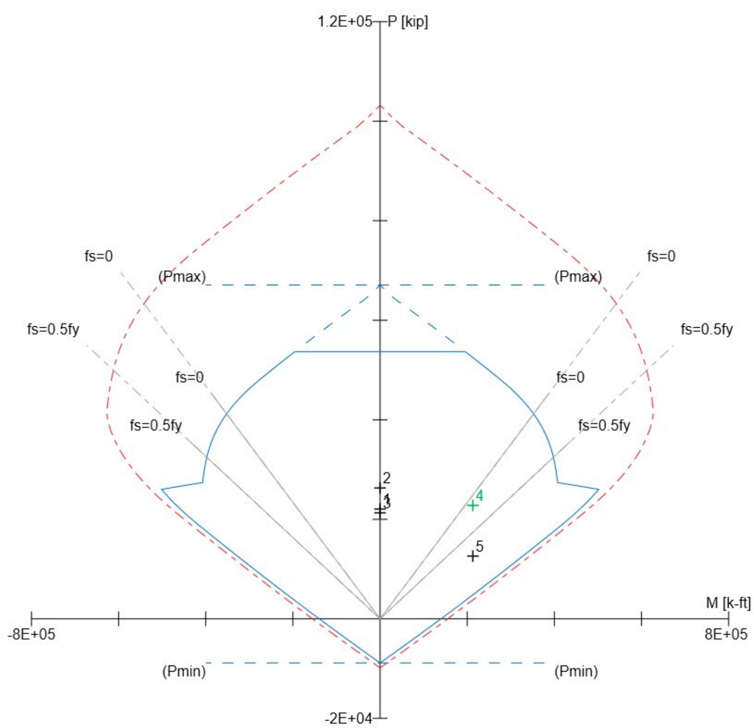
Design of Special Structural Shear Wall – Compressive Stress Approach (ACI 318-19)





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Design of Special Structural Shear Wall – Compressive Stress Approach (ACI 318-19)

When properly proportioned so that they possess adequate lateral stiffness to reduce interstory distortions due to earthquake-induced motions, structural walls (also called shear walls) reduce the likelihood of damage to the nonstructural elements of a building. Structural walls are normally much stiffer than regular frame elements and are therefore subjected to correspondingly greater lateral forces due to earthquake motions. Because of their relatively greater depth, the lateral deformation capacities of walls are limited, so that, for a given amount of lateral displacement, structural walls tend to exhibit greater apparent distress than frame members. However, over a broad period range, a structure with structural walls, which is substantially stiffer and hence has a shorter period than a structure with frames, will suffer less lateral displacement than the frame, when subjected to the same ground motion intensity. Structural walls with a height-to-horizontal length ratio, h_w/l_w, in excess of 2 behave essentially as vertical cantilever beams and should therefore be designed as flexural members, with their strength governed by flexure rather than by shear. Special reinforced concrete structural shear walls are required in structures assigned to Seismic Design Category (SDC) D or higher. This example illustrates the design procedure and steps of a special structural shear wall in the first story of a 30-story building providing lateral and gravity load resistance for the factored loads shown in the "Design Data" section. This example will conclude with a comparison of the calculated results with the exact values obtained from the spColumn engineering software program from StructurePoint.

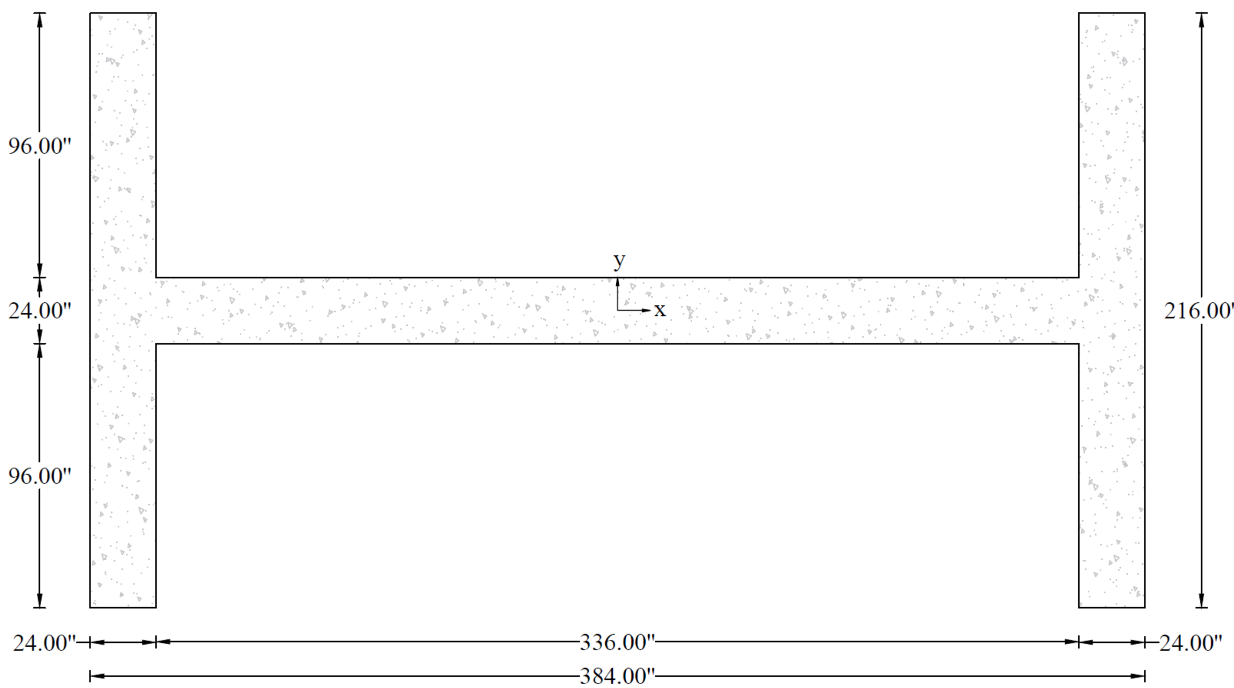


Figure 1 – Wall Cross-Section

Contents

Code

Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)

References

- Design Guide on the ACI 318 Building Code Requirements for Structural Concrete, First Edition, 2020 Concrete Reinforcing Steel Institute (CRSI), Example 14.11
- spColumn Engineering Software Program Manual v10.10, STRUCTUREPOINT, 2023
- "Interaction Diagram - Barbell Concrete Shear Wall Unsymmetrical Boundary Elements (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022
- "Building Elevator Reinforced Concrete Core Wall Design Strength (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022
- "Biaxial Bending Interaction Diagrams for C-Shaped Concrete Core Wall Design (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022
- "Interaction Diagram - Tied Reinforced Concrete Column with High-Strength Reinforcing Bars (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022

- Contact Support@StructurePoint.org to obtain supplementary materials (spColumn models: CRSI-Ex-14-11.colx and CRSI-Ex-14-11-M_{pr}.colx)

Design Data

f_c' = 6,000 psi

f_y = 80,000 psi

Clear cover = 0.75 in.

Story heights = 11 ft

Number of stories = 30

l_w = 32.0 ft (Required extent of special boundary elements)

Table 1 – Wall Loads			
Load Case	Axial Load, kip	Bending Moment, ft-kip	Shear Force, kip
Dead, D	15,766.1	0	0
Live, L	4,594.4	0	0
Roof Live, L_r	47.1	---	---
Seismic, Q_e	0	±163,801	±901.3

Table 2 – Wall Factored Loads						
ACI Eq. Reference	No.	Load Combination		Axial Load, kip	Bending Moment, ft-kip	Shear Force, kip
5.3.1a	1	1.4D		22,072.5	0	0
5.3.1b	2	$1.2D + 1.6L + 0.5L_r$		26,293.9	0	0
5.3.1c	3	$1.2D + 0.5L + 1.6L_r$		21,291.9	0	0
5.3.1e	4	$1.3D + 0.5L + 1.3Q_e$	SSR	22,793.1	-212,941.3	-1,171.7
	5		SSL	22,793.1	212,941.3	1,171.7
5.3.1g	6	$0.8D + 1.3Q_e$	SSR	12,612.9	-212,941.3	-1,171.7
	7		SSL	12,612.9	212,941.3	1,171.7

SSR = sidesway to the right, SSL = sidesway to the left

1. Notations

This section (based on ACI 318-19 provisions) defines notation and terminology used in this design example:

A_{ch} = cross-sectional area of a member measured to the outside edges of transverse reinforcement, in.^{2}

A_{cv} = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered in the case of walls, and gross area of concrete section in the case of diaphragms. Gross area is total area of the defined section minus area of any openings, in.^{2}

A_{cw} = area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear, in.^{2}

A_g = gross area of concrete section, in.^{2} For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s)

A_{sh} = total cross-sectional area of transverse reinforcement, including crossties, within spacing s and perpendicular to dimension b_c, in.^{2}

b = width of compression face of member, in.

b_c = cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}, in.

c = distance from extreme compression fiber to neutral axis, in.

d_b = nominal diameter of bar, wire, or prestressing strand, in.

f_c' = specified compressive strength of concrete, psi

f_{cu} = The maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects, psi

f_y = specified yield strength for nonprestressed reinforcement, psi

f_{yt} = specified yield strength of transverse reinforcement, psi

h_u = laterally unsupported height at extreme compression fiber of wall or wall pier, in., equivalent to l_u for compression members

h_w = height of entire wall from base to top, or clear height of wall segment or wall pier considered, in.

h_{wcs} = height of entire structural wall above the critical section for flexural and axial loads, in.

h_x = maximum center-to-center spacing of longitudinal bars laterally supported by corners of crossties or hoop legs around the perimeter of a column or wall boundary element, in.

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.^{4}

l_{be} = length of boundary element from compression face of member, in.

l_w = length of entire wall, or length of wall segment or wall pier considered in direction of shear force, in.

M_{pr} = probable flexural strength of members, with or without axial load, determined using the properties of the member at joint faces assuming a tensile stress in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor ϕ of 1.0, in.-lb

M_u = factored moment at section, in.-lb

n_s = number of stories above the critical section

P_u = factored axial force; to be taken as positive for compression and negative for tension, lb

s = center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, tendons, or anchors, in.

s_o = center-to-center spacing of transverse reinforcement within the length l_o, in.

V_e = design shear force for load combinations including earthquake effects, lb

V_n = nominal shear strength, lb

V_u = factored shear force at section, lb

α_c = coefficient defining the relative contribution of concrete strength to nominal wall shear strength

λ = modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normalweight concrete of the same compressive strength

ρ_l = ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement

ρ_t = ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement

ϕ = strength reduction factor

Ω_v = overstrength factor equal to the ratio of M_{pr}/M_u at the wall critical section

ω_v = factor to account for dynamic shear amplification

2. Minimum Web Reinforcement Ratios

The distributed web reinforcement ratios, ρ_l and ρ_t, for structural walls shall be at least 0.0025, except that if V_u does not exceed $\lambda\sqrt{f'_c}A_{cv}$ shall be permitted to be reduced to the values in ***ACI 318-19 (11.6)***. Reinforcement spacing each way in structural walls shall not exceed 18 in. Reinforcement contributing to V_n shall be continuous and shall be distributed across the shear plane. ***ACI 318-19 (18.10.2.1)***

$$V_u = 1{,}171.7 \text{ kip} > \lambda\sqrt{f'_c}A_{cv}$$ ***ACI 318-19 (18.10.2.1)***

$$V_u = 1{,}171.7 \text{ kip} > 1.0\times\sqrt{6{,}000}\times\left(24\times\left(32\times12\right)\right)$$

$$V_u = 1{,}171.7 \text{ kip} > 713.87 \text{ kip}$$

Therefore, minimum $\rho_l = \rho_t = 0.0025$.

3. Number of Curtains of Web Reinforcement

At least two curtains of reinforcement shall be used in a wall if $V_u > 2\lambda\sqrt{f'_c}A_{cv}$ or $h_w / l_w \geq 2.0$, in which h_w and l_w refer to height and length of entire wall, respectively. ***ACI 318-19 (18.10.2.2)***

$$\frac{h_w}{l_w} = \frac{11 \text{ ft}\times30}{32 \text{ ft}} = \frac{330}{32} = 10.3 > 2.0$$ ***ACI 318-19 (18.10.2.2)***

Therefore, 2 curtains of longitudinal and transverse reinforcement are required in the web.

4. Required Area of Longitudinal Reinforcement for Flexure and Axial Forces

ACI 318-19 (18.10.5)

The design strength interaction diagram for this wall is given in the figure below. Detailed discussion about generating interaction diagrams for shear walls can be found in "Interaction Diagram – Barbell Concrete Shear Wall Unsymmetrical Boundary Elements (ACI 318-19)" Design Example. The longitudinal reinforcement selected by the reference consists of 44-#8 bars in each flange and 2-#8 bars spaced at 10.0 in. on center in the web. 4-#8 bars are required at the mid-depth of the flange (two bars each end) in order to satisfy the maximum spacing requirement in ***ACI 318-19 (18.10.6.4(f))*** for bars around the perimeter of the special boundary element (the maximum spacing is the lesser of 14 in. and two-thirds the boundary element thickness, which is equal to 16.0 in.; see Special Boundary Elements – Compressive Stress Approach and Special boundary element required transverse reinforcement below). All load combinations fall within the boundary of the design strength interaction diagram.

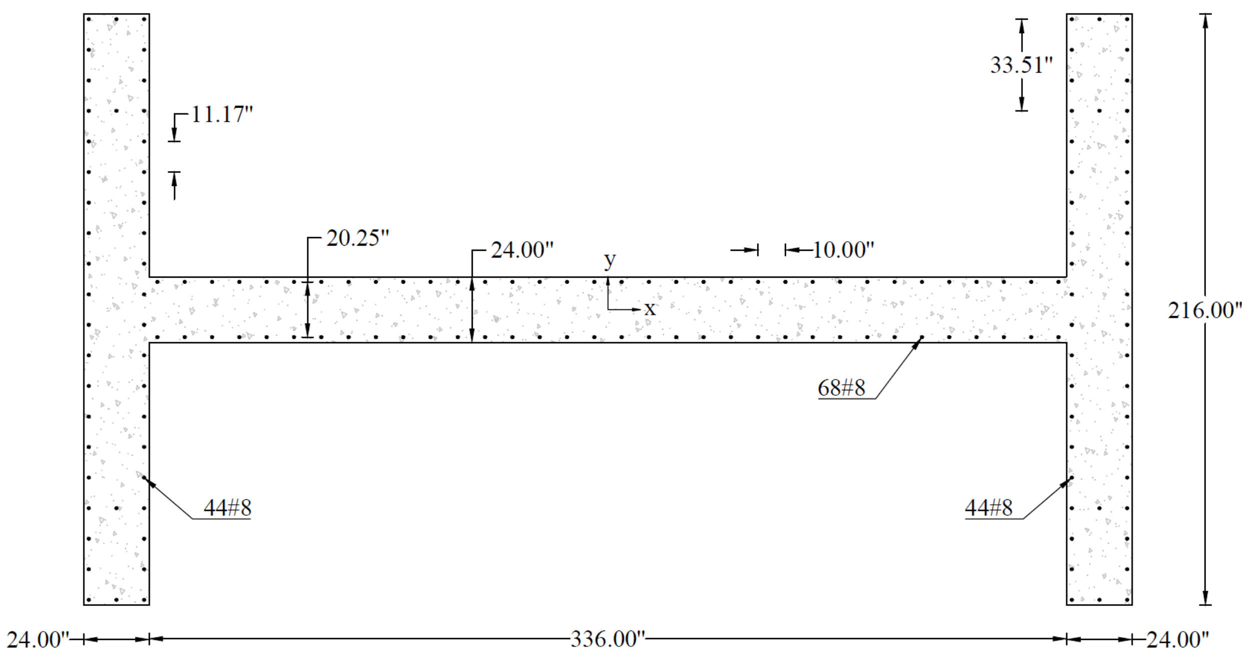


Figure 2 – Reinforced Concrete Wall Cross-Section

Table 3 – Factored Loads and Moments with Corresponding Capacities					
No.	Load Combination	P_u, kip	M_u, ft-kip	ϕP_n, kip	ϕM_n, ft-kip
1	1.4D	22,072.5	0.0	22,072.5	458,616.69
2	1.2D + 1.6L + 0.5L$_r$	26,293.9	0.0	26,293.9	504,114.69
3	1.2D + 0.5L + 1.6L$_r$	21,291.9	0.0	21,291.9	448,922.66
4	1.3D + 0.5L + 1.3Q$_e$	22,793.1	212,941.3	22,793.1	467,210.25
5	0.8D + 1.3Q$_e$	12,612.9	212,941.3	12,612.9	327,498.25

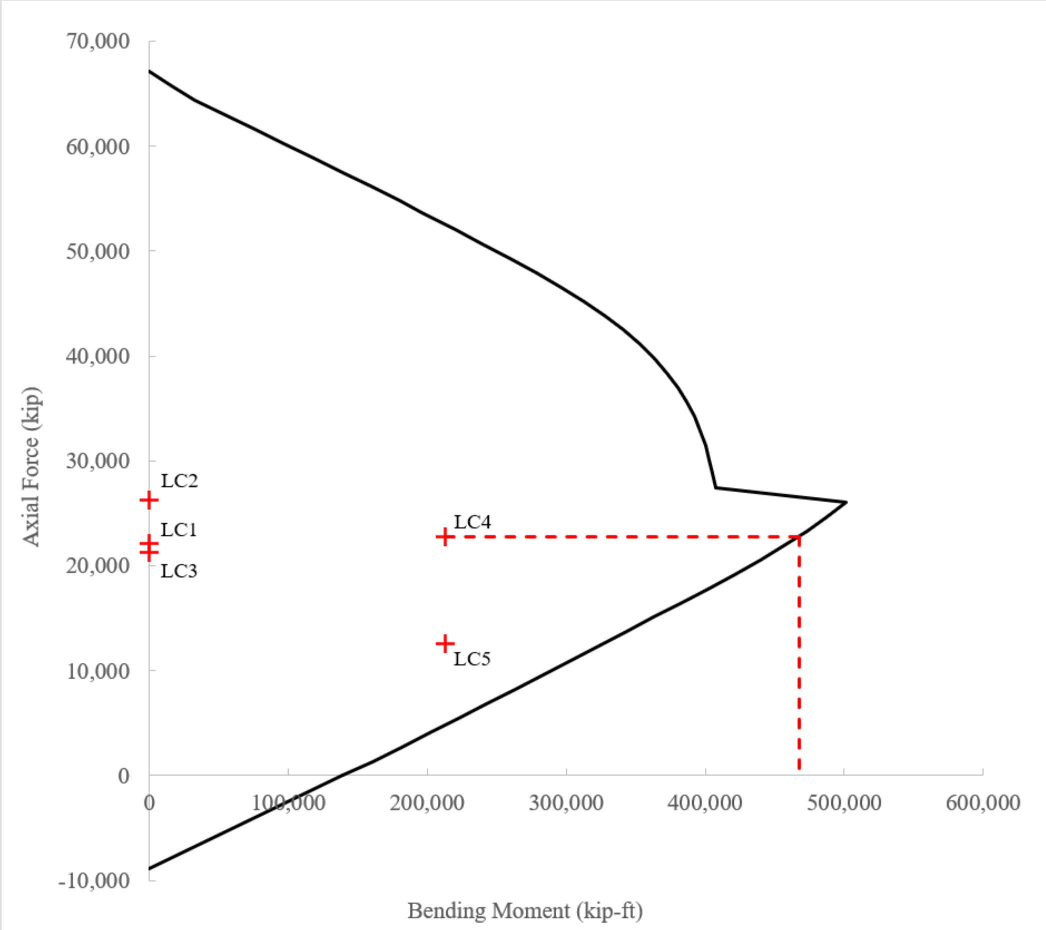


Figure 3 – Design Strength Interaction Diagram

Walls or wall piers with $h_w / l_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall and are designed to have a single critical section for flexure and axial loads shall have longitudinal reinforcement at the ends of a vertical wall segment that satisfies (a) through (c).

a) Longitudinal reinforcement ratio within **0.15*l*w** from the end of a vertical wall segment, and over a width equal to the wall thickness, shall be at least $\dfrac{6\sqrt{f_c'}}{f_y}$

b) The longitudinal reinforcement required by (a) shall extend vertically above and below the critical section at least the greater of $\boldsymbol{l_w}$ and $\boldsymbol{M_u / 3V_u}$.

c) No more than 50 percent of the reinforcement required by (a) shall be terminated at any one section.

ACI 318-19 (18.10.2.4)

Check the minimum longitudinal reinforcement requirements of ***ACI 318-19 (18.10.2.4(a))*** (see Figure 5).

$$0.15l_w = 0.15 \times 18 = 2.7 \text{ ft}$$ ***ACI 318-19 (Fig. 18.10.2.4)***

There are 7-#8 bars within the 2.7-ft width at each end of both flanges; therefore,

$$\frac{7 \times 0.79}{24 \times (2.7 \times 12)} = 0.0071 > \frac{6\sqrt{f'_c}}{f_y} = \frac{6\sqrt{6,000}}{80,000} = 0.0058$$ ***ACI 318-19 (18.10.2.4(a))***

There are 6-#8 bars within the 2.7-ft width in the web; therefore,

$$\frac{6 \times 0.79}{24 \times (2.7 \times 12)} = 0.0061 > \frac{6\sqrt{f'_c}}{f_y} = \frac{6\sqrt{6,000}}{80,000} = 0.0058$$ ***ACI 318-19 (18.10.2.4(a))***

Therefore, the requirements of ***ACI 18.10.2.4(a)*** are satisfied.

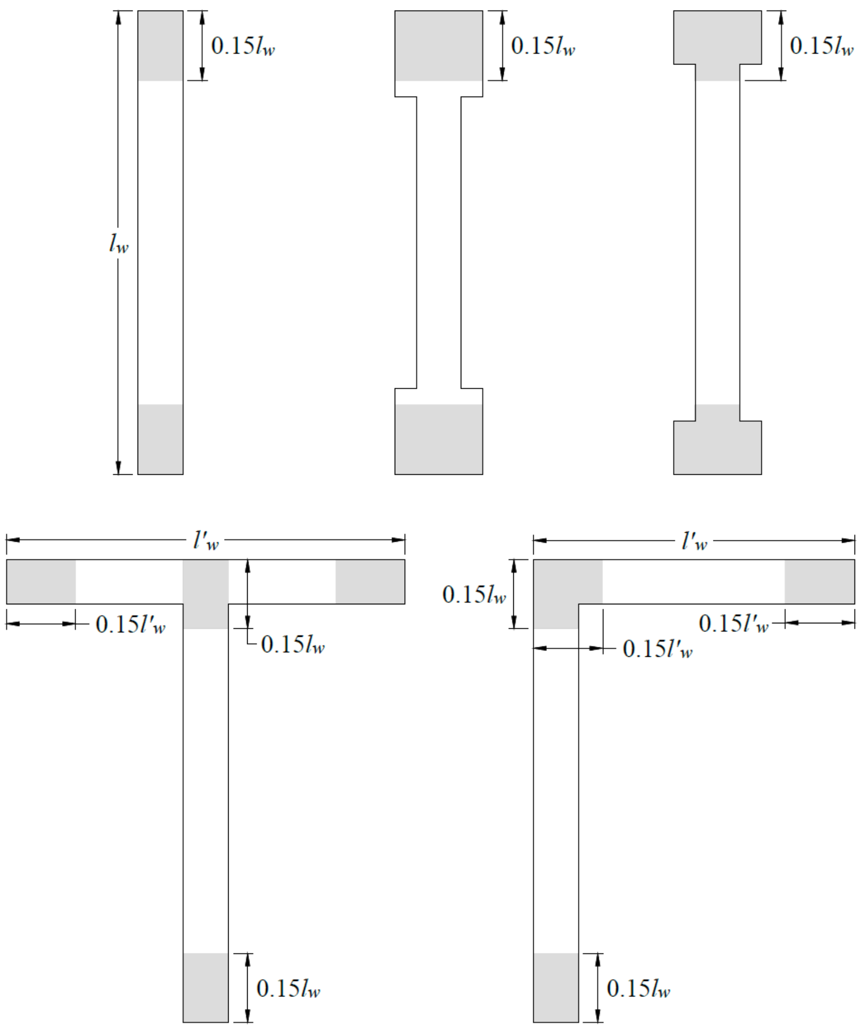


Figure 4 – Locations of longitudinal reinforcement required by ACI 318-19 (18.10.2.4(a)) in different configurations of wall sections

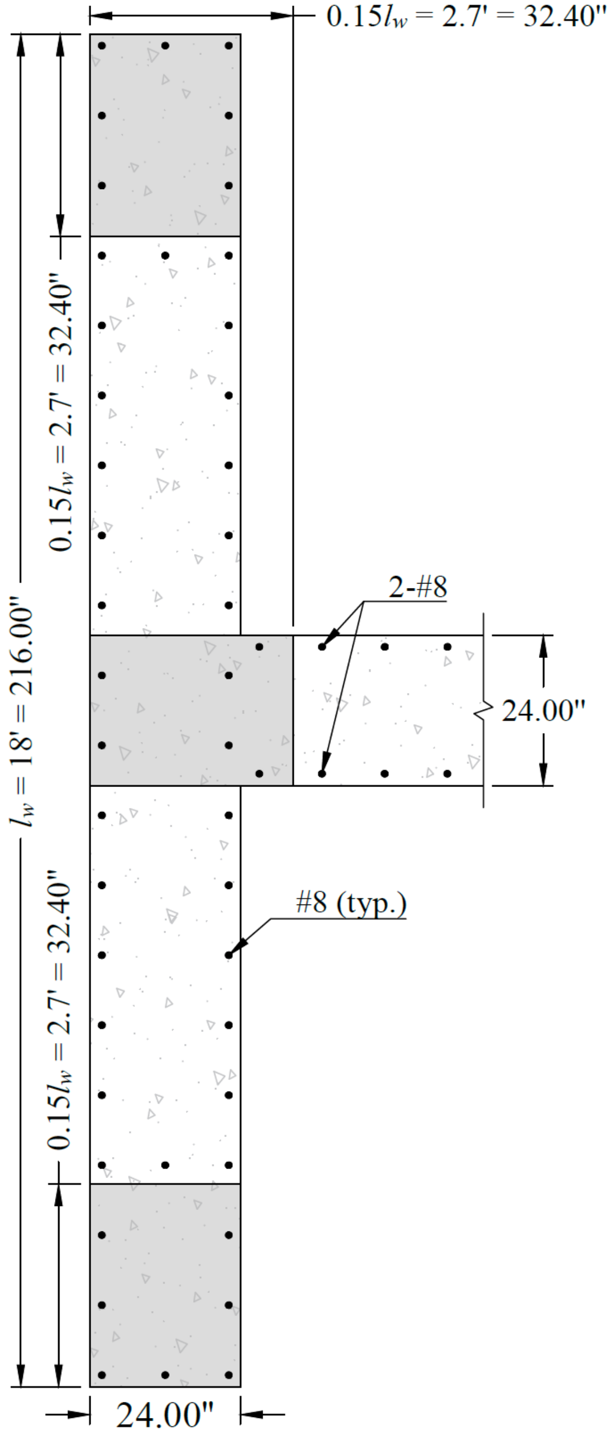


Figure 5 – Minimum longitudinal reinforcement in accordance with ***ACI 318-19 (18.10.2.4)***

5. Design Shear Force

The design shear force, V_e, is determined in accordance with ***ACI 318-19 (18.10.3.1)***:

$$V_e = \Omega_v \omega_v V_u \le 3V_u$$ ***ACI 318-19 (18.10.3.1)***

Because $h_{wcs} / l_w = 10.3 > 1.5$,

$$\Omega_v = \text{Greater of} \begin{Bmatrix} \dfrac{M_{pr}}{M_u} \\ 1.5 \end{Bmatrix}$$ ***ACI 318-19 (Table 18.10.3.1.2)***

The nominal strength interaction diagram for this wall with $f_y = 1.25 \times 80 = 100$ ksi and $\phi = 1.0$ is given in Figure 6.

The largest M_{pr} = 520,216.38 ft-kip

Corresponding axial force = 22,793.1 kip.

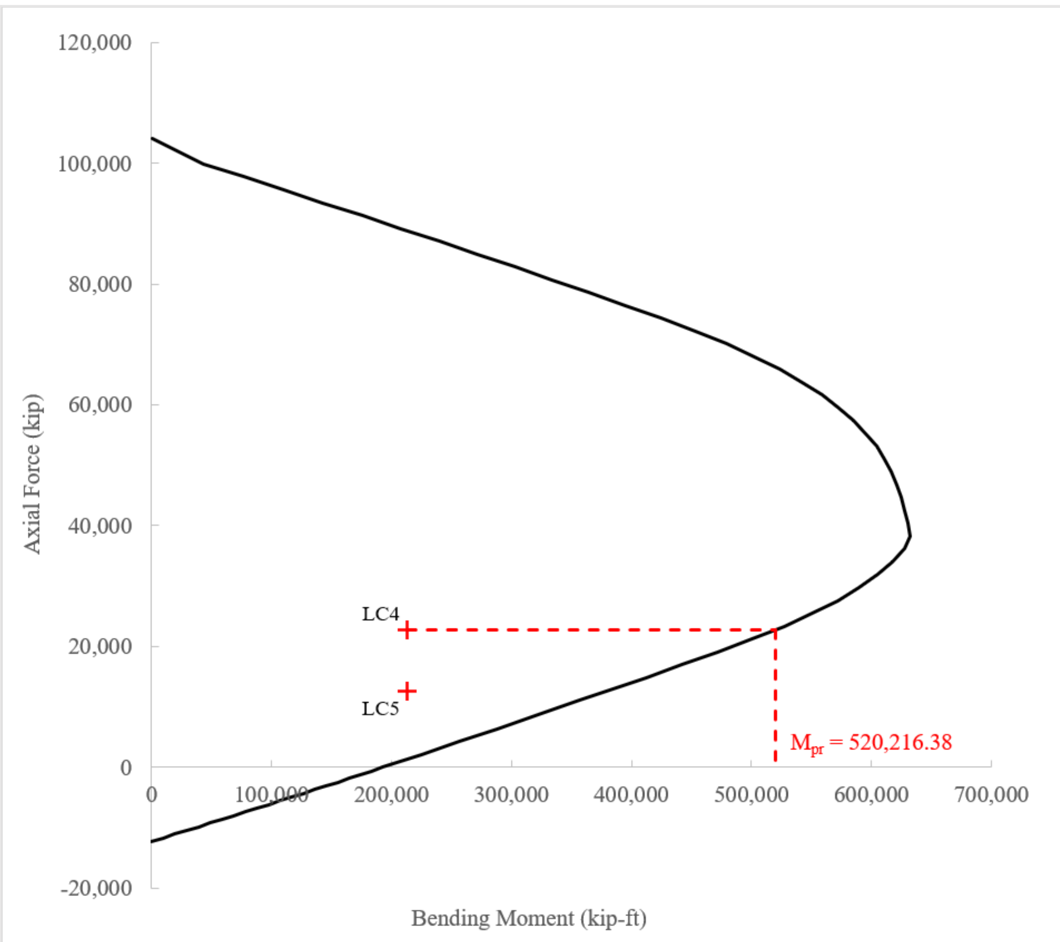


Figure 6 – Nominal Strength Interaction Diagram for the wall with $f_y = 1.25 \times 80 = 100$ ksi and $\phi = 1.0$

$$\Omega_v = \text{Greater of} \begin{Bmatrix} \dfrac{M_{pr}}{M_u} = \dfrac{520,216.38}{212,941.3} = 2.44 \\ 1.5 \end{Bmatrix} = 2.44$$

For this 30-story building, the dynamic shear amplification factor is equal to the following:

$$\omega_v = 1.3 + \frac{n_s}{30} \le 1.8$$ ***ACI 318-19 (18.10.3.1.3)***

$$\omega_v = 1.3 + \frac{30}{30} = 2.3 > 1.8,\ \text{use } 1.8$$

Therefore,

$$V_e = 2.44 \times 1.8 \times 1,171.7 = 5,152.4 \text{ kip} < 3 \times V_u = 3 \times 1,171.7 = 3,515.1 \text{ kip}$$ ***ACI 318-19 (18.10.3.1)***

6. Shear Strength Requirements

The design shear strength, ϕV_n, must be greater than or equal to V_e.

Determine the required transverse reinforcement ratio using ***ACI 318-19 (18.10.4.1)***.

$$\phi V_n = \phi\left(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_{yt}\right) A_{cv}$$ ***ACI 318-19 (18.10.4.1)***

Where:

$\alpha_c = 2$ for $h_w / l_w = 330 / 32 = 10.31 > 2.0$ ***ACI 318-19 (18.10.4.1)***

$$\phi V_n = 0.75 \times \left(2 \times 1.0 \times \sqrt{6,000} + \rho_t \times 80\right) \times \left(24 \times 384\right) = V_e = 3,515.1 \text{ kip}$$

Therefore, required $\rho_t = 0.0044$.

Assuming two layers of #7 horizontal bars in the web, the required spacing is equal to the following:

$$s = \frac{2 \times 0.60}{24 \times 0.0044} = 11.31 \text{ in.}$$

Check the upper limit on shear strength based on 2-#7 bars with a center-to-center spacing of 11.0 in.:

$$\rho_t = \frac{2 \times 0.60}{24 \times 11} = 0.0045$$

$$\phi V_n = 0.75 \times \left(2 \times 1.0 \times \sqrt{6,000} + 0.0045 \times 80\right) \times \left(24 \times 384\right) = 3,584.26 \text{ kip}$$

For all vertical wall segments sharing a common lateral force, $\boldsymbol{V_n}$ shall not be taken greater than $8\sqrt{f_c'}A_{cv}$. For any one of the individual vertical wall segments, $\boldsymbol{V_n}$ shall not be taken greater than $10\sqrt{f_c'}A_{cw}$, where $\boldsymbol{A_{cw}}$ is the area of concrete section of the individual vertical wall segment considered. ***ACI 318-19 (18.10.4.4)***

$$\phi V_n = 3,584.26 \text{ kip} < \phi 8\sqrt{f_c'}A_{cv}$$ ***ACI 318-19 (18.10.4.4)***

$$\phi V_n = 3,584.26 \text{ kip} < 0.75 \times 8 \times \sqrt{6,000} \times \left(24 \times 384\right) = 4,283.21 \text{ kip}$$

Use 2-#7 horizontal bars spaced at 11.0 in. on center.

7. Special Boundary Elements – Compressive Stress Approach

Special boundary elements are required where the following equation is satisfied:

$$f_{cu} = \frac{P_u}{A_g} + \frac{M_u \times l_w}{2 \times I_g} > 0.2 f_c'$$

ACI 318-19 (18.10.6.3)

From Table 2, the load combination that includes earthquake effects which results in the largest f_{cu} is ACI Eq. (5.3.1e):

$$f_{cu} = \frac{22,793.1 \times 1,000}{18,432} + \frac{212,941.3 \times 12,000 \times 384}{2 \times 412,286,976} > 0.2 f_c'$$

$$f_{cu} = 1,236.61 + 1,189.99 = 2,426.59 \text{ psi} > 0.2 f_c' = 0.2 \times 6,000 = 1,200 \text{ psi}$$

Therefore, special boundary elements are required at the ends of the wall.

8. Vertical Extent of the Special Boundary Element Transverse Reinforcement

Special boundary elements are permitted to be discontinued where the calculated $f_{cu} < 0.15 f_c'$.

The reference determined that special boundary elements can be terminated above the twenty-third-floor level (the wall thickness is 12.0 in. and the compressive strength of the concrete is equal to 5,000 psi at this elevation, which makes the stress limit equal to $0.15 f_c' = 750$ psi. ***ACI 318-19 (18.10.6.3)***

9. Horizontal Length of the Special Boundary Elements

The special boundary elements must extend horizontally from the extreme compression fiber a distance equal to the following:

$$l_{be} = \text{greater of}\begin{Bmatrix} c-0.1\times l_w \\ c/2 \end{Bmatrix}$$ ***ACI 318-19 (18.10.6.4(a))***

$$l_{be} = \text{greater of}\begin{Bmatrix} 61.95-0.1\times 384 \\ 61.95/2 \end{Bmatrix} = \begin{Bmatrix} 23.55\text{ in.} \\ 30.98\text{ in.} \end{Bmatrix} = 30.98\text{ in.}$$

The largest neutral axis depth, c, used in the determination of l_{be} is calculated from a strain compatibility analysis using the axial force and nominal moment strength from ACI Eq. (5.3.1e) in Table 2.

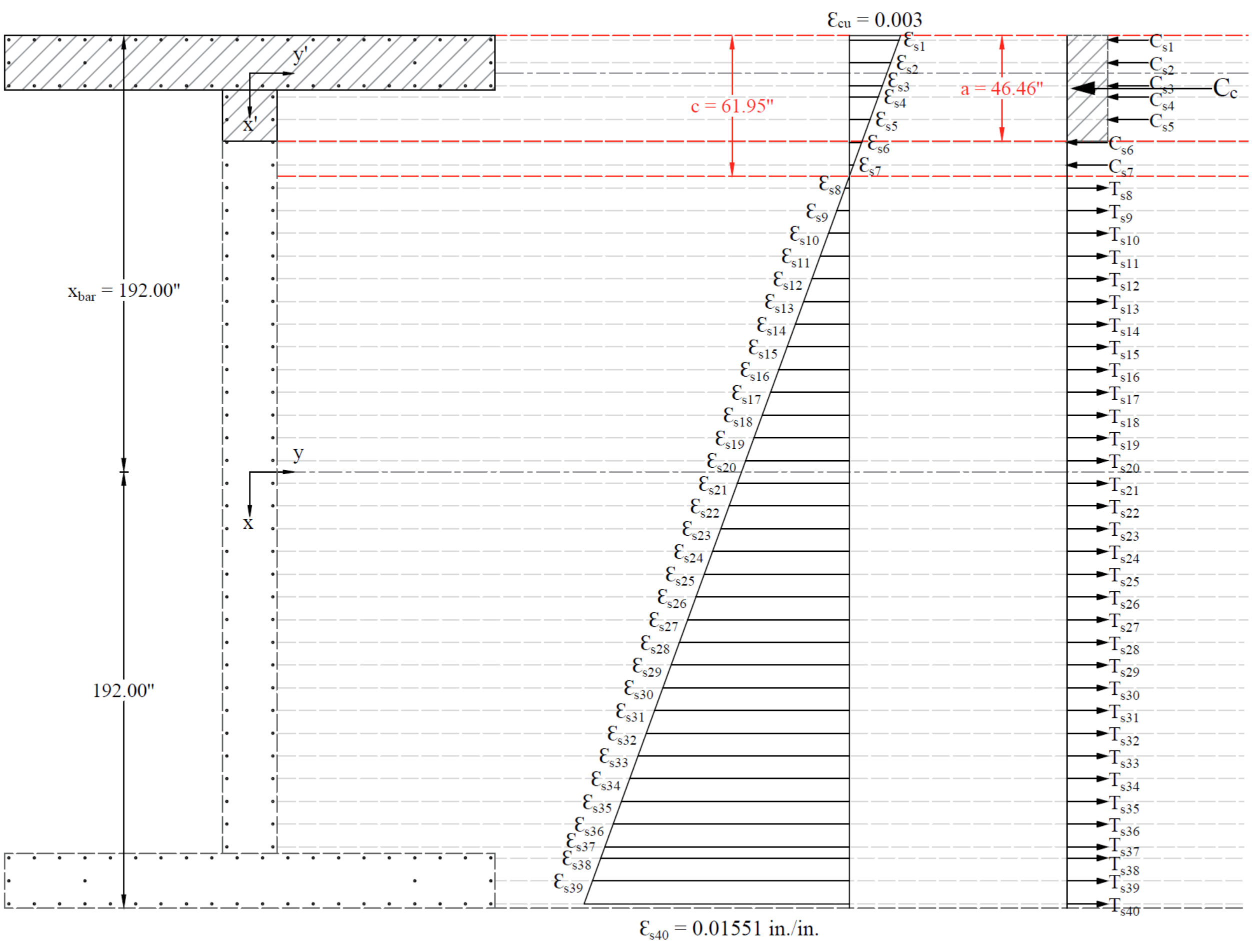


Figure 7 – Strain and Force Diagrams ($\phi P_n = P_u$ = 22,793.1 kip and ϕM_n = 467,210.25 ft-kip)

It is evident that the calculated horizontal length of the special boundary element includes the 24 in. thick flange and extends 30.98 – 24.0 = 6.98 in. into the web. According to ***ACI 318-19 (18.10.6.4(d))***, a special boundary element must extend at least 12 in. into the web for flanged sections. Based on the 10.0 in. spacing of the #8 longitudinal bars in the web, use a 24 in. extension into the web.

10. Flexural compression zone width

$$b = \begin{cases} 216 \text{ in.} \\ 24 \text{ in.} \end{cases} > \frac{h_u}{16} = \frac{((11\times12)-28.5)}{16} = 6.47 \text{ in.}$$ ***ACI 318-19 (18.10.6.4(b))***

$$\frac{c}{l_w} = \frac{61.95}{384} = 0.16 < 3/8$$ ***ACI 318-19 (18.10.6.4(c))***

Therefore, the requirement of ***ACI 318-19 (18.10.6.4(b))*** is satisfied and the requirement of ***ACI 318-19 (18.10.6.4(c))*** is not applicable.

11. Special boundary element required transverse reinforcement

ACI 318-19 (18.10.6.4(e) and (f))

11.1. Confinement of the flange

The maximum vertical spacing of the special boundary element transverse reinforcement in the flange is equal to the following:

$$s_{max} = \text{Lesser of} \begin{Bmatrix} b/3 \\ \text{For Grade 80 bars, Lesser of} \begin{Bmatrix} 5\text{d}_\text{b} \\ 6 \text{ in.} \end{Bmatrix} \\ s_o = 4 + \left(\frac{14-h_x}{3}\right) \le 6 \text{ in.} \end{Bmatrix}$$

ACI 318-19 18.10.6.4(e)
ACI 318-19 Table 18.10.6.5(b)

$$s_{max} = \text{Lesser of} \begin{Bmatrix} 24/3 \\ \text{Lesser of} \begin{Bmatrix} 5\times1.0 \\ 6 \end{Bmatrix} \\ 4 + \left(\frac{14-11.17}{3}\right) \end{Bmatrix} = \begin{Bmatrix} 8.00 \text{ in.} \\ 5.00 \text{ in.} \\ 4.94 \text{ in.} \end{Bmatrix} = 4.94 \text{ in.}$$

Thus, use vertical spacing $s = 4.00$ in.

$$h_{xf} = \frac{(18\times12)-(2\times(0.75+0.625))-1.0}{20-1} = 11.17 \text{ in.}$$ (see Figure 8)

In the equation for s_o, the maximum center-to-center spacing of the laterally supported longitudinal bars, h_x, is equal to approximately 11.17 in. assuming #5 transverse reinforcement confining all the longitudinal bars in the flange.

$$b_{c,1} = 216 - (2\times0.75) = 214.5 \text{ in.}$$

$$b_{c,2} = 24 - (2 \times 0.75) = 22.5 \text{ in.}$$

$$A_{ch} = 214.5 \times 22.5 = 4{,}826.25 \text{ in.}^2$$

For confinement of the flange in the direction of analysis, $b_c = b_{c,1} = 214.5$ in.

$$A_{sh} = \text{larger of} \begin{Bmatrix} 0.3 \times s \times b_c \times \left(\dfrac{A_g}{A_{ch}} - 1 \right) \dfrac{f'_c}{f_{yt}} \\ 0.09 \times s \times b_c \times \dfrac{f'_c}{f_{yt}} \end{Bmatrix} = \begin{Bmatrix} 0.3 \times 4.0 \times 214.5 \times \left(\dfrac{24 \times 216}{4{,}826.3} - 1 \right) \times \dfrac{6}{80} \\ 0.09 \times 4.0 \times 214.5 \times \dfrac{6}{80} \end{Bmatrix} = \begin{Bmatrix} 1.43 \text{ in.}^2 \\ 5.79 \text{ in.}^2 \end{Bmatrix} = 5.79 \text{ in.}^2$$

ACI 318-19 (Table 18.10.6.4(g))

With #5 overlapping hoops and #5 crossties engaging all the longitudinal reinforcement in the flange, provided $A_{sh} = 20 \times 0.31 = 6.20 \text{ in.}^2 > 5.79 \text{ in.}^2$.

Use #5 hoops and #5 crossties around all the longitudinal bars spaced at 4.0 in. on center in the flanges.

11.2. Confinement of the web

The maximum vertical spacing of the special boundary element transverse reinforcement in the web is equal to the following:

$$s_{max} = \text{Lesser of} \begin{Bmatrix} b/3 \\ \text{For Grade 80 bars, Lesser of} \begin{Bmatrix} 5d_b \\ 6 \text{ in.} \end{Bmatrix} \\ s_o = 4 + \left(\dfrac{14 - h_x}{3}\right) \leq 6 \text{ in.} \end{Bmatrix}$$

ACI 318-19 18.10.6.4(e)
ACI 318-19 Table 18.10.6.5(b)

$$s_{max} = \text{Lesser of} \begin{Bmatrix} 24/3 \\ \text{Lesser of} \begin{Bmatrix} 5 \times 1.0 \\ 6 \end{Bmatrix} \\ 4 + \left(\dfrac{14 - 10.07}{3}\right) \end{Bmatrix} = \begin{Bmatrix} 8.00 \text{ in.} \\ 5.00 \text{ in.} \\ 5.31 \text{ in.} \end{Bmatrix} = 5.00 \text{ in.}$$

Thus, use vertical spacing s = 4.00 in.

$$h_{xw} = \frac{336 - \left(2 \times \left(0.75 + 0.625\right)\right) - 1.0}{33} = 10.07 \text{ in. (see Figure 8)}$$

In the equation for s_o, the maximum center-to-center spacing of the laterally supported longitudinal bars, h_x, is equal to 10.07 in.

$$b_{c,1} = 24 - \left(2 \times 0.75\right) = 22.5 \text{ in.}$$

$$b_{c,2} = 24 \text{ in.}$$

For confinement of the web in the direction of analysis, $b_c = b_{c,1}$ = 22.5 in. (see Figure 8)

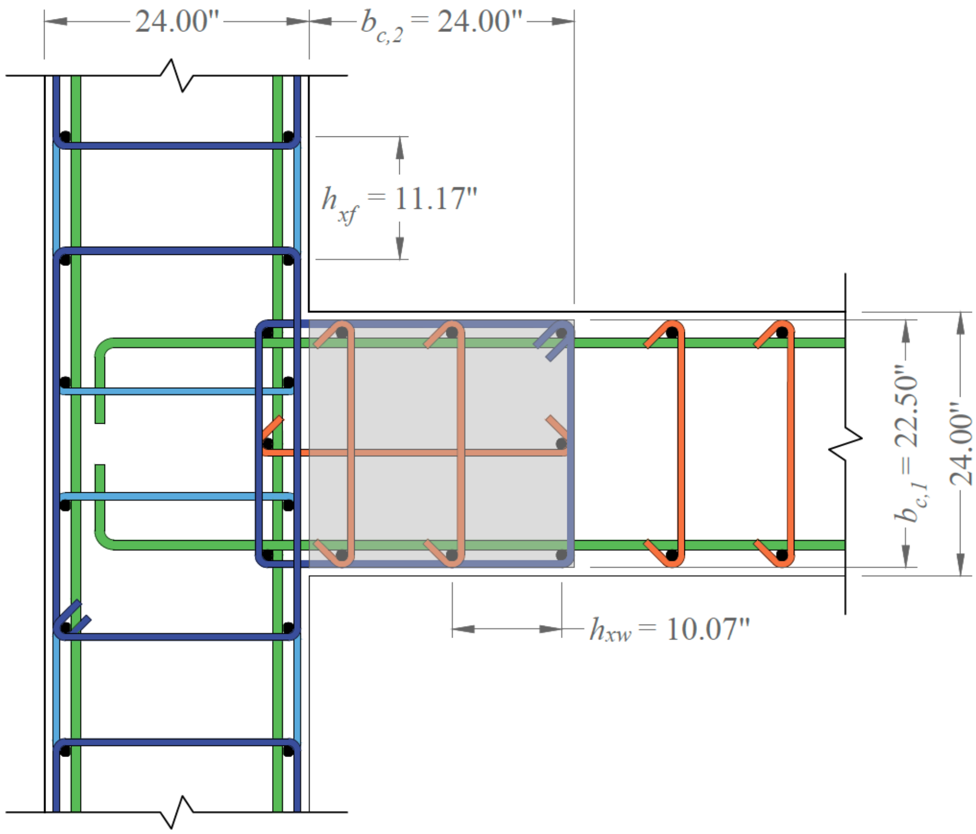


Figure 8 – Special boundary transverse reinforcement in the web of the wall

$$A_{sh} = \text{larger of}\begin{Bmatrix} 0.3\times s\times b_c\times\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f'_c}{f_{yt}} \\ 0.09\times s\times b_c\times\dfrac{f'_c}{f_{yt}}\end{Bmatrix} = \begin{Bmatrix} 0.3\times 4.0\times 22.5\times\left(\dfrac{24\times 24}{22.5\times 24}-1\right)\times\dfrac{6}{80} \\ 0.09\times 4.0\times 22.5\times\dfrac{6}{80}\end{Bmatrix} = \begin{Bmatrix} 0.14\text{ in.}^2 \\ 0.61\text{ in.}^2\end{Bmatrix} = 0.61\text{ in.}^2$$

ACI 318-19 (Table 18.10.6.4(g))

Where a vertical spacing of 4.0 in. is used in the web to match the vertical spacing of the transverse reinforcement in the flanges.

With a #5 hoop and one #5 crosstie, provided A_{sh} = 3 × 0.31 = 0.93 in.^{2} > 0.61 in.^{2}

In the direction perpendicular to the direction of analysis, $b_c = b_{c,2}$ = 24.0 in.

$$A_{sh} = \text{larger of}\begin{Bmatrix} 0.3\times s\times b_c\times\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f'_c}{f_{yt}} \\ 0.09\times s\times b_c\times\dfrac{f'_c}{f_{yt}}\end{Bmatrix} = \begin{Bmatrix} 0.3\times 4.0\times 24\times\left(\dfrac{24\times 24}{22.5\times 24}-1\right)\times\dfrac{6}{80} \\ 0.09\times 4.0\times 24\times\dfrac{6}{80}\end{Bmatrix} = \begin{Bmatrix} 0.14\text{ in.}^2 \\ 0.65\text{ in.}^2\end{Bmatrix} = 0.65\text{ in.}^2$$

ACI 318-19 (Table 18.10.6.4(g))

With a #5 hoop and two #5 crossties, provided A_{sh} = 3 × 0.31 = 0.93 in.^{2} > 0.65 in.^{2} (one of the hoop legs is in the flange, so the area of that leg is not included in the provided area of transverse reinforcement).

Use a #5 hoop and #5 crossties spaced at 4.0 in. on center in the web.

12. Transverse Reinforcement Required by ACI 18.10.6.5 at the Ends of the Wall

It can be determined that a reinforcement ratio greater than 400 / f_y = 0.005 must be provided in the flanges in stories 24 and above. Therefore, transverse reinforcement in accordance with ***ACI 318-19 (18.10.6.5)*** is required in the flanges in these stories. The size of the transverse reinforcement must satisfy the requirements of ***ACI 318-19 (25.7.2.2)***.

13. Reinforcement Details

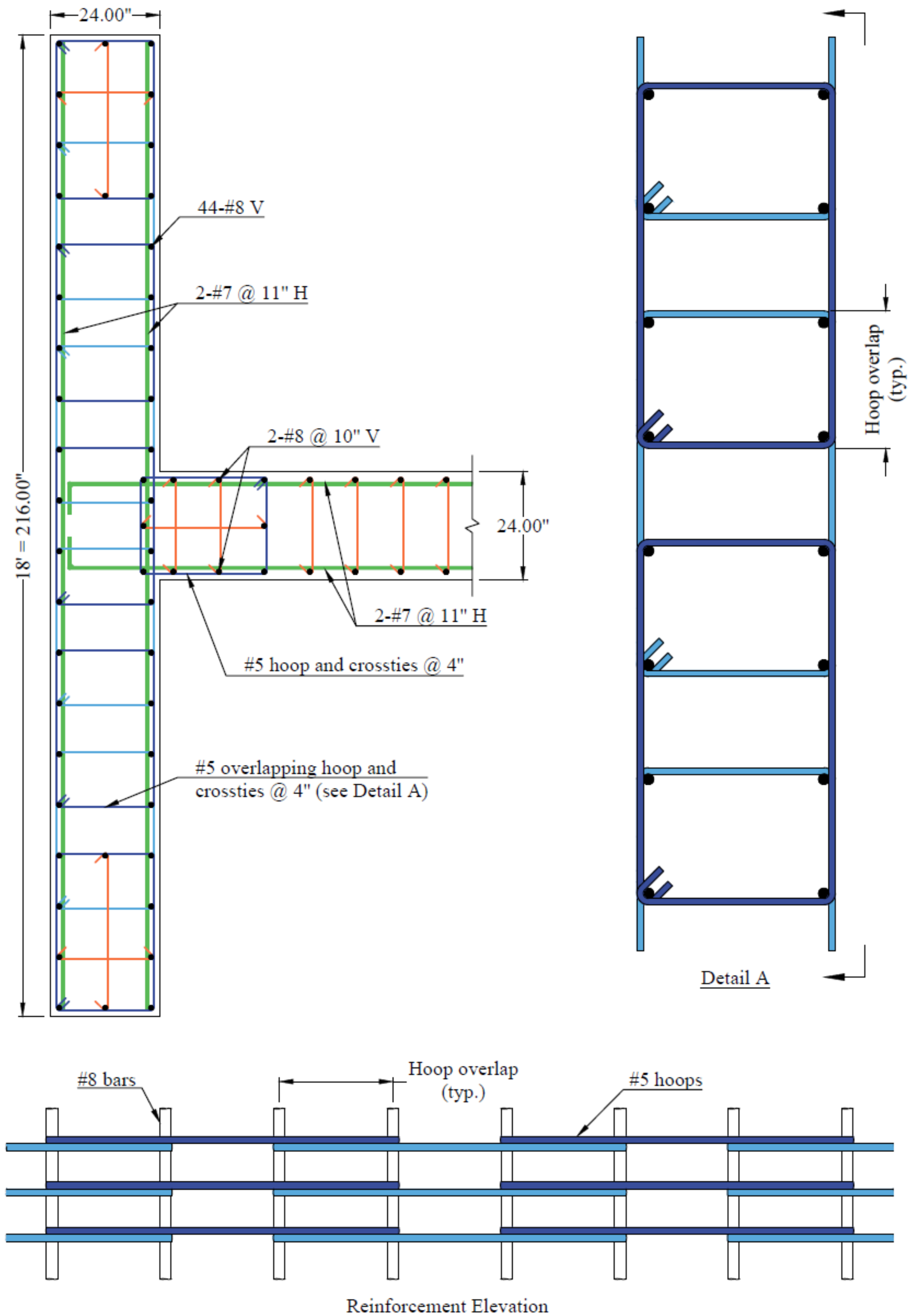


Figure 9 – Reinforcement details for the wall

Reinforcement details for this wall are given in the Figure above. The special transverse reinforcement in the flanges consists of a series of overlapping hoops and supplemental 135-degree crossties, similar to Detail (b) depicted in ***ACI 318-19 (Figure R18.10.6.4a)***. The geometry of the overlapping hoops depicted in Detail A of the Figure above satisfies the requirements in ***ACI 318-19 (18.10.6.4(f))***.

Comments. For a construction sequence where (1) a wall is constructed to the underside of the slab, (2) the slab is constructed, and (3) the wall above the slab is constructed, the concrete within the thickness of the slab at the location of the special boundary elements must have a specified compressive strength of at least 0.7 times the specified compressive strength of the wall ***(ACI 318-19 (18.10.6.4(h))***. In this example, the slab in floor levels 1 through 10 must have a minimum compressive strength of 0.7 × 6,000 = 4,200 psi. For a construction sequence where the wall is constructed ahead of the slab (for example, where slip forms are used to construct the wall), this requirement is not applicable.

14. Column Interaction Diagram - spColumn Software

spColumn is a StructurePoint software program that performs the analysis and design of reinforced concrete sections subjected to axial force combined with uniaxial or biaxial bending. Using the provisions of the Strength Design Method and Unified Design Provisions, slenderness considerations are used for moment magnification due to second order effect (P-Delta) for sway and non-sway frames.

For this wall section, investigation mode was used with factored loads and no slenderness considerations using ACI 318-19.

14.1. Design Strength Interaction Diagram

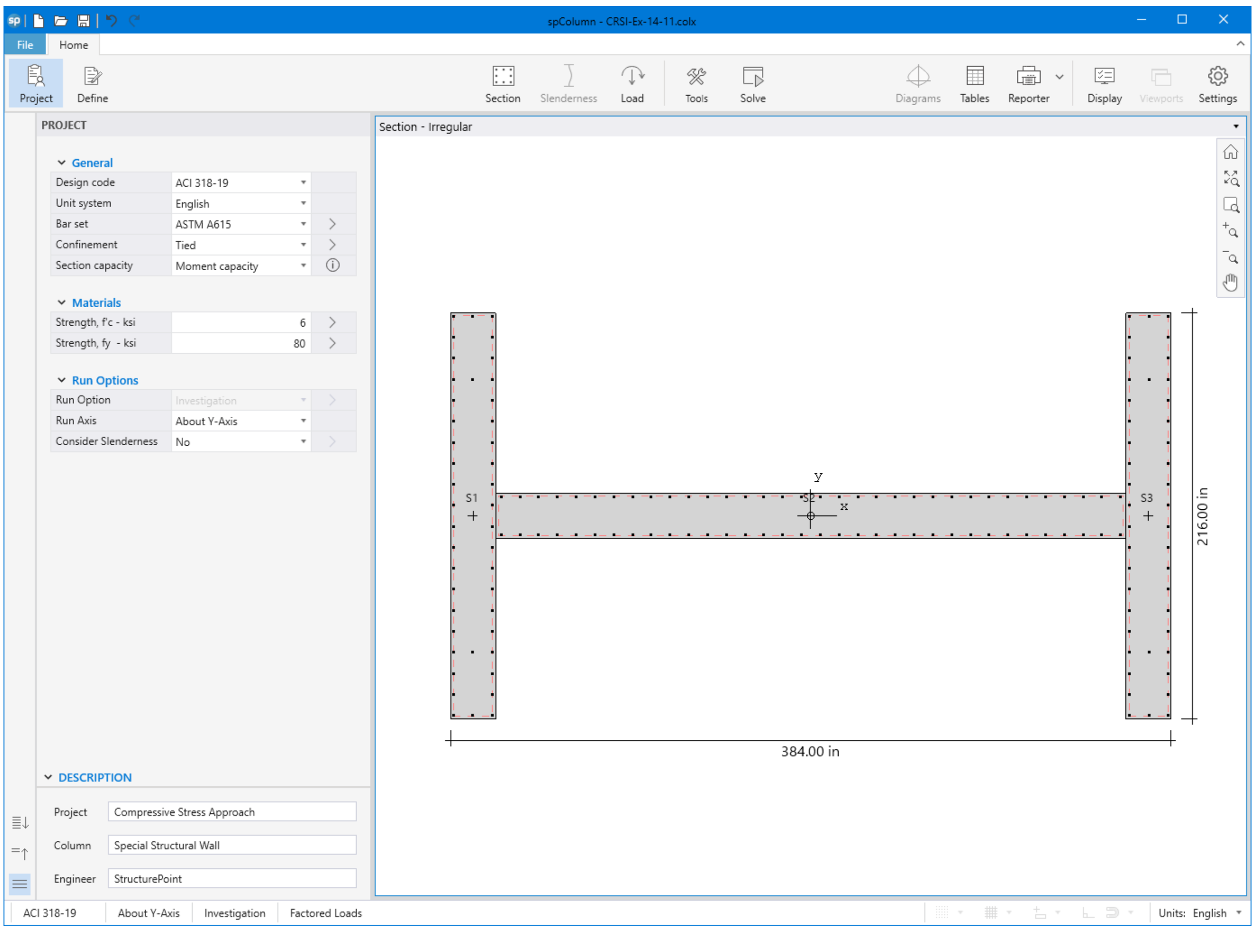
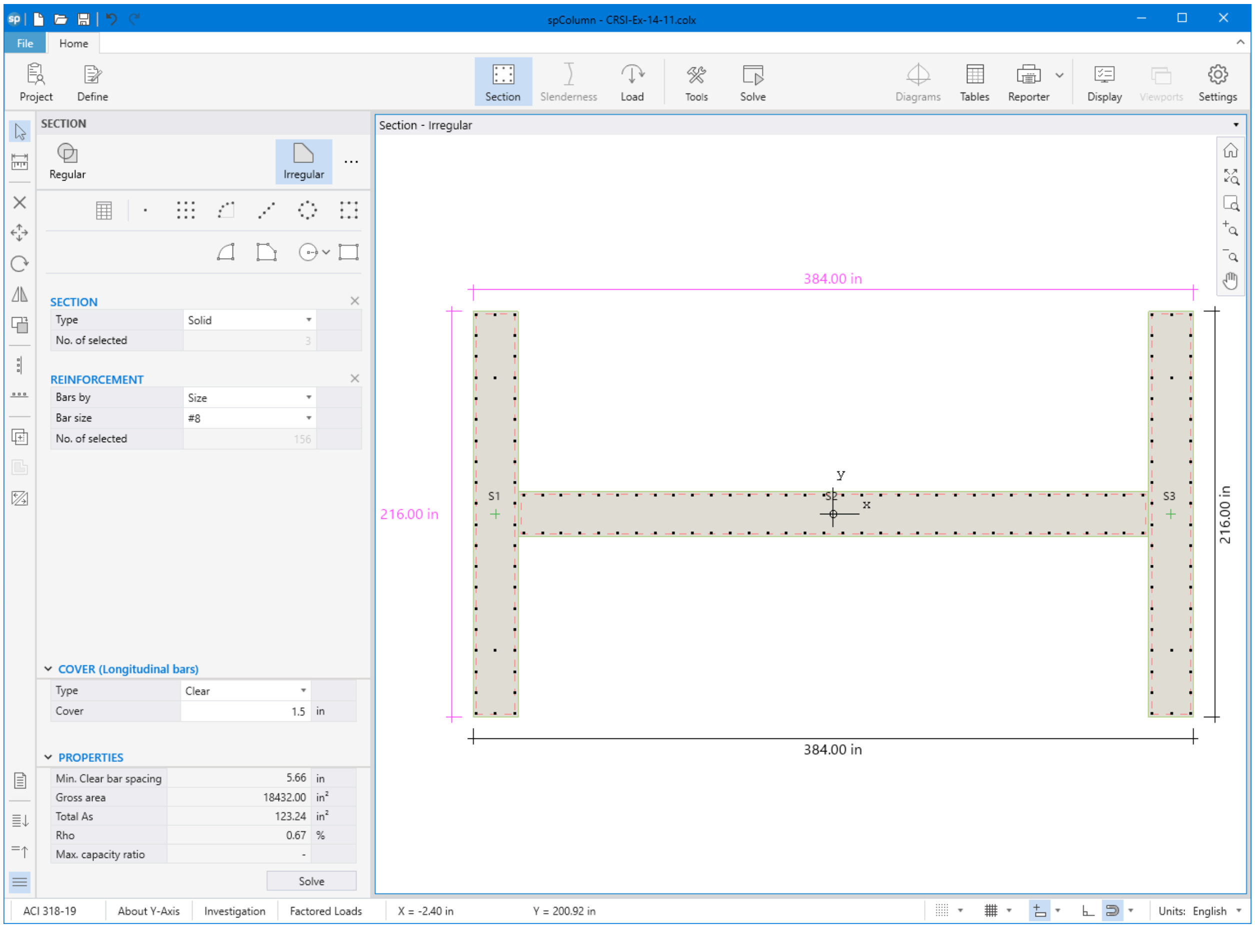


Figure 10 – spColumn Interface



<u>Figure 11 – spColumn Model Editor</u>

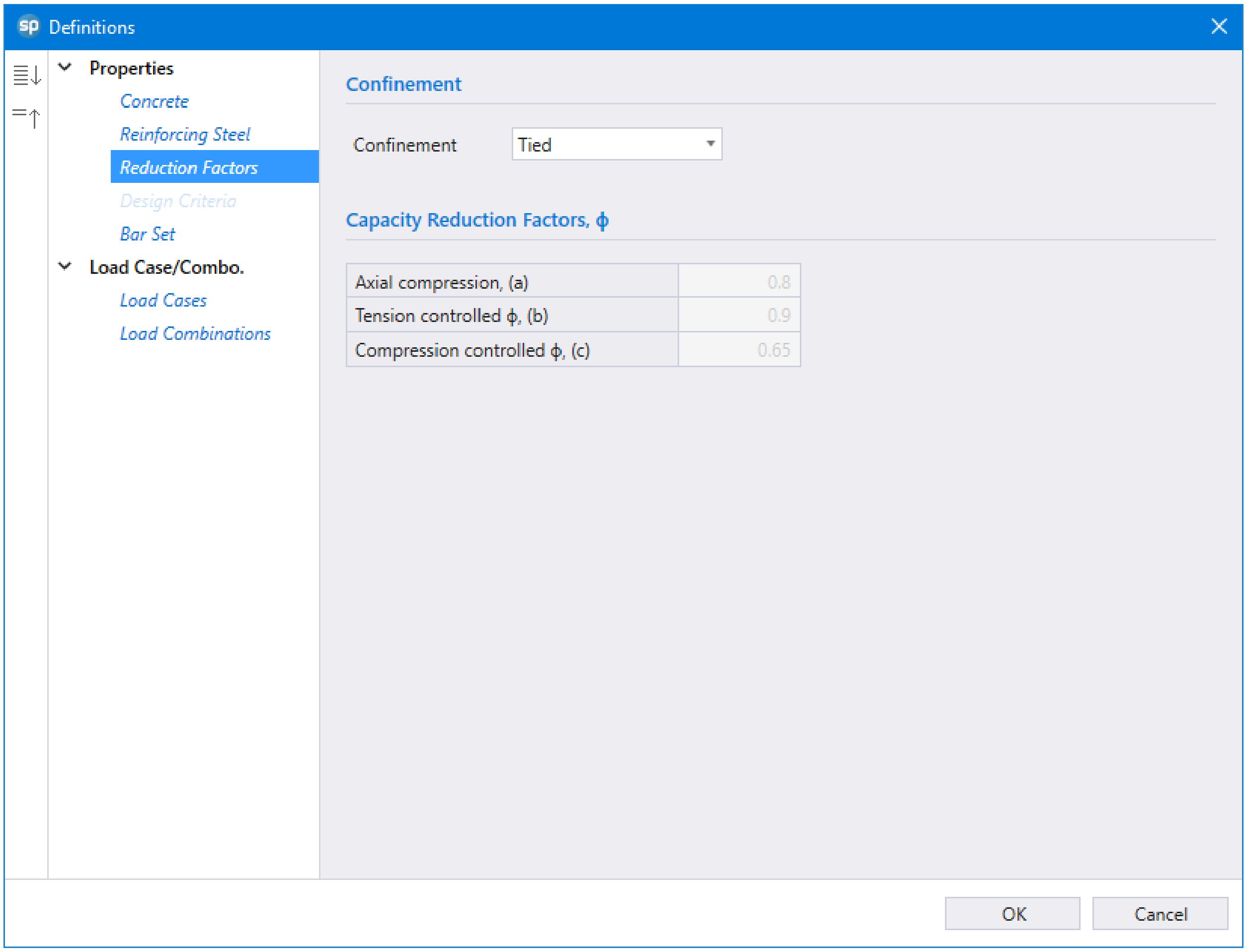


Figure 12 – Defining Reduction Factors (spColumn)

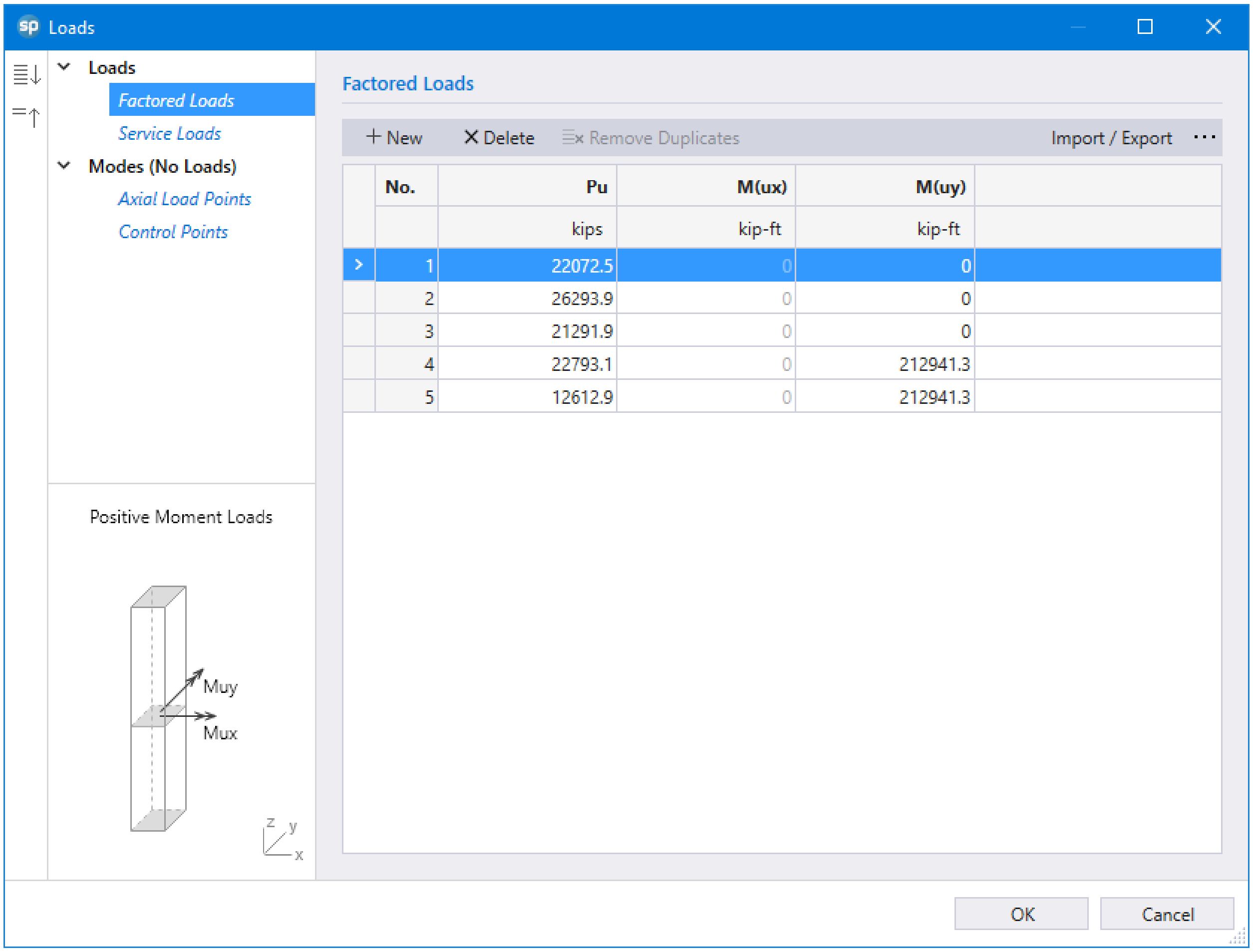


Figure 13 – Defining Loads / Modes (spColumn)

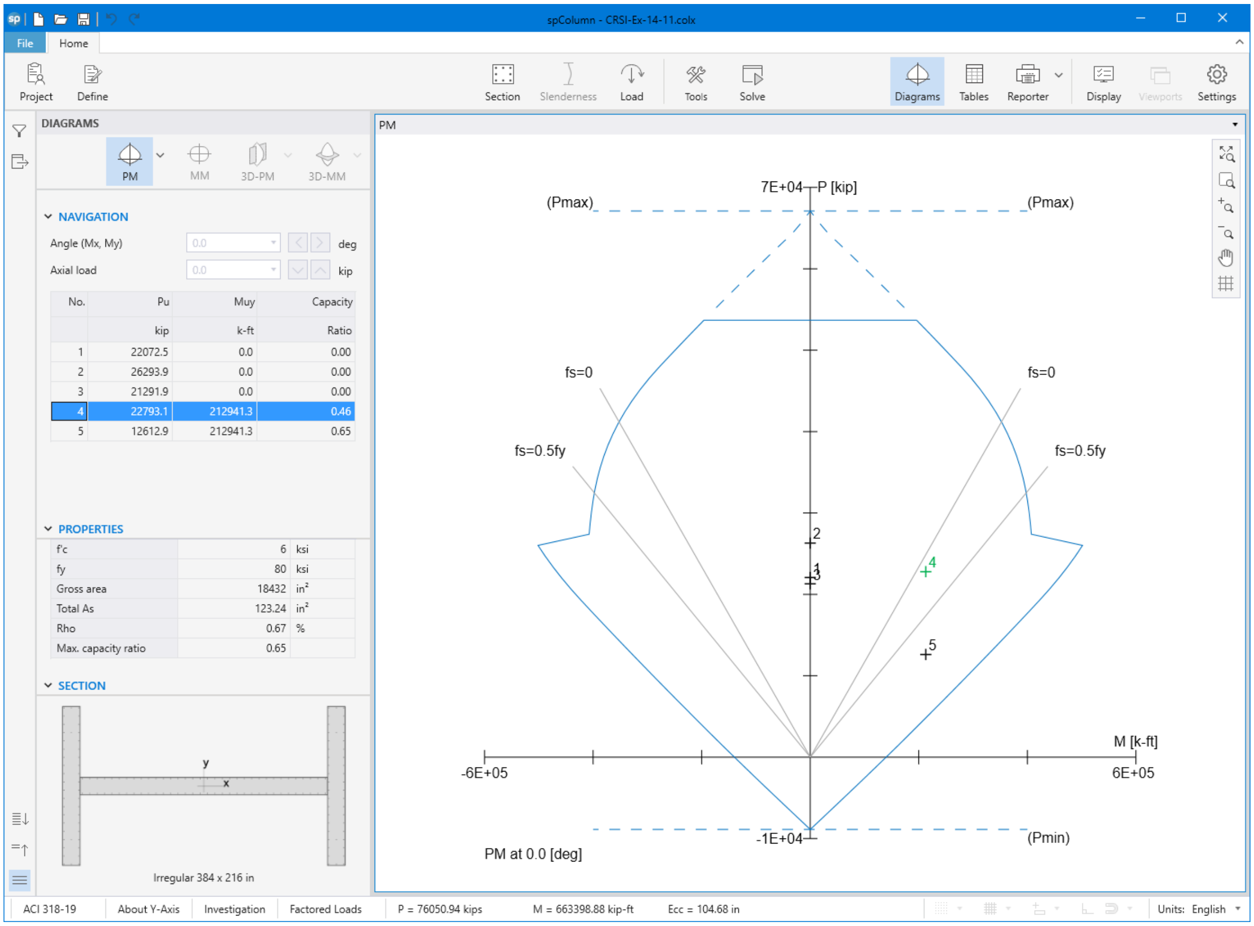
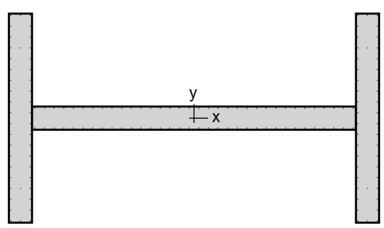


Figure 14 – Wall Design Strength Interaction Diagram about the Y-Axis (spColumn)

spColumn v10.10 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections



Contents

List of Figures

1. General Information

File Name	E:\StructurePoint\spColumn\CRSI-Ex-14-11.colx
Project	Compressive Stress Approach
Column	Special Structural Wall
Engineer	StructurePoint
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Y - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	6	ksi
E_c	4415.21	ksi
f_c	5.1	ksi
ε_u	0.003	in/in
β_1	0.75	

2.2. Steel

Type	Standard	
f_y	80	ksi
E_s	29000	ksi
ε_{ty}	0.00275862	in/in

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	18432	in^2
I_x	4.06979e+007	in^4
I_y	4.12287e+008	in^4
r_x	46.9894	in
r_y	149.559	in
X_o	0	in
Y_o	0	in

3.2. Section Figure

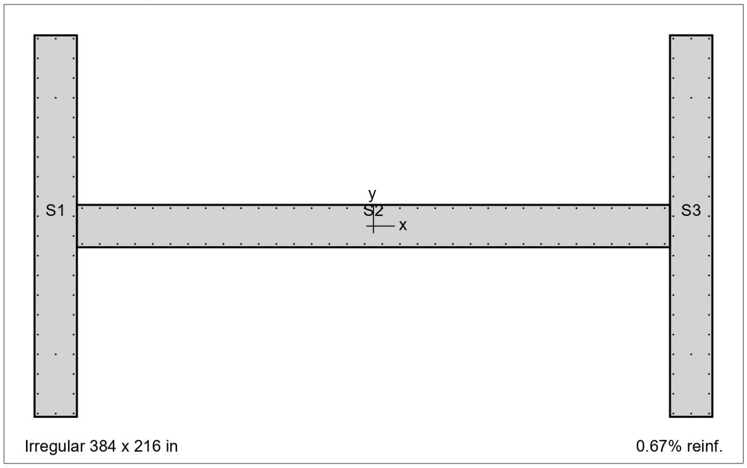


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-192.0	108.0	2	-192.0	-108.0	3	-168.0	-108.0
4	-168.0	108.0						

3.3.2. S2

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	-168.0	12.0	2	-168.0	-12.0	3	168.0	-12.0
4	168.0	12.0						

3.3.3. S3

Points	X in	Y in	Points	X in	Y in	Points	X in	Y in
1	192.0	108.0	2	192.0	-108.0	3	168.0	-108.0
4	168.0	108.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter in	Area in^2	Bar	Diameter in	Area in^2	Bar	Diameter 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			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	123.24 in^2
Rho	0.67 %
Minimum clear spacing	5.66 in

(Note: Rho < 1.0%)

4.4. Bars Provided

Area	X	Y	Area	X	Y	Area	X	Y
in^2	in	in	in^2	in	in	in^2	in	in
0.79	-190.1	-106.1	0.79	-190.1	-95.0	0.79	-190.1	-83.8
0.79	-190.1	-72.6	0.79	-190.1	-61.4	0.79	-190.1	-50.3
0.79	-190.1	-39.1	0.79	-190.1	-27.9	0.79	-190.1	-16.8
0.79	-190.1	-5.6	0.79	-190.1	5.6	0.79	-190.1	16.8
0.79	-190.1	27.9	0.79	-190.1	39.1	0.79	-190.1	50.3
0.79	-190.1	61.4	0.79	-190.1	72.6	0.79	-190.1	83.8
0.79	-190.1	95.0	0.79	-190.1	106.1	0.79	-180.0	-106.1
0.79	-180.0	106.1	0.79	-169.9	-106.1	0.79	-169.9	-95.0
0.79	-169.9	-83.8	0.79	-169.9	-72.6	0.79	-169.9	-61.4
0.79	-169.9	-50.3	0.79	-169.9	-39.1	0.79	-169.9	-27.9
0.79	-169.9	-16.8	0.79	-169.9	-5.6	0.79	-169.9	5.6
0.79	-169.9	16.8	0.79	-169.9	27.9	0.79	-169.9	39.1
0.79	-169.9	50.3	0.79	-169.9	61.4	0.79	-169.9	72.6
0.79	-169.9	83.8	0.79	-169.9	95.0	0.79	-169.9	106.1
0.79	-180.0	72.6	0.79	-180.0	-72.6	0.79	165.0	10.1
0.79	165.0	-10.1	0.79	-165.0	-10.1	0.79	-165.0	10.1
0.79	-155.0	10.1	0.79	-145.0	10.1	0.79	-135.0	10.1
0.79	-125.0	10.1	0.79	-115.0	10.1	0.79	-105.0	10.1
0.79	-95.0	10.1	0.79	-85.0	10.1	0.79	-75.0	10.1
0.79	-65.0	10.1	0.79	-55.0	10.1	0.79	-45.0	10.1
0.79	-35.0	10.1	0.79	-25.0	10.1	0.79	-15.0	10.1
0.79	-5.0	10.1	0.79	5.0	10.1	0.79	15.0	10.1
0.79	25.0	10.1	0.79	35.0	10.1	0.79	45.0	10.1
0.79	55.0	10.1	0.79	65.0	10.1	0.79	75.0	10.1
0.79	85.0	10.1	0.79	95.0	10.1	0.79	105.0	10.1
0.79	115.0	10.1	0.79	125.0	10.1	0.79	135.0	10.1
0.79	145.0	10.1	0.79	155.0	10.1	0.79	-145.0	-10.1
0.79	-135.0	-10.1	0.79	-125.0	-10.1	0.79	-115.0	-10.1

Area	X	Y	Area	X	Y	Area	X	Y
in^2	in	in	in^2	in	in	in^2	in	in
0.79	-105.0	-10.1	0.79	-95.0	-10.1	0.79	-85.0	-10.1
0.79	-75.0	-10.1	0.79	-65.0	-10.1	0.79	-55.0	-10.1
0.79	-45.0	-10.1	0.79	-35.0	-10.1	0.79	-25.0	-10.1
0.79	-15.0	-10.1	0.79	-5.0	-10.1	0.79	5.0	-10.1
0.79	15.0	-10.1	0.79	25.0	-10.1	0.79	35.0	-10.1
0.79	45.0	-10.1	0.79	55.0	-10.1	0.79	65.0	-10.1
0.79	75.0	-10.1	0.79	85.0	-10.1	0.79	95.0	-10.1
0.79	105.0	-10.1	0.79	115.0	-10.1	0.79	125.0	-10.1
0.79	135.0	-10.1	0.79	145.0	-10.1	0.79	155.0	-10.1
0.79	-155.0	-10.1	0.79	190.1	-95.0	0.79	190.1	-83.8
0.79	190.1	-72.6	0.79	190.1	-61.4	0.79	190.1	-50.3
0.79	190.1	-39.1	0.79	190.1	-27.9	0.79	190.1	-16.8
0.79	190.1	-5.6	0.79	190.1	5.6	0.79	190.1	16.8
0.79	190.1	27.9	0.79	190.1	39.1	0.79	190.1	50.3
0.79	190.1	61.4	0.79	190.1	72.6	0.79	190.1	83.8
0.79	190.1	95.0	0.79	190.1	106.1	0.79	180.0	-106.1
0.79	180.0	106.1	0.79	169.9	-106.1	0.79	169.9	-95.0
0.79	169.9	-83.8	0.79	169.9	-72.6	0.79	169.9	-61.4
0.79	169.9	-50.3	0.79	169.9	-39.1	0.79	169.9	-27.9
0.79	169.9	-16.8	0.79	169.9	-5.6	0.79	169.9	5.6
0.79	169.9	16.8	0.79	169.9	27.9	0.79	169.9	39.1
0.79	169.9	50.3	0.79	169.9	61.4	0.79	169.9	72.6
0.79	169.9	83.8	0.79	169.9	95.0	0.79	169.9	106.1
0.79	180.0	72.6	0.79	180.0	-72.6	0.79	190.1	-106.1

5. Control Points

About	Point	P	X-Moment	Y-Moment	NA Depth	d_t Depth	ε_t	ϕ	
		kip	k-ft	k-ft	in	in			
Y	@ Max compression	67102.0	0.00	-0.02	4749.25	382.12	-0.00276	0.65000	
Y	@ Allowable comp.	53681.6	0.00	195926.19	491.13	382.12	-0.00067	0.65000	
Y	@ f_s = 0.0	41195.7	0.00	352228.28	382.12	382.12	0.00000	0.65000	
Y	@ f_s = 0.5 f_y	32476.9	0.00	397760.22	261.77	382.12	0.00138	0.65000	
Y	@ Balanced point	27151.3	0.00	407624.50	199.07	382.12	0.00276	0.65000	
Y	@ Tension control	30250.3	0.00	535871.94	130.89	382.12	0.00576	0.90000	*
Y	@ Pure bending	0.0	0.00	139631.30	8.99	382.12	0.12452	0.90000	
Y	@ Max tension	-8873.3	0.00	-0.02	0.00	382.12	9.99999	0.90000	
-Y	@ Max compression	67102.0	0.00	-0.03	4749.25	382.13	-0.00276	0.65000	
-Y	@ Allowable comp.	53681.6	-0.03	-195926.33	491.13	382.13	-0.00067	0.65000	
-Y	@ f_s = 0.0	41195.7	0.00	-352228.28	382.13	382.13	0.00000	0.65000	
-Y	@ f_s = 0.5 f_y	32476.9	0.00	-397760.22	261.77	382.13	0.00138	0.65000	
-Y	@ Balanced point	27151.3	0.00	-407624.44	199.07	382.13	0.00276	0.65000	
-Y	@ Tension control	30250.3	0.00	-535871.94	130.89	382.13	0.00576	0.90000	*
-Y	@ Pure bending	0.0	-0.06	-139630.80	8.99	382.13	0.12452	0.90000	
-Y	@ Max tension	-8873.3	0.00	-0.02	0.00	382.13	9.99999	0.90000	

* Axial load capacity increase in transition zone between Balanced Point and Tension Control is not represented graphically and is not considered in section design and investigation.

6. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	P_u	M_{uy}	ϕP_n	ϕM_{ny}	NA Depth	ε_t	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	22072.50	0.00	22072.50	458616.69	55.60	0.01762	0.900	0.00
2	26293.90	0.00	26293.90	504114.69	93.93	0.00920	0.900	0.00

No.	Demand		Capacity		Parameters at Capacity			Capacity
	P_u	M_{uy}	ϕP_n	ϕM_{ny}	NA Depth	ε_t	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
3	21291.90	0.00	21291.90	448922.66	48.76	0.02051	0.900	0.00
4	22793.10	212941.30	22793.10	467210.25	61.95	0.01551	0.900	0.46
5	12612.90	212941.30	12612.90	327498.25	23.66	0.04546	0.900	0.65

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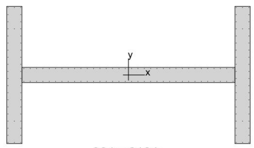
7. Diagrams

7.1. PM at θ=0 [deg]



384 x 216 in

General Information

Project	Compressiv...s Approach
Column	Special Structural Wall
Engineer	StructurePoint
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Y - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials

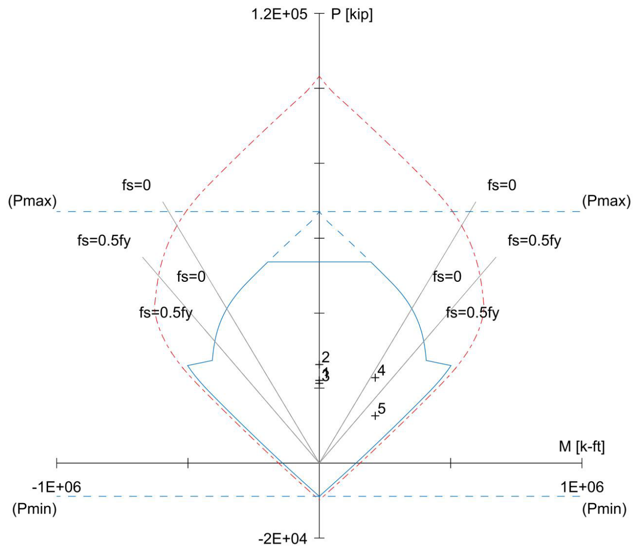
f'_c	6 ksi
E_c	4415.21 ksi
f_y	80 ksi
E_s	29000 ksi

Section

Type	Irregular
A_g	18432 in²
I_x	4.06979e+007 in⁴
I_y	4.12287e+008 in⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	123.24 in²
Rho	0.67 %
Min. clear spacing	5.66 in



PM at 0.0 [deg]

No.	P_u	M_{uy}	ϕP_n	ϕM_{ny}	Capacity
	kip	k-ft	kip	k-ft	Ratio
5	12612.9	212941.3	12612.90	327498.25	0.65
4	22793.1	212941.3	22793.10	467210.25	0.46
1	22072.5	0.0	22072.50	458616.69	0.00
2	26293.9	0.0	26293.90	504114.69	0.00
3	21291.9	0.0	21291.90	448922.66	0.00

Max. Capacity Ratio: 0.65

14.2. Nominal Strength Interaction Diagram with 1.25f_y and ϕ = 1.0 for M_{pr} Calculations

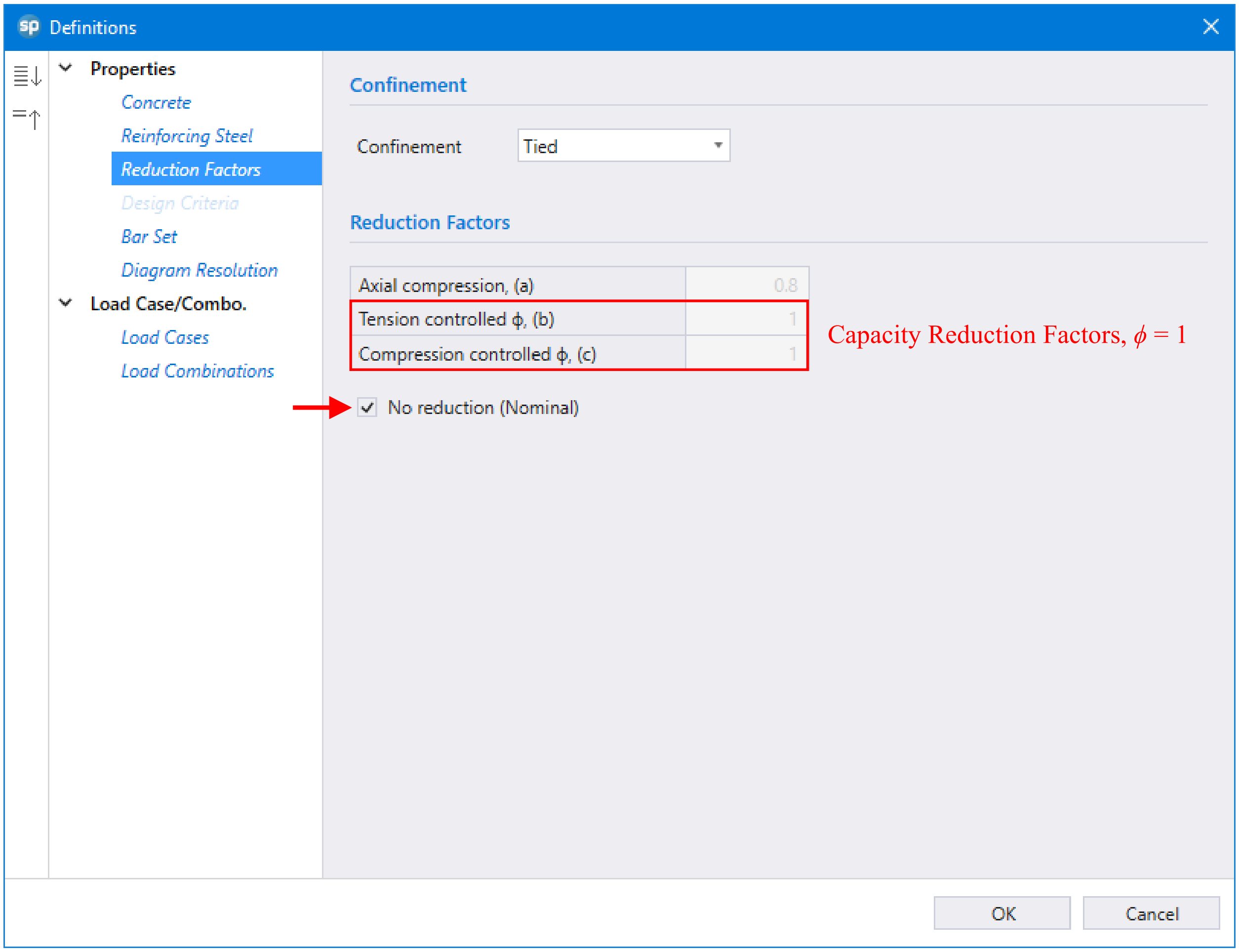


Figure 15 – Defining Reduction Factors (spColumn)

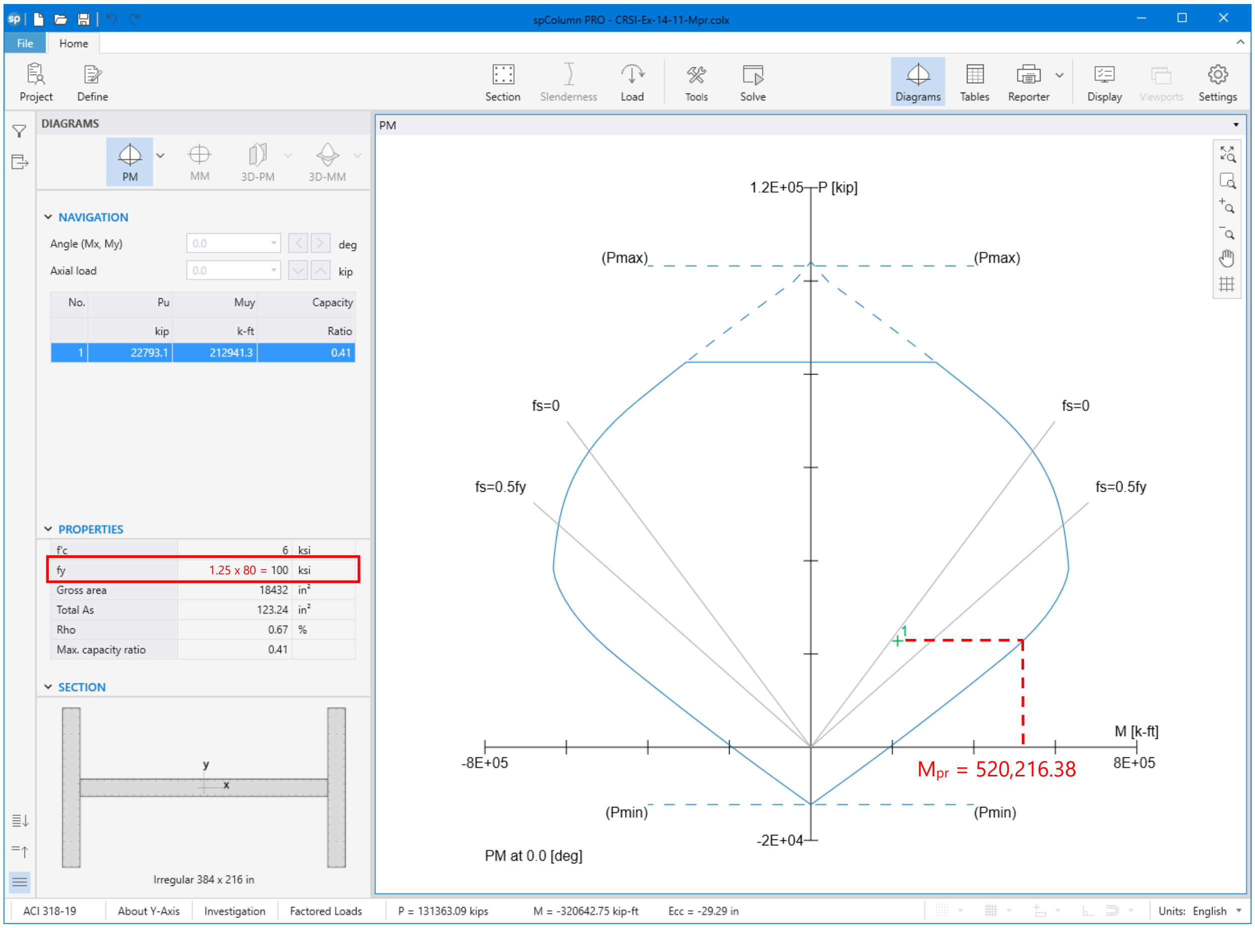


Figure 16 – Wall Nominal Strength Interaction Diagram about the Y-Axis (f_y' = 1.25 × f_y and ϕ = 1.0) (spColumn)

15. Summary and Comparison of Design Results

Table 4 – Comparison of Results			
Parameter	Reference	Hand	spColumn
ϕP_n, kip	22,793.10	22,793.10	22,793.10
ϕM_n, kip-ft	467,017.18	467,210.25	467,210.25
M_{pr}, kip-ft	520,152.00	520,216.38	520,216.38
c, in.	61.80	61.95	61.95
A, in.^{2}	18,432.00	18,432.00	18,432.00
I, in.^{4}	412,286,976	412,286,976	412,286,976

In all of the hand calculations and the reference used illustrated above, the results are in good agreement with the automated exact results obtained from the spColumn program.

16. Conclusions & Observations

ACI 318 allows two approaches to evaluate the need for special boundary elements to confine the concrete and restrain the longitudinal reinforcement in walls so buckling of the reinforcing bars doesn't occur:

1) Displacement-Based Approach **_ACI 318-19 (18.10.6.2)_**

   In this approach, special transverse reinforcement is provided in special structural walls compression zones where the strain at the extreme compression fiber exceeds a critical value when the wall is subjected to 1.5 times the design displacement which occurs at the top of the wall. Special boundary elements are required when:

   $$\frac{1.5\times\delta_u}{h_{wcs}} \geq \frac{l_w}{600\times c}$$ **_ACI 318-19 (18.10.6.2(a))_**

2) Compressive Stress Approach **_ACI 318-19 (18.10.3.1)_**

   In this approach, special transverse reinforcement is provided in special structural walls compression zones where:

   $$f_{cu} = \frac{P_u}{A_g} + \frac{M_u \times l_w}{2\times I_g} > 0.2 f_c'$$ **_ACI 318-19 (18.10.6.3)_**

   The special boundary elements may be discontinued where the combined stress is less than $0.15f_c'$.

In this example, the Compressive Stress Approach is used to determine the need for special boundary elements. The Displacement-Based Approach will be used in future examples.

ACI 318 distinguishes between ordinary and special reinforced concrete structural walls in the calculation of design shear forces and nominal shear strengths. For special structural walls, ACI 318 requires that the design shear forces obtained from lateral load analysis be amplified with appropriate load factors to account for flexural overstrength at critical sections where yielding of longitudinal reinforcement is expected, and dynamic amplification due to higher mode effects. The approach used to determine the amplified shear forces is similar to that used in New Zealand Standard 3101 (2006). **_ACI 318-19 (R18.10.3.1)_**

The nominal shear strength of special structural walls is given by ACI 318 (18.10.4.1). This equation recognizes the higher shear strength of walls with high shear-to-moment ratios. The nominal shear strength is given in terms of the gross area of the section resisting shear. For a rectangular section without openings, that term refers to the gross area of the cross section rather than to the product of the width and the effective depth. **_ACI 318-19 (R18.10.4)_**

ACI 318 indicates that the flexural strength of special structural (shear) walls is determined according to procedures commonly used for columns based on a strain compatibility analysis (Section 22.4 Axial Strength or Combined Flexural and Axial Strength). The generation of a design strength interaction diagram (or nominal interaction diagram for M_{pr} calculations) for these walls could be challenging and cumbersome due to the complexity of section geometry and/or reinforcement configurations and the use of a computer aid can save time and eliminate errors. StuctureP oint's spColumn program can, quickly, simply, and accurately generate the interaction diagram for all commonly encountered structural wall (shear and core walls) sections regardless of their complexity.

The reference mentioned that the wall covered in this example is part of the Seismic Force Resisting System (SFRS) for earthquake loads in the north-south direction. Because the axial force due to seismic forces acting along either of the principal plan axes is negligible, orthogonal load effects in accordance with ASCE/SEI need not be considered. Thus, the wall provided reinforcement must satisfy the effects from axial forces, bending moments, and shear forces in both orthogonal directions independently. spColumn allows user to generate the interaction diagram about either the X- or Y-Axes in Uniaxial bending run mode, and 3D failure surface accounting for the biaxial bending in two directions simultaneously. The added flexibility in the run options allow the user to perform a comprehensive analysis and design of special structural (shear) walls quickly, simply, and accurately by reducing time consumed and minimizing errors.

17. spColumn – Model Creation with Templates

Utilizing Templates is a quick and simple option for new models in spColumn. The user can select from a set of pre-defined templates and edit their properties for simple and quick model generation.

Each template focuses on a particular structural shape with a specific reinforcement pattern consistent with the shape. The user can edit the geometric dimensions of the shape and the bars' properties. Other shape-specific options may also be available for some templates. This modeling option is provided in spColumn Pro v11.00 and later. Contact StructurePoint for support in modeling with templates.

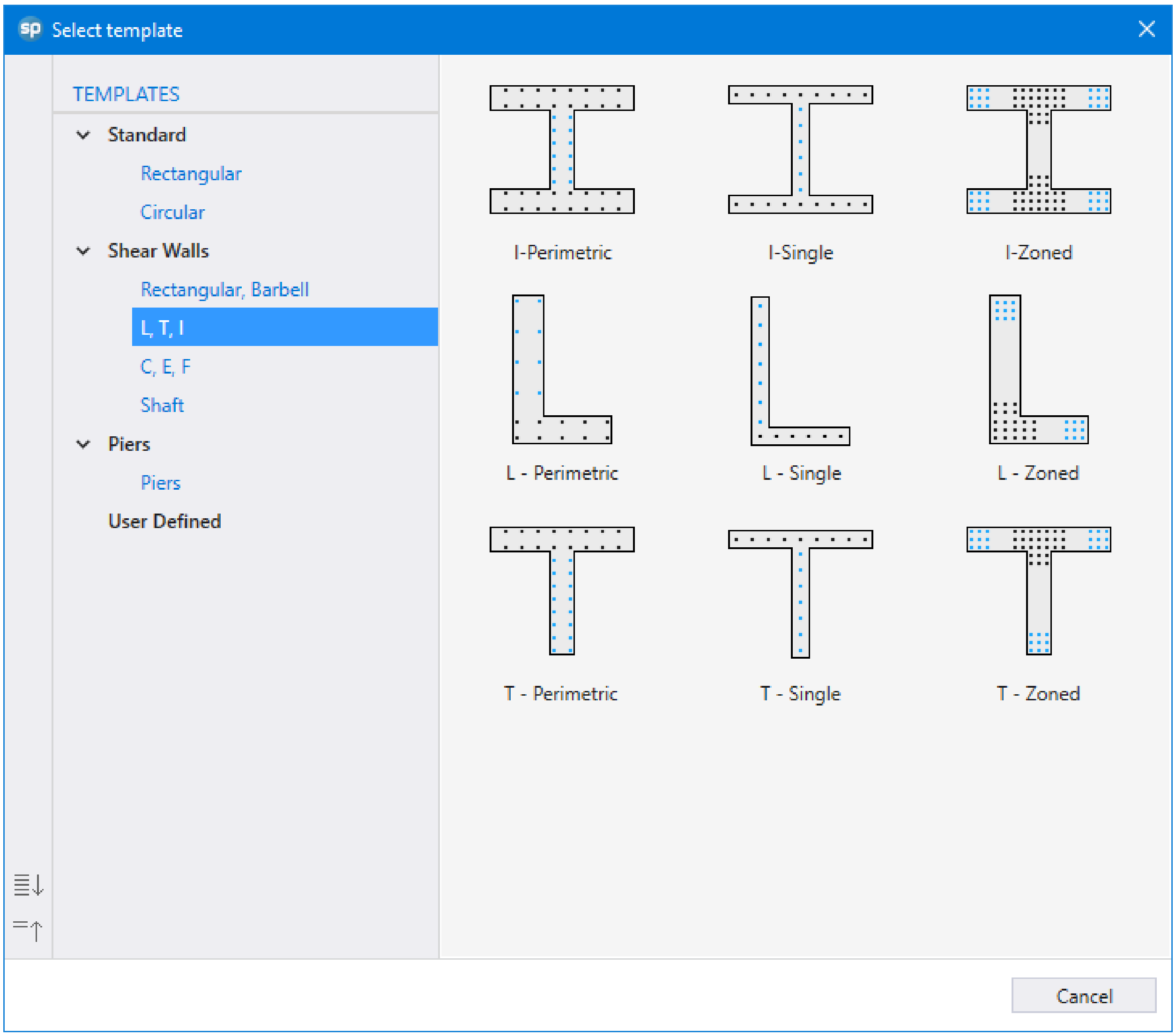


Figure 17 – spColumn Pro v11.00 Templates

The user can utilize the templates module to save time and eliminate errors associated with the creation of complex wall section geometry and/or reinforcement configurations as demonstrated in this example. "I-Zoned" template from "Shear Walls" module is used to create special structural (shear) wall covered in this example, the section then is imported to spColumn model editor to refine the reinforcement configuration for fine-tuning reinforcement bar locations.

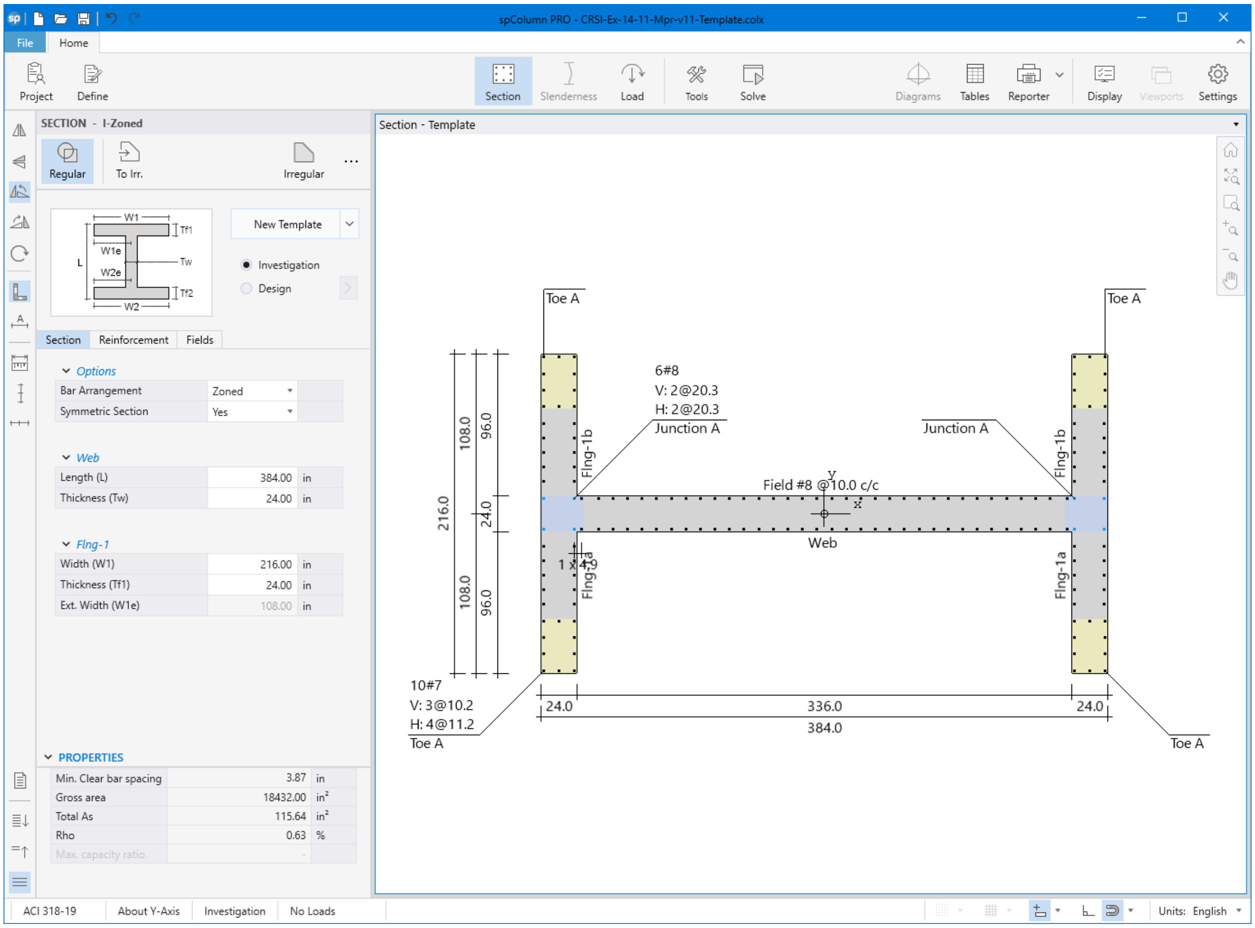


Figure 18 – spColumn v11.00 Pro Templates Editor

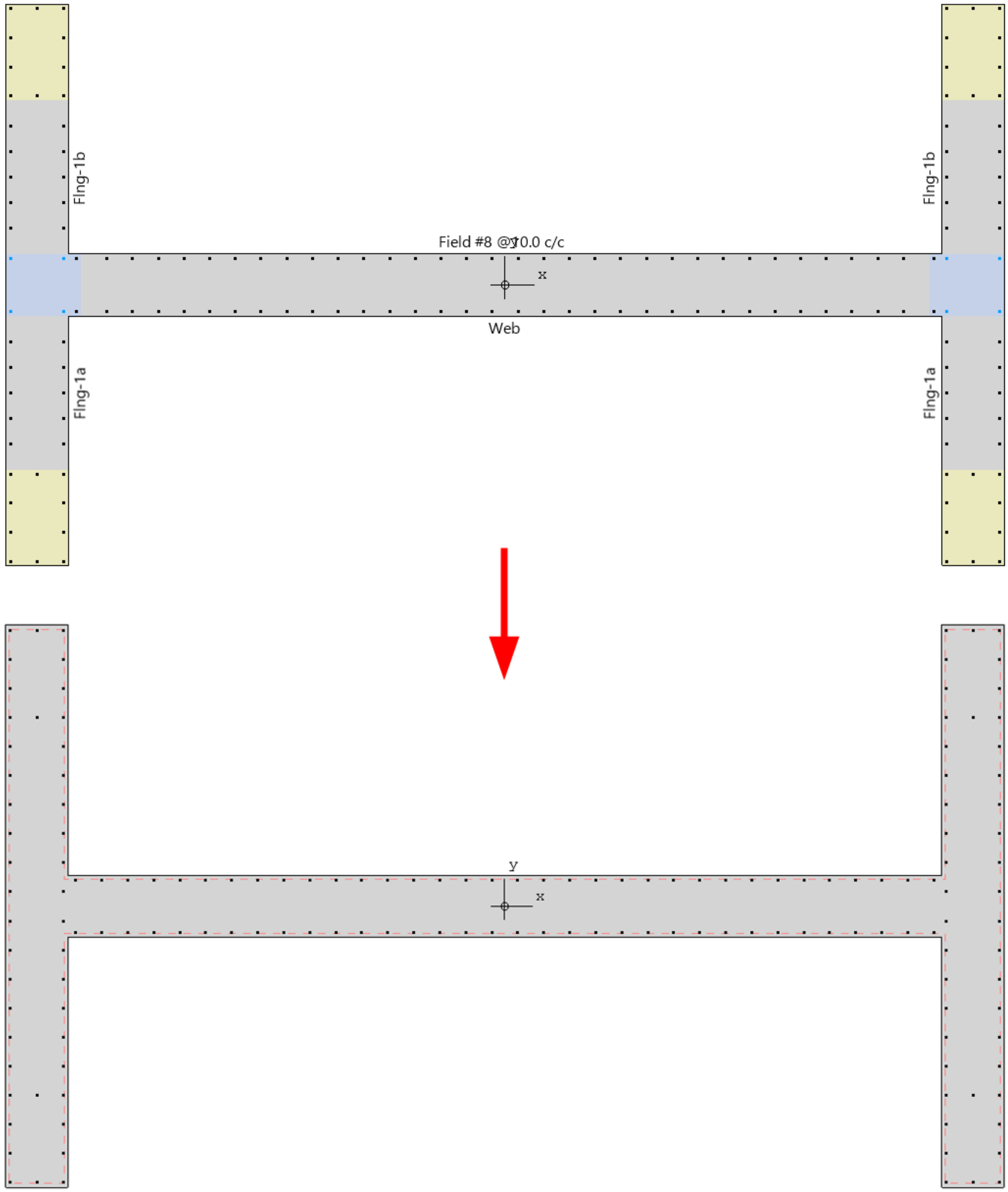


Figure 19 – Fine-Tuning Reinforcement Bar Locations – spColumn v11.00 Pro