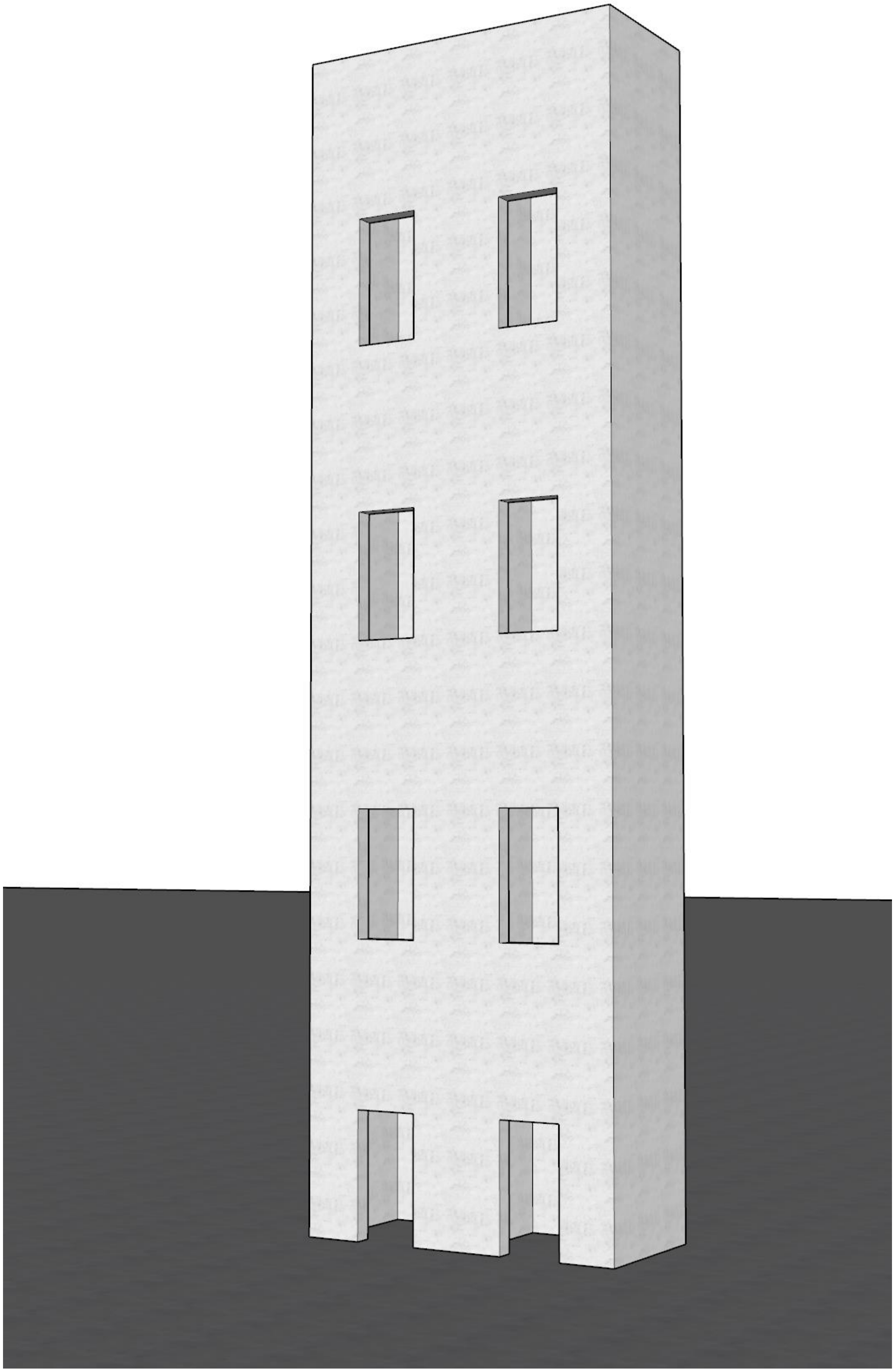
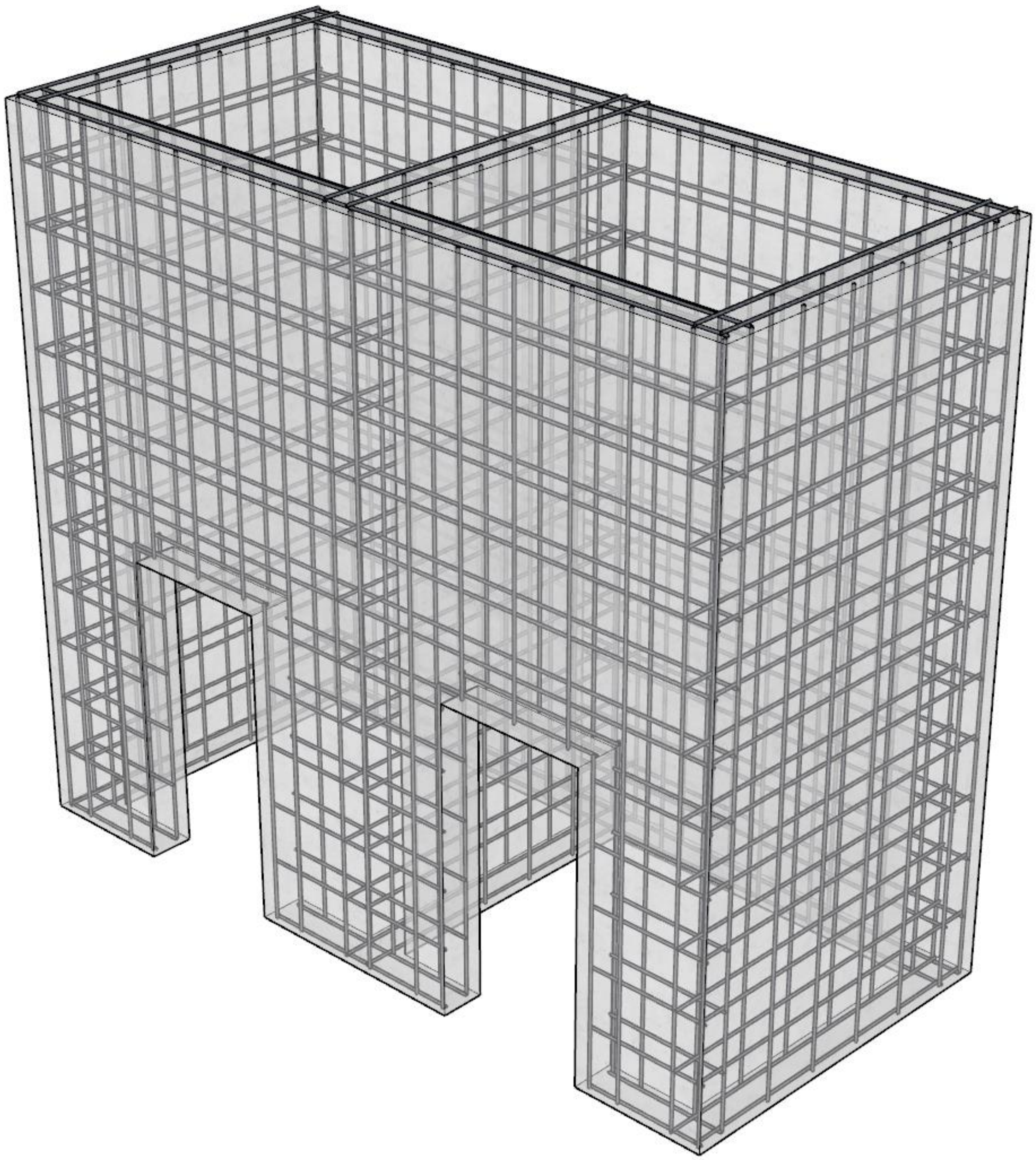
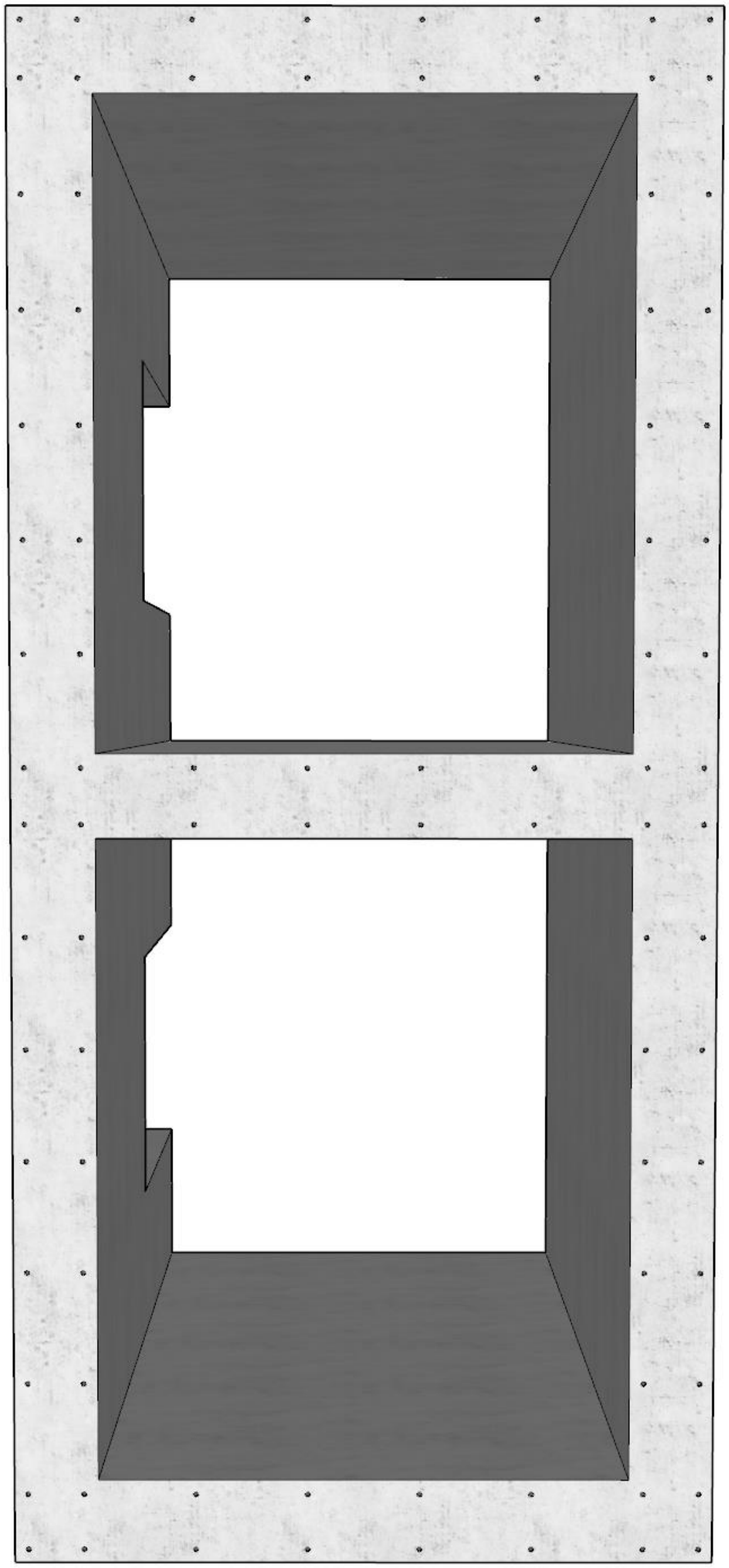
Building Elevator Reinforced Concrete Core Wall Design Strength – ACI 318-14







Building Elevator Reinforced Concrete Core Wall Design Strength – ACI 318-14

Reinforced concrete core walls are utilized in building framed with concrete as well as other framing materials such as steel and wood. Used in conjunction with concrete shear walls, core walls house elevator banks, stair cases, MEP chases, and many other service equipment and spaces. Along with important functions such as isolating equipment and elevator vibration and noise reduction, core wall systems regularly double as a building lateral load resistance system. In multi-story concrete, steel, and wood buildings, reinforced concrete cores are subject to significant axial loads coupled with simultaneous bending moments about two orthogonal axes (biaxial bending). This design example investigates the strength and capacity of a standard two-lift elevator bank reinforced concrete core wall shown below. The P-M interaction diagram about the strong axis (x-axis) is manually developed by determining seven key control points on the P-M interaction diagram. The hand calculated values are then compared with exact values from the complete interaction diagram generated by the spColumn engineering software program from StructurePoint.

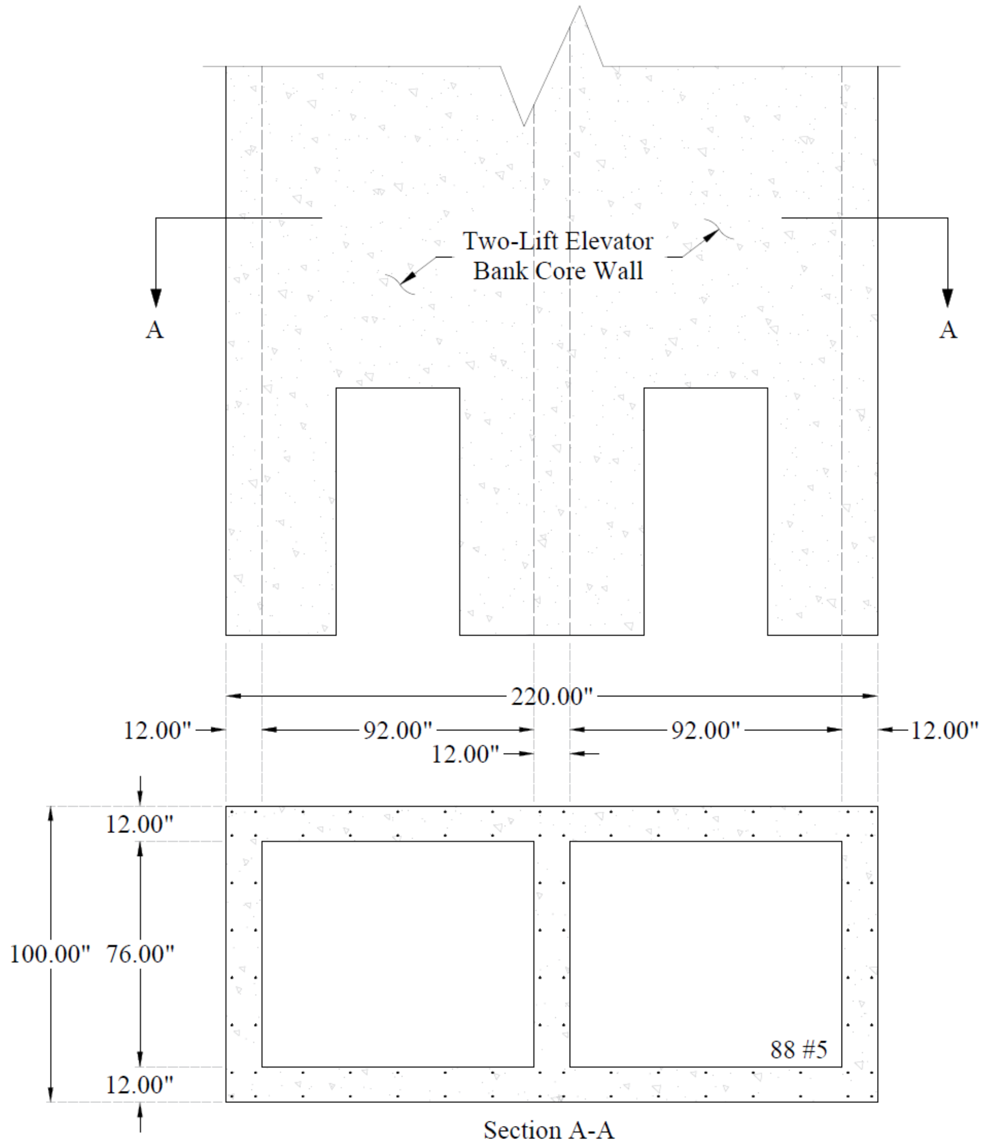


Figure 1 – Reinforced Concrete Core Wall Cross-Section

Contents

Code

Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)

References

spColumn Engineering Software Program Manual v10.0, STRUCTUREPOINT, 2021

"Interaction Diagram - Tied Reinforced Concrete Column" Design Example, STRUCTUREPOINT, 2017

"Interaction Diagram - Circular Reinforced Concrete Column" Design Example, STRUCTUREPOINT, 2020

"Interaction Diagram - Tied Reinforced Concrete Column with High-Strength Reinforcing Bars" Design Example, STRUCTUREPOINT, 2020

"Interaction Diagram - Dumbbell Concrete Shear Wall Unsymmetrical Boundary Elements" Design Example, STRUCTUREPOINT, 2018

Design Data

f_c' = 6,000 psi

f_y = 60,000 psi

Cover = 2 in. (to bar center)

The reinforcement size and location selected for this core wall section are shown in the following figure.

Detailed relevant steel bar and concrete shape data are tabulated below.

Table 1 - Reinforcement Data

Layer	Bar size	A_s/bar, in^2	# of bars	d, in
1	#5	0.31	8	2.0
2	#5	0.31	8	10.0
3	#5	0.31	4	26.0
4	#5	0.31	4	42.0
5	#5	0.31	4	58.0
6	#5	0.31	4	74.0
7	#5	0.31	4	90.0
8	#5	0.31	8	106.0
9	#5	0.31	8	114.0
10	#5	0.31	4	130.0
11	#5	0.31	4	146.0
12	#5	0.31	4	162.0
13	#5	0.31	4	178.0
14	#5	0.31	4	194.0
15	#5	0.31	8	210.0
16	#5	0.31	8	218.0

Table 2 - Concrete Shape Data

Part	h, in	b, in	A_c/part, in^2
1	12.0	100.0	1200.0
2	92.0	24.0	2208.0
3	12.0	100.0	1200.0
4	92.0	24.0	2208.0
5	12.0	100.0	1200.0
		$A_{c(total)}$, in^2	8016.0

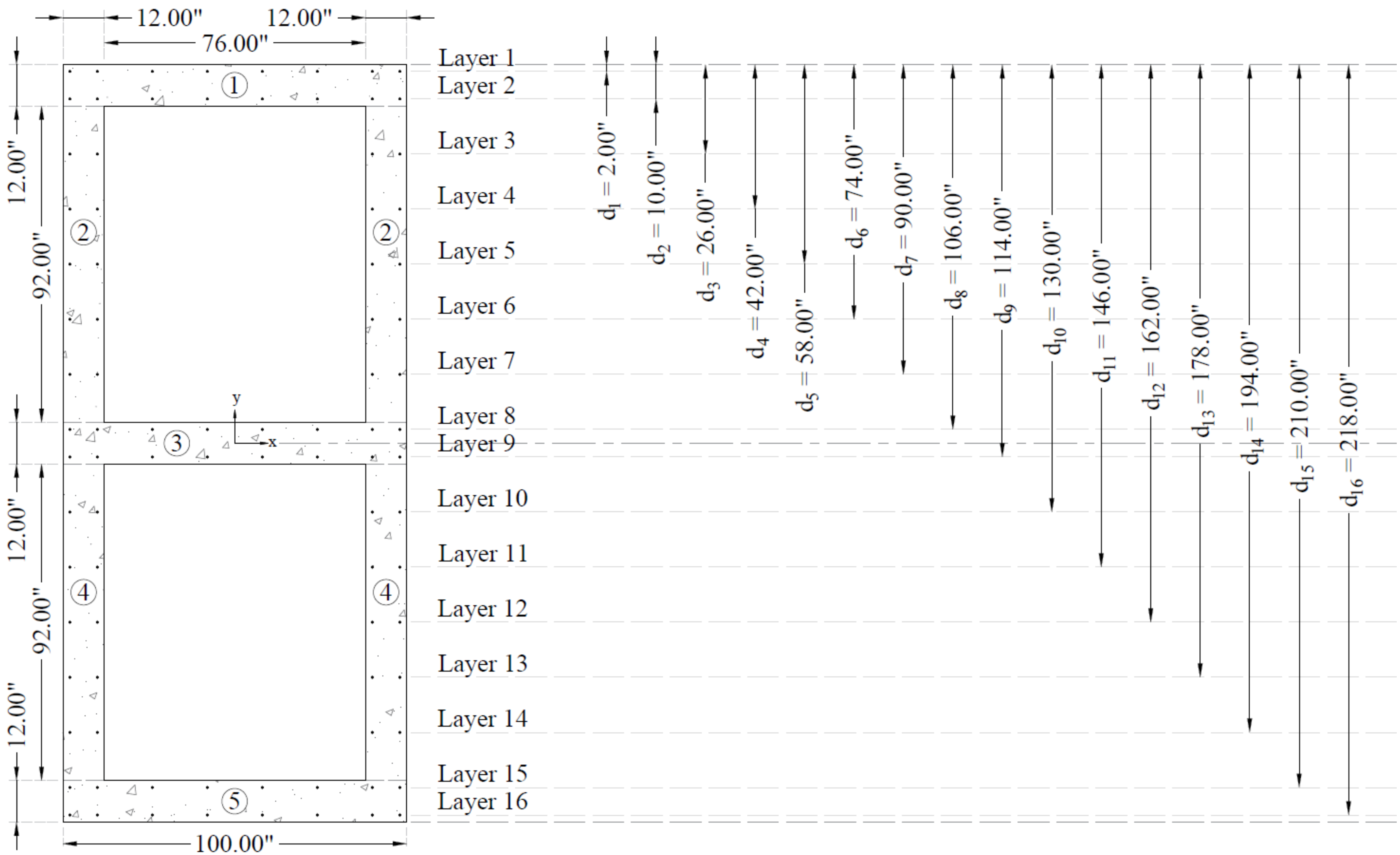


Figure 2 – Reinforced Concrete Core Wall – Cross-Section and Reinforcement Design Data

Solution

Use the traditional detailed approach to generate the interaction diagram for the concrete wall section shown above by determining the following seven control points for positive and negative moment about the x-axis:

Point 1: Maximum compression

Point 2: Bar stress near tension face equal to zero, ($f_s = 0$)

Point 3: Bar stress near tension face equal to 0.5 f_y ($f_s = 0.5\ f_y$)

Point 4: Bar stress near tension face equal to f_y ($f_s = f_y$)

Point 5: Bar strain near tension face equal to 0.005

Point 6: Pure bending

Point 7: Maximum tension

Several terms are used to facilitate the following calculations:

A_g = gross area of concrete section, in^2.

$\bar{y}$ = geometric centroid location along the y-axis, in.

P_o = nominal axial compressive strength, kip

ϕP_o = factored axial compressive strength, kip

ϕM_o = moment strength associated with the factored axial compressive strength, kip-ft

$\phi P_{n,max}$ = maximum (allowable) factored axial compressive strength, kip

c = distance from the fiber of maximum compressive strain to the neutral axis, in.

a = depth of equivalent rectangular stress block, in.

A_p = gross area of equivalent rectangular stress block, in^2.

$\bar{y}_p$ = plastic centroid location along the y-axis, in.

C_c = compression force in equivalent rectangular stress block, kip

$\varepsilon_{s,i}$ = strain value in reinforcement layer i, in./in.

$C_{s,i}$ = compression force in reinforcement layer i, kip

$T_{s,i}$ = tension force in reinforcement layer i, kip

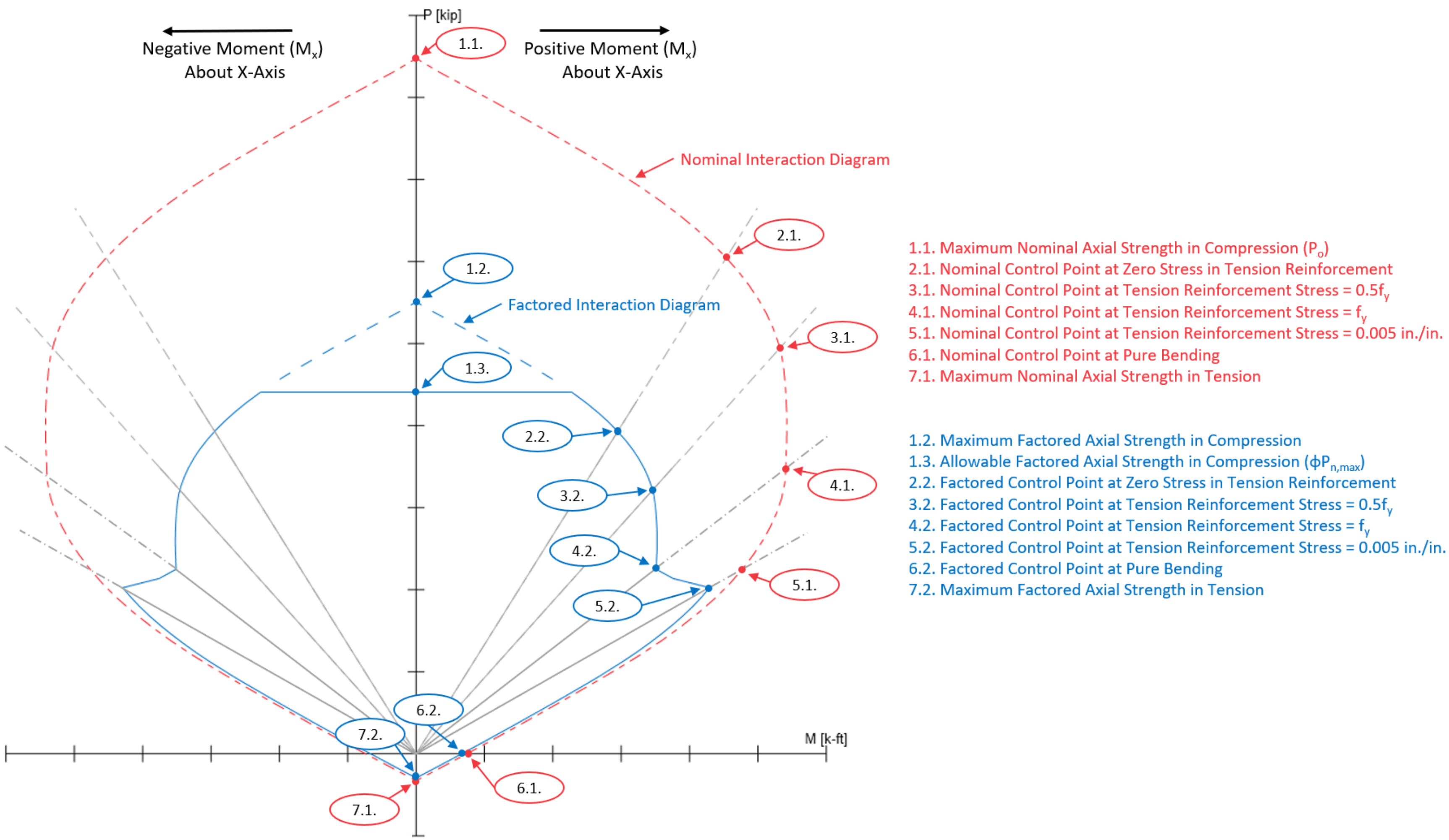


Figure 3 – Core Wall Section Interaction Diagram Control Points

1. Maximum Compression

1.1. Nominal axial compressive strength

From Tables 1 and 2:

Calculate total gross cross-sectional area:

$$A_g = b_1 \times h_1 + b_2 \times h_2 + b_3 \times h_3 + b_4 \times h_4 + b_5 \times h_5$$

$$A_g = 100 \times 12 + 24 \times 92 + 100 \times 12 + 24 \times 92 + 100 \times 12 = 8,016 \text{ in.}^2$$

Calculate the center of gravity (geometric centroid):

$$\bar{y} = \frac{\sum_{i=1}^{n=5} b_i \times h_i \times d_i}{\sum_{i=1}^{n=5} b_i \times h_i} = \frac{881,760 \text{ in.}^3}{8,016 \text{ in.}^2} = 110 \text{ in.}$$

Where d_i is the distance from the centroid of segment i to the reference point (top of the section).

Also due to symmetry about the x axis $\bar{y} = \frac{y}{2} = \frac{220 \text{ in.}}{2} = 110 \text{ in.}$

$$A_{st} = 88 \times 0.31 = 27.28 \text{ in.}^2$$

$$P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st}$$ ***ACI 318-14 (22.4.2.2)***

$$P_o = 0.85 \times 6,000 \times (8,016 - 27.28) + 60,000 \times 27.28 = 42,379 \text{ kips}$$

Since the section is regular (symmetrical) about the x-axis, the moment capacity associated with the maximum axial compressive strength is equal to zero.

$$M_o = 0 \text{ kip-ft}$$

1.2. Factored axial compressive strength

$$\phi = 0.65$$ ***ACI 318-14 (Table 21.2.2)***

$$\phi P_o = 0.65 \times 42,379 = 27,546.5 \text{ kips}$$

$$\phi M_o = 0 \text{ kip-ft}$$

1.3. Maximum (allowable) factored axial compressive strength

$$\phi P_{n,max} = 0.80 \times \phi P_o = 0.80 \times 27,546.5 = 22,037.2 \text{ kips}$$ ***ACI 318-14 (Table 22.4.2.1)***

2. Bar Stress Near Tension Face Equal to Zero, ($\varepsilon_s = f_s = 0$)

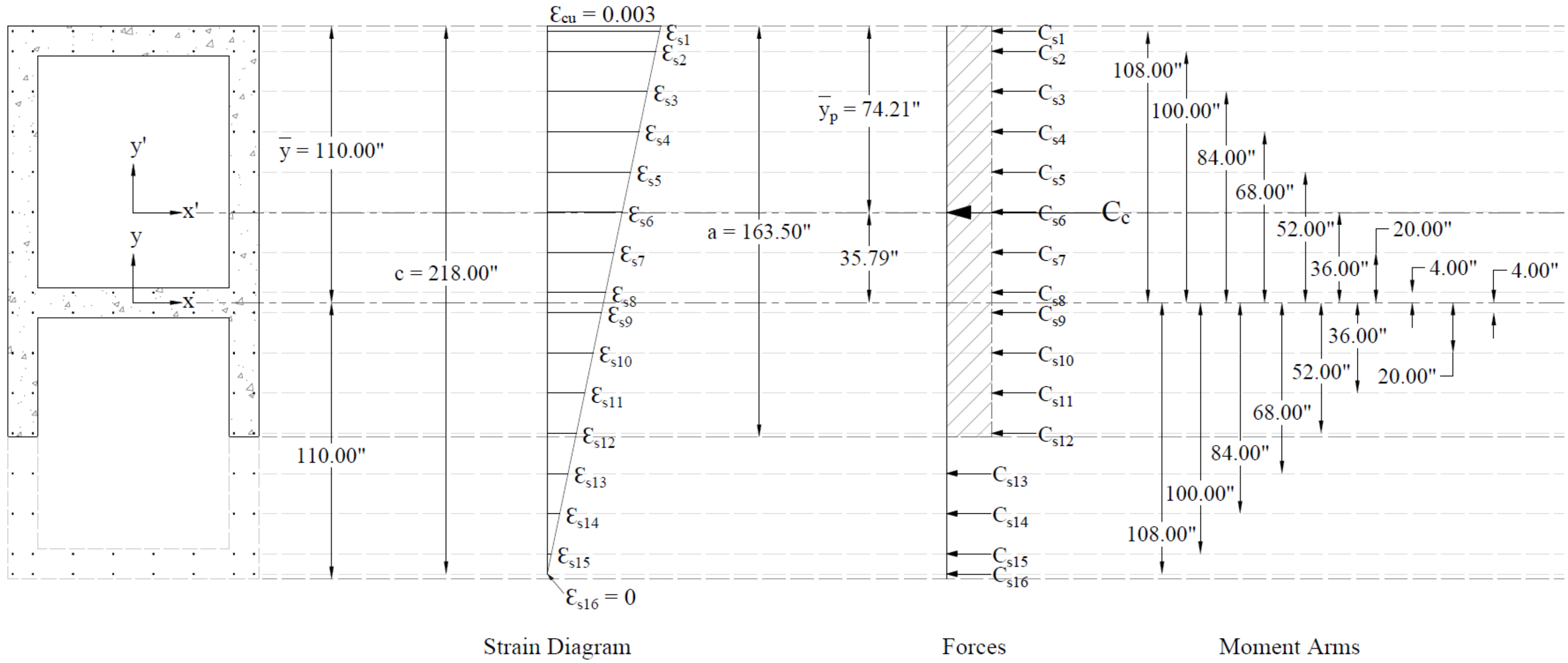


Figure 4 – Strain Diagram, Forces, and Moment Arms ($\varepsilon_t = f_s = 0$)

Strain ε_s is zero in the extreme layer of tension steel. This case is considered when calculating an interaction diagram because it marks the change from compression lap splices being allowed on all longitudinal bars, to the more severe requirement of tensile lap splices. ***ACI 318-14 (10.7.5.2.1 and 2)***

The following shows the general procedure to calculate the axial and moment capacities of the core wall section at this control point, all the calculated values are shown in the next Table.

2.1. c, *a*, and strains in the reinforcement

$$c = d_{16} = 218 \text{ in.}$$

Where *c* is the distance from the fiber of maximum compressive strain to the neutral axis.

ACI 318-14 (22.2.2.4.2)

$$a = \beta_1 \times c = 0.75 \times 218 = 163.5 \text{ in.}$$ ***ACI 318-14 (22.2.2.4.1)***

Where:

a = Depth of equivalent rectangular stress block

$$\beta_1 = 0.85 - \frac{0.05 \times \left(f_c^{'} - 4,000\right)}{1,000} = 0.85 - \frac{0.05 \times \left(6,000 - 4,000\right)}{1,000} = 0.75$$ ***ACI 318-14 (Table 22.2.2.4.3)***

$$\varepsilon_{s,16} = 0$$

$$\therefore \phi = 0.65$$ ***ACI 318-14 (Table 21.2.2)***

$$\varepsilon_{cu} = 0.003$$ ***ACI 318-14 (22.2.2.1)***

$$\varepsilon_{s,i} = \varepsilon_{cu}\left(\frac{d_i}{c} - 1\right)$$

$$\varepsilon_y = \frac{F_y}{E_s} = \frac{60}{29,000} = 0.00207$$

2.2. Forces in the concrete and steel

Since $h_1 + h_2 + h_3 + h_4 = 208$ in. > a = 163.5 in. > $h_1 + h_2 + h_3 = 116$ in., the area and centroid of the concrete equivalent block (see Figure 2 and 4) can be found as follows:

$$\begin{aligned} A_p &= A_1 + A_2 + A_3 + A_{4a} \\ &= \left(b_1 \times h_1\right) + \left(b_2 \times h_2\right) + \left(b_3 \times h_3\right) + \left(b_4 \times \left(a - \left(h_1 + h_2 + h_3\right)\right)\right) \\ &= \left(100 \times 12\right) + \left(\left(2 \times 12\right) \times 92\right) + \left(100 \times 12\right) + \left(\left(2 \times 12\right) \times \left(163.5 - \left(12 + 92 + 12\right)\right)\right) = 5,748 \text{ in.}^2 \end{aligned}$$

$$\bar{y}_p = \frac{A_1 \times d_1 + A_2 \times d_2 + A_3 \times d_3 + A_{4a} \times d_{4a}}{A_p}$$

Where:

$$A_1 \times d_1 = (b_1 \times h_1) \times \left(\frac{h_1}{2}\right) = (100 \times 12) \times \left(\frac{12}{2}\right) = 7,200 \text{ in.}^3$$

$$A_2 \times d_2 = (b_2 \times h_2) \times \left(h_1 + \frac{h_2}{2}\right) = ((2 \times 12) \times 92) \times \left(12 + \frac{92}{2}\right) = 128,064 \text{ in.}^3$$

$$A_3 \times d_3 = (b_3 \times h_3) \times \left(h_1 + h_2 + \frac{h_3}{2}\right) = (100 \times 12) \times \left(12 + 92 + \frac{12}{2}\right) = 132,000 \text{ in.}^3$$

$$A_{4a} \times d_{4a} = \left(b_4 \times \left(a - (h_1 + h_2 + h_3)\right)\right) \times \left(h_1 + h_2 + h_3 + \frac{\left(a - (h_1 + h_2 + h_3)\right)}{2}\right)$$

$$= \left((2 \times 12) \times \left(163.5 - (12 + 92 + 12)\right)\right) \times \left(12 + 92 + 12 + \frac{\left(163.5 - (12 + 92 + 12)\right)}{2}\right) = 159,315 \text{ in.}^3$$

$$\bar{y}_p = \frac{7,200 + 128,064 + 132,000 + 159,315}{5,748} = 74.21 \text{ in.}$$

$$C_c = 0.85 \times f_c^{'} \times A_p = 0.85 \times 6,000 \times 5,748 = 29,314.8 \text{ kip (compression)}$$ ***ACI 318-14 (22.2.2.4.1)***

$$\text{if} \begin{cases} \varepsilon_{s,i} \geq \varepsilon_y \rightarrow \text{reinforcement has yielded} \rightarrow f_{s,i} = f_y \\ \varepsilon_{s,i} < \varepsilon_y \rightarrow \text{reinforcement has not yielded} \rightarrow f_{s,i} = \varepsilon_{s,i} \times E_s \end{cases}$$

If the reinforcement layer is located within the depth of the equivalent rectangular stress block (a), it is necessary to subtract $0.85f_c'$ from $f_{s,i}$ before computing $F_{s,i}$ since the area of the reinforcement in this layer has been included in the area used to compute C_c.

$$if \begin{cases} d_i < a \rightarrow F_{s,i} = \left(f_{s,i} - 0.85 f_c^{'}\right) \times A_{s,i} \\ d_i > a \rightarrow F_{s,i} = f_{s,i} \times A_{s,i} \end{cases}$$

The force developed in the reinforcement layer ($F_{s,i}$) is considered as compression force ($C_{s,i}$) if the effective depth of this steel layer (d_i) is less than c (the distance from the fiber of maximum compressive strain to the neutral axis), otherwise it is considered as tension force ($T_{s,i}$).

2.3. ϕP_n and ϕM_n

Using values from the next Table:

$$P_n = C_c + \sum_{i=1}^{16} C_{s,i} - \sum_{i=1}^{16} T_{s,i} = -30,229.3 \text{ kip}$$

$$\phi P_n = 0.65 \times -30,229.3 = -19,649.0 \text{ kip}$$

$$M_n = C_c \times (\bar{y} - \bar{y}_p) + \sum_{i=1}^{16} C_{s,i} \times (\bar{y} - d_i) + \sum_{i=1}^{16} T_{s,i} \times (d_i - \bar{y}) = -90,728.74 \text{ kip-ft}$$

$$\phi M_n = 0.65 \times -90,728.74 = -58,973.68 \text{ kip-ft}$$

Table 3 - Axial and Moment Capacity for the Second Control Point								
Layer	A_s/bar, in^2	# of bars	d, in	ε_s, in./in.	$f_{s,i}$, ksi	$C_{s,i}$, kip	$T_{s,i}$, kip	$M_{n,i}$, kip-ft
1	0.31	8	2.0	-0.00297	60.0	-136.2	0.00	-1225.37
2	0.31	8	10.0	-0.00286	60.0	-136.2	0.00	-1134.60
3	0.31	4	26.0	-0.00264	60.0	-68.1	0.00	-476.53
4	0.31	4	42.0	-0.00242	60.0	-68.1	0.00	-385.76
5	0.31	4	58.0	-0.00220	60.0	-68.1	0.00	-295.00
6	0.31	4	74.0	-0.00198	57.5	-64.9	0.00	-194.81
7	0.31	4	90.0	-0.00176	51.1	-57.0	0.00	-95.03
8	0.31	8	106.0	-0.00154	44.7	-98.2	0.00	-32.73
9	0.31	8	114.0	-0.00143	51.5	-90.3	0.00	30.09
10	0.31	4	130.0	-0.00121	35.1	-37.2	0.00	62.03
11	0.31	4	146.0	-0.00099	28.7	-29.3	0.00	87.92
12	0.31	4	162.0	-0.00077	22.3	-21.4	0.00	92.68
13	0.31	4	178.0	-0.00055	16.0	-19.8	0.00	112.17
14	0.31	4	194.0	-0.00033	9.6	-11.9	0.00	83.14
15	0.31	8	210.0	-0.00011	3.2	-7.9	0.00	65.98
16	0.31	8	218.0	0.00000	0.0	0.0	0.00	0.00
Concrete	---	$\bar{y}_p$ =	74.21	---	---	-29314.8	0.00	-87422.93
					P_n, kip	-30229.3	M_n, kip-ft	-90728.74

3. Bar Stress Near Tension Face Equal to 0.5 f_y, (f_s = 0.5 f_y)

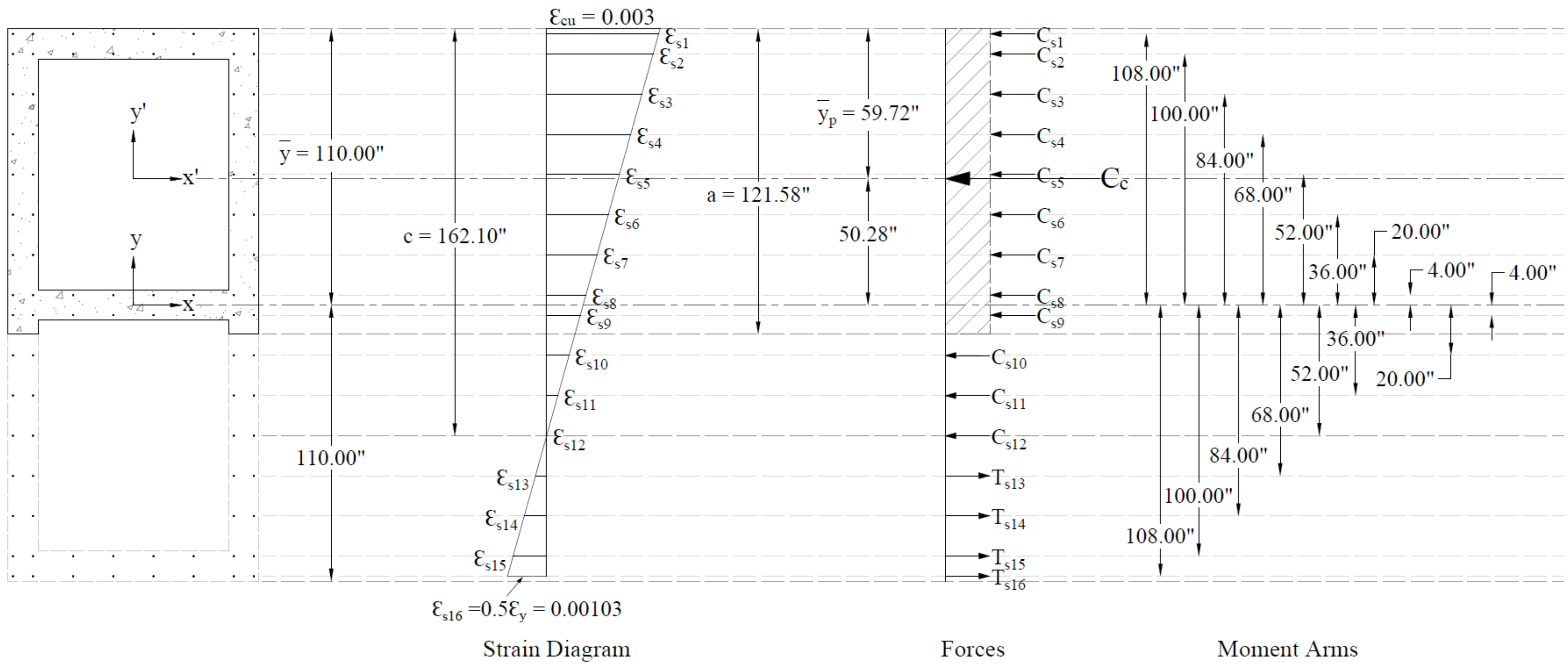


Figure 5 – Strains, Forces, and Moment Arms (f_s = 0.5 f_y)

The following show the general procedure to calculate the axial and moment capacities of the core wall section at this control point, all the calculated values are shown in the next Table.

3.1. c, *a*, and strains in the reinforcement

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29,000} = 0.00207$$

$$\varepsilon_{s,16} = \frac{\varepsilon_y}{2} = \frac{0.00207}{2} = 0.00103 < \varepsilon_y \rightarrow \text{tension reinforcement has not yielded}$$

$$\therefore \phi = 0.65$$ ***ACI 318-14 (Table 21.2.2)***

$$\varepsilon_{cu} = 0.003$$ ***ACI 318-14 (22.2.2.1)***

$$c = \frac{d_{16}}{\varepsilon_{s,16} + \varepsilon_{cu}} \times \varepsilon_{cu} = \frac{218}{0.00103 + 0.003} \times 0.003 = 162.10 \text{ in.}$$

Where c is the distance from the fiber of maximum compressive strain to the neutral axis.

ACI 318-14 (22.2.2.4.2)

$$a = \beta_1 \times c = 0.75 \times 162.10 = 121.58 \text{ in.}$$ ***ACI 318-14 (22.2.2.4.1)***

Where:

a = Depth of equivalent rectangular stress block

$$\beta_1 = 0.85 - \frac{0.05 \times \left(f'_c \times 4,000\right)}{1,000} = 0.85 - \frac{0.05 \times (6,000 - 4,000)}{1,000} = 0.75$$ ***ACI 318-14 (Table 22.2.2.4.3)***

$$\varepsilon_{s,i} = \varepsilon_{cu}\left(\frac{d_i}{c} - 1\right)$$

3.2. Forces in the concrete and steel

Since $h_1 + h_2 + h_3 + h_4 = 208$ in. > a = 121.58 in. > $h_1 + h_2 + h_3 = 116$ in., the area and centroid of the concrete equivalent block (see Figure 2 and 5) can be found as follows:

$$\begin{aligned} A_p &= A_1 + A_2 + A_3 + A_{4a} \\ &= (b_1 \times h_1) + (b_2 \times h_2) + (b_3 \times h_3) + \left(b_4 \times \left(a - (h_1 + h_2 + h_3)\right)\right) \\ &= (100 \times 12) + \left((2 \times 12) \times 92\right) + (100 \times 12) + \left((2 \times 12) \times \left(121.58 - (12 + 92 + 12)\right)\right) = 4,741.85 \text{ in.}^2 \end{aligned}$$

$$\bar{y}_p = \frac{A_1 \times d_1 + A_2 \times d_2 + A_3 \times d_3 + A_{4a} \times d_{4a}}{A_p}$$

Where:

$$A_1 \times d_1 = (b_1 \times h_1) \times \left(\frac{h_1}{2}\right) = (100 \times 12) \times \left(\frac{12}{2}\right) = 7,200 \text{ in.}^3$$

$$A_2 \times d_2 = (b_2 \times h_2) \times \left(h_1 + \frac{h_2}{2}\right) = ((2 \times 12) \times 92) \times \left(12 + \frac{92}{2}\right) = 128,064 \text{ in.}^3$$

$$A_3 \times d_3 = (b_3 \times h_3) \times \left(h_1 + h_2 + \frac{h_3}{2}\right) = (100 \times 12) \times \left(12 + 92 + \frac{12}{2}\right) = 132,000 \text{ in.}^3$$

$$A_{4a} \times d_{4a} = \left(b_4 \times \left(a - (h_1 + h_2 + h_3)\right)\right) \times \left(h_1 + h_2 + h_3 + \frac{\left(a - (h_1 + h_2 + h_3)\right)}{2}\right)$$

$$= \left((2 \times 12) \times \left(121.58 - (12 + 92 + 12)\right)\right) \times \left(12 + 92 + 12 + \frac{\left(121.58 - (12 + 92 + 12)\right)}{2}\right) = 15,899.38 \text{ in.}^3$$

$$\bar{y}_p = \frac{7,200 + 128,064 + 132,000 + 15,899.38}{4,741.85} = 59.72 \text{ in.}$$

$$C_c = 0.85 \times f_c^{'} \times A_p = 0.85 \times 6,000 \times 4,741.85 = 24{,}183.4 \text{ kip (compression)}$$ ***<u>ACI 318-14 (22.2.2.4.1)</u>***

$$\text{if} \begin{cases} \varepsilon_{s,i} \geq \varepsilon_y \rightarrow \text{reinforcement has yielded} \rightarrow f_{s,i} = f_y \\ \varepsilon_{s,i} < \varepsilon_y \rightarrow \text{reinforcement has not yielded} \rightarrow f_{s,i} = \varepsilon_{s,i} \times E_s \end{cases}$$

If the reinforcement layer is located within the depth of the equivalent rectangular stress block (a), it is necessary to subtract $0.85f_c'$ from $f_{s,i}$ before computing $F_{s,i}$ since the area of the reinforcement in this layer has been included in the area used to compute C_c.

$$if \begin{cases} d_i < a \rightarrow F_{s,i} = \left(f_{s,i} - 0.85 f_c^{'}\right) \times A_{s,i} \\ d_i > a \rightarrow F_{s,i} = f_{s,i} \times A_{s,i} \end{cases}$$

The force developed in the reinforcement layer ($F_{s,i}$) is considered as compression force ($C_{s,i}$) if the effective depth of this steel layer (d_i) is less than c (the distance from the fiber of maximum compressive strain to the neutral axis), otherwise it is considered as tension force ($T_{s,i}$).

3.3. ϕP_n and ϕM_n

Using values from the next Table:

$$P_n = C_c + \sum_{i=1}^{16} C_{s,i} - \sum_{i=1}^{16} T_{s,i} = -24,724.4 \text{ kip}$$

$$\phi P_n = 0.65 \times -24,724.4 = -16,070.9 \text{ kip}$$

$$M_n = C_c \times (\bar{y} - \bar{y}_p) + \sum_{i=1}^{16} C_{s,i} \times (\bar{y} - d_i) + \sum_{i=1}^{16} T_{s,i} \times (d_i - \bar{y}) = -106,403.22 \text{ kip-ft}$$

$$\phi M_n = 0.65 \times -106,403.22 = -69{,}162.09 \text{ kip-ft}$$

Table 4 - Axial and Moment Capacity for the Third Control Point								
Layer	A_s/bar, in^2	# of bars	d, in	ε_s, in./in.	$f_{s,i}$, ksi	$C_{s,i}$, kip	$T_{s,i}$, kip	$M_{n,i}$, kip-ft
1	0.31	8	2.0	-0.00296	60.0	-136.2	0.0	-1225.37
2	0.31	8	10.0	-0.00281	60.0	-136.2	0.0	-1134.60
3	0.31	4	26.0	-0.00252	60.0	-68.1	0.0	-476.53
4	0.31	4	42.0	-0.00222	60.0	-68.1	0.0	-385.76
5	0.31	4	58.0	-0.00193	55.9	-63.0	0.0	-272.81
6	0.31	4	74.0	-0.00163	47.3	-52.3	0.0	-156.93
7	0.31	4	90.0	-0.00133	38.7	-41.7	0.0	-69.44
8	0.31	8	106.0	-0.00104	30.1	-62.0	0.0	-20.68
9	0.31	8	114.0	-0.00089	25.8	-51.4	0.0	17.13
10	0.31	4	130.0	-0.00059	17.2	-21.4	0.0	35.61
11	0.31	4	146.0	-0.00030	8.6	-10.7	0.0	32.15
12	0.31	4	162.0	0.00000	0.1	-0.1	0.0	0.29
13	0.31	4	178.0	0.00029	8.5	0.0	10.6	-59.95
14	0.31	4	194.0	0.00059	17.1	0.0	21.2	-148.60
15	0.31	8	210.0	0.00089	25.7	0.0	63.8	-531.27
16	0.31	8	218.0	0.00103	30.0	0.0	74.4	-669.60
Concrete	---	$\bar{y}_p$ =	59.72	---	---	-24183.4	0.0	-101336.87
					P_n, kip	-24724.4	M_n, kip-ft	-106403.22

4. Bar Stress Near Tension Face Equal to f_y, ($f_s = f_y$)

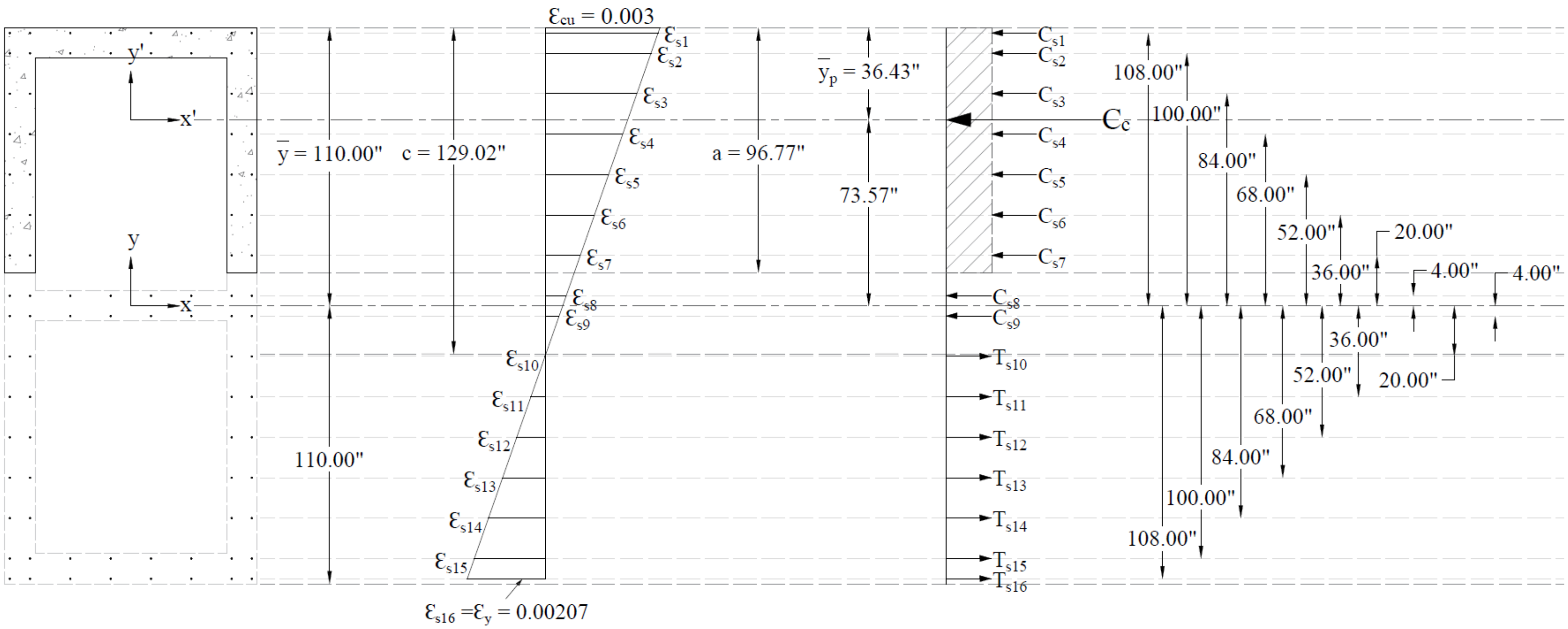


Figure 6 – Strains, Forces, and Moment Arms ($f_s = f_y$)

This strain distribution is called the balanced failure case and the compression-controlled strain limit. It marks the change from compression failures originating by crushing of the compression surface of the section, to tension failures initiated by yield of longitudinal reinforcement. It also marks the start of the transition zone for ϕ for columns and walls in which ϕ increases from 0.65 (or 0.75 for spiral columns) up to 0.90.

The following show the general procedure to calculate the axial and moment capacities of the core wall section at this control point, all the calculated values are shown in the next Table.

4.1. c, *a*, and strains in the reinforcement

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29,000} = 0.00207$$

$$\varepsilon_{s,16} = \varepsilon_y = 0.00207 \rightarrow \text{tension reinforcement has yielded}$$

$$\therefore \phi = 0.65$$ ***ACI 318-14 (Table 21.2.2)***

$$\varepsilon_{cu} = 0.003$$ ***ACI 318-14 (22.2.2.1)***

$$c = \frac{d_{16}}{\varepsilon_{s,16} + \varepsilon_{cu}} \times \varepsilon_{cu} = \frac{218}{0.00207 + 0.003} \times 0.003 = 129.02 \text{ in.}$$

Where *c* is the distance from the fiber of maximum compressive strain to the neutral axis.

ACI 318-14 (22.2.2.4.2)

$$a = \beta_1 \times c = 0.75 \times 129.02 = 96.77 \text{ in.}$$ ***ACI 318-14 (22.2.2.4.1)***

Where:

$$\beta_1 = 0.85 - \frac{0.05 \times \left(f_c^{'} \times 4000\right)}{1000} = 0.85 - \frac{0.05 \times (6,000 \times 4,000)}{1,000} = 0.75$$ ***ACI 318-14 (Table 22.2.2.4.3)***

$$\varepsilon_{s,i} = \varepsilon_{cu}\left(\frac{d_i}{c} - 1\right)$$

4.2. Forces in the concrete and steel

Since $h_1 + h_2 = 104$ in. > a = 96.77 in. > $h_1 = 12$ in., the area and centroid of the concrete equivalent block (see Figure 2 and 6) can be found as follows:

$$\begin{aligned} A_p &= A_1 + A_{2a} \\ &= (b_1 \times h_1) + (b_2 \times (a - h_1)) \\ &= (100 \times 12) + ((2 \times 12) \times (96.77 - 12)) = 3,234.37 \text{ in.}^2 \end{aligned}$$

$$\bar{y}_p = \frac{A_1 \times d_1 + A_{2a} \times d_{2a}}{A_p}$$

Where:

$$A_1 \times d_1 = (b_1 \times h_1) \times \left(\frac{h_1}{2}\right) = (100 \times 12) \times \left(\frac{12}{2}\right) = 7,200 \text{ in.}^3$$

$$\begin{aligned} A_{2a} \times d_{2a} &= \left(b_2 \times (a - h_1)\right) \times \left(h_1 + \frac{(a - h_1)}{2}\right) \\ &= \left((2 \times 12) \times (96.77 - 12)\right) \times \left(12 + \frac{(96.77 - 12)}{2}\right) = 110,634.29 \text{ in.}^3 \end{aligned}$$

$$\bar{y}_p = \frac{7,200 + 110,634.29}{3,234.37} = 36.43 \text{ in.}$$

$$C_c = 0.85 \times f_c^{'} \times A_p = 0.85 \times 6,000 \times 3,234.37 = 16,495.27 \text{ kip (compression)}$$ ***ACI 318-14 (22.2.2.4.1)***

$$\text{if} \begin{Bmatrix} \varepsilon_{s,i} \geq \varepsilon_y \rightarrow \text{reinforcement has yielded} \rightarrow f_{s,i} = f_y \\ \varepsilon_{s,i} < \varepsilon_y \rightarrow \text{reinforcement has not yielded} \rightarrow f_{s,i} = \varepsilon_{s,i} \times E_s \end{Bmatrix}$$

If the reinforcement layer is located within the depth of the equivalent rectangular stress block (a), it is necessary to subtract 0.85f_c' from $f_{s,i}$ before computing $F_{s,i}$ since the area of the reinforcement in this layer has been included in the area used to compute C_c.

$$if \begin{Bmatrix} d_i < a \rightarrow F_{s,i} = \left(f_{s,i} - 0.85 f_c^{'}\right) \times A_{s,i} \\ d_i > a \rightarrow F_{s,i} = f_{s,i} \times A_{s,i} \end{Bmatrix}$$

The force developed in the reinforcement layer ($F_{s,i}$) is considered as compression force ($C_{s,i}$) if the effective depth of this steel layer (d_i) is less than c (the distance from the fiber of maximum compressive strain to the neutral axis), otherwise it is considered as tension force ($T_{s,i}$).

4.3. ϕP_n and ϕM_n

Using values from the next Table:

$$P_n = C_c + \sum_{i=1}^{16} C_{s,i} - \sum_{i=1}^{16} T_{s,i} = -16,662.7 \text{ kip}$$

$$\phi P_n = 0.65 \times -16,662.7 = -10,830.7 \text{ kip}$$

$$M_n = C_c \times \left(\bar{y} - \bar{y}_p\right) + \sum_{i=1}^{16} C_{s,i} \times \left(\bar{y} - d_i\right) + \sum_{i=1}^{16} T_{s,i} \times \left(d_i - \bar{y}\right) = -107,980.91 \text{ kip-ft}$$

$$\phi M_n = 0.65 \times -107,980.91 = -70,187.59 \text{ kip-ft}$$

Table 5 - Axial and Moment Capacity for the Fourth Control Point								
Layer	A_s/bar, in^2	# of bars	d, in	ε_s, in./in.	$f_{s,i}$, ksi	$C_{s,i}$, kip	$T_{s,i}$, kip	$M_{n,i}$, kip-ft
1	0.31	8	2.0	-0.00295	60.0	-136.2	0.0	-1225.37
2	0.31	8	10.0	-0.00277	60.0	-136.2	0.0	-1134.60
3	0.31	4	26.0	-0.00240	60.0	-68.1	0.0	-476.53
4	0.31	4	42.0	-0.00202	58.7	-66.4	0.0	-376.48
5	0.31	4	58.0	-0.00165	47.9	-53.1	0.0	-229.92
6	0.31	4	74.0	-0.00128	37.1	-39.7	0.0	-119.04
7	0.31	4	90.0	-0.00091	26.3	-26.3	0.0	-43.84
8	0.31	8	106.0	-0.00054	15.5	-38.5	0.0	-12.83
9	0.31	8	114.0	-0.00035	10.1	-25.1	0.0	8.37
10	0.31	4	130.0	0.00002	0.7	0.0	0.8	-1.37
11	0.31	4	146.0	0.00039	11.5	0.0	14.2	-42.59
12	0.31	4	162.0	0.00077	22.2	0.0	27.6	-119.50
13	0.31	4	178.0	0.00114	33.0	0.0	41.0	-232.07
14	0.31	4	194.0	0.00151	43.8	0.0	54.3	-380.32
15	0.31	8	210.0	0.00188	54.6	0.0	135.4	-1128.52
16	0.31	8	218.0	0.00207	60.0	0.0	148.8	-1339.20
Concrete	---	$\bar{y}_p$ =	36.43	---	---	-16495.3	0.0	-101127.10
					P_n, kip	-16662.7	M_n, kip-ft	-107980.91

5. Bar Strain Near Tension Face Equal to 0.005 in./in., (ε_s = - 0.005 *in./in.*)

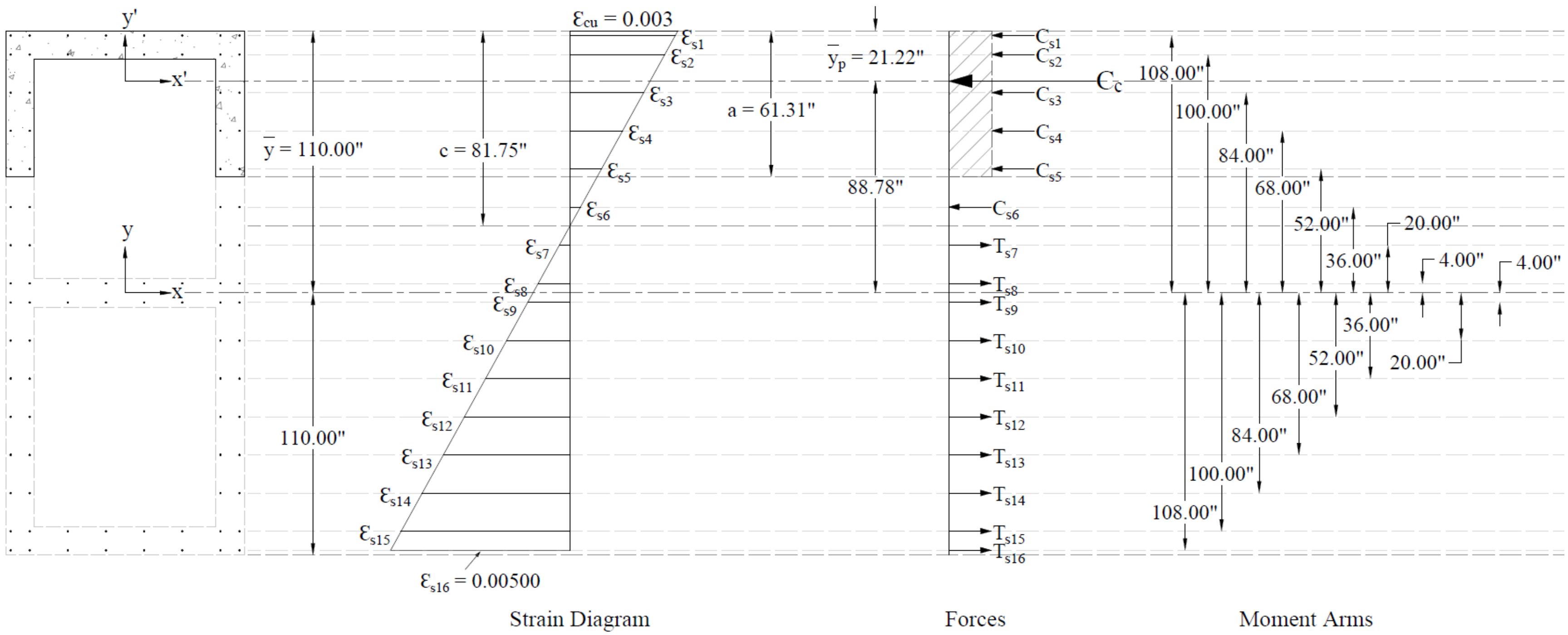


Figure 7 – Strains, Forces, and Moment Arms (ε_s = 0.005 in./in.)

This corresponds to the tension-controlled strain limit of 0.005. It is the strain at the tensile limit of the transition zone for ϕ, used to define a tension-controlled section.

The following show the general procedure to calculate the axial and moment capacities of the core wall section at this control point, all the calculated values are shown in the next Table.

5.1. c, *a*, and strains in the reinforcement

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29,000} = 0.00207$$

$$\varepsilon_{s,16} = 0.005 > \varepsilon_y \rightarrow \text{tension reinforcement has yielded}$$

$$\therefore \phi = 0.9$$ ***ACI 318-14 (Table 21.2.2)***

$$\varepsilon_{cu} = 0.003$$ ***ACI 318-14 (22.2.2.1)***

$$c = \frac{d_{16}}{\varepsilon_{s,16} + \varepsilon_{cu}} \times \varepsilon_{cu} = \frac{218}{0.005 + 0.003} \times 0.003 = 81.75 \text{ in.}$$

Where *c* is the distance from the fiber of maximum compressive strain to the neutral axis.

ACI 318-14 (22.2.2.4.2)

$$a = \beta_1 \times c = 0.75 \times 81.75 = 61.31 \text{ in.}$$ ***ACI 318-14 (22.2.2.4.1)***

Where:

$$\beta_1 = 0.85 - \frac{0.05 \times \left(f_c^{'} \times 4,000\right)}{1,000} = 0.85 - \frac{0.05 \times \left(6,000 - 4,000\right)}{1,000} = 0.75$$ ***ACI 318-14 (Table 22.2.2.4.3)***

$$\varepsilon_{s,i} = \varepsilon_{cu}\left(\frac{d_i}{c} - 1\right)$$

5.2. Forces in the concrete and steel

Since $h_1 + h_2 = 104$ in. > a = 61.31 in. > $h_1 = 12$ in., the area and centroid of the concrete equivalent block (see Figure 2 and 7) can be found as follows:

$$\begin{aligned} A_p &= A_1 + A_{2a} \\ &= \left(b_1 \times h_1\right) + \left(b_2 \times \left(a - h_1\right)\right) \\ &= \left(100 \times 12\right) + \left(\left(2 \times 12\right) \times \left(61.31 - 12\right)\right) = 2,383.5 \text{ in.}^2 \end{aligned}$$

$$\bar{y}_p = \frac{A_1 \times d_1 + A_{2a} \times d_{2a}}{A_p}$$

Where:

$$A_1 \times d_1 = (b_1 \times h_1) \times \left(\frac{h_1}{2}\right) = (100 \times 12) \times \left(\frac{12}{2}\right) = 7,200 \text{ in.}^3$$

$$A_{2a} \times d_{2a} = \left(b_2 \times (a - h_1)\right) \times \left(h_1 + \frac{(a - h_1)}{2}\right)$$

$$= \left((2 \times 12) \times (61.31 - 12)\right) \times \left(12 + \frac{(61.31 - 12)}{2}\right) = 43,382.67 \text{ in.}^3$$

$$\bar{y}_p = \frac{7,200 + 43,382.67}{2,383.5} = 21.22 \text{ in.}$$

$$C_c = 0.85 \times f_c' \times A_p = 0.85 \times 6,000 \times 2,383.5 = 12,155.85 \text{ kip (compression)}$$ ***ACI 318-14 (22.2.2.4.1)***

$$\text{if} \begin{Bmatrix} \varepsilon_{s,i} \geq \varepsilon_y \rightarrow \text{reinforcement has yielded} \rightarrow f_{s,i} = f_y \\ \varepsilon_{s,i} < \varepsilon_y \rightarrow \text{reinforcement has not yielded} \rightarrow f_{s,i} = \varepsilon_{s,i} \times E_s \end{Bmatrix}$$

If the reinforcement layer is located within the depth of the equivalent rectangular stress block (a), it is necessary to subtract 0.85f_c' from $f_{s,i}$ before computing $F_{s,i}$ since the area of the reinforcement in this layer has been included in the area used to compute C_c.

$$if \begin{Bmatrix} d_i < a \rightarrow F_{s,i} = \left(f_{s,i} - 0.85 f_c'\right) \times A_{s,i} \\ d_i > a \rightarrow F_{s,i} = f_{s,i} \times A_{s,i} \end{Bmatrix}$$

The force developed in the reinforcement layer ($F_{s,i}$) is considered as compression force ($C_{s,i}$) if the effective depth of this steel layer (d_i) is less than c (the distance from the fiber of maximum compressive strain to the neutral axis), otherwise it is considered as tension force ($T_{s,i}$).

5.3. ϕP_n and ϕM_n

Using values from the next Table:

$$P_n = C_c + \sum_{i=1}^{16} C_{s,i} - \sum_{i=1}^{16} T_{s,i} = -11,757.9 \text{ kip}$$

$$\phi P_n = 0.9 \times -11,757.9 = -10,582.1 \text{kip}$$

$$M_n = C_c \times \left(\bar{y} - \bar{y}_p\right) + \sum_{i=1}^{16} C_{s,i} \times \left(\bar{y} - d_i\right) + \sum_{i=1}^{16} T_{s,i} \times \left(d_i - \bar{y}\right) = -97,324.39 \text{ kip-ft}$$

$$\phi M_n = 0.9 \times -97,324.39 = -87,591.95 \text{kip-ft}$$

Table 6 - Axial and Moment Capacity for the Fifth Control Point								
Layer	A_s/bar, in^2	# of bars	d, in	ε_s, in./in.	$f_{s,i}$, ksi	$C_{s,i}$, kip	$T_{s,i}$, kip	$M_{n,i}$, kip-ft
1	0.31	8	2.0	-0.00293	60.00	-136.2	0.0	-1225.37
2	0.31	8	10.0	-0.00263	60.00	-136.2	0.0	-1134.60
3	0.31	4	26.0	-0.00205	59.33	-67.2	0.0	-470.72
4	0.31	4	42.0	-0.00146	42.30	-46.1	0.0	-261.41
5	0.31	4	58.0	-0.00087	25.28	-25.0	0.0	-108.41
6	0.31	4	74.0	-0.00028	8.25	-10.2	0.0	-30.68
7	0.31	4	90.0	0.00030	8.78	0.0	10.9	18.15
8	0.31	8	106.0	0.00089	25.81	0.0	64.0	21.33
9	0.31	8	114.0	0.00118	34.32	0.0	85.1	-28.37
10	0.31	4	130.0	0.00177	51.35	0.0	63.7	-106.12
11	0.31	4	146.0	0.00236	60.00	0.0	74.4	-223.20
12	0.31	4	162.0	0.00294	60.00	0.0	74.4	-322.40
13	0.31	4	178.0	0.00353	60.00	0.0	74.4	-421.60
14	0.31	4	194.0	0.00412	60.00	0.0	74.4	-520.80
15	0.31	8	210.0	0.00471	60.00	0.0	148.8	-1240.00
16	0.31	8	218.0	0.00500	60.00	0.0	148.8	-1339.20
Concrete	---	$\bar{y}_p$ =	21.22	---	---	-12155.9	0.0	-89930.99
					P_n, kip	-11757.9	M_n, kip-ft	-97324.39

6. Pure Bending

This corresponds to the case where the nominal axial load capacity, P_n, is equal to zero. The following show the general iterative procedure to calculate the moment capacity of the core wall section at this control point, all the calculated values are shown in the next Table.

6.1. c, *a*, and strains in the reinforcement

Try $c = 3.67$ in.

Where c is the distance from the fiber of maximum compressive strain to the neutral axis.

ACI 318-14 (22.2.2.4.2)

$$a = \beta_1 \times c = 0.85 \times 3.67 = 2.75 \text{ in.}$$ ***ACI 318-14 (22.2.2.4.1)***

Where:

$$\beta_1 = 0.85 - \frac{0.05 \times (f_c^{'} \times 4,000)}{1,000} = 0.85 - \frac{0.05 \times (6,000 - 4,000)}{1,000} = 0.75$$ ***ACI 318-14 (Table 22.2.2.4.3)***

$$\varepsilon_{cu} = 0.003$$ ***ACI 318-14 (22.2.2.1)***

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29,000} = 0.00207$$

$$\varepsilon_{s,16} = 0.003 \times \left(\frac{d_{16}}{c} - 1\right) = 0.003 \times \left(\frac{218}{3.67} - 1\right) = 0.17535 \text{ (Tension)} > \varepsilon_y \rightarrow \text{tension reinforcement has yielded}$$

$$\therefore \phi = 0.9$$ ***ACI 318-14 (Table 21.2.2)***

$$\varepsilon_{s,i} = \varepsilon_{cu}\left(\frac{d_i}{c} - 1\right)$$

6.2. Forces in the concrete and steel

Since a = 2.75 in. < h_1 = 12 in., the area and centroid of the concrete equivalent block can be found as follows:

$$A_p = a \times b_1 = 2.75 \times 100 = 275 \text{ in.}^2$$

$$\bar{y}_p = \frac{a}{2} = \frac{2.75}{2} = 1.375 \text{ in.}$$

$$C_c = 0.85 \times f_c^{'} \times A_p = 0.85 \times 6,000 \times 275 = 1402.57 \text{ kip (compression)}$$ ***ACI 318-14 (22.2.2.4.1)***

$$\text{if} \begin{Bmatrix} \varepsilon_{s,i} \geq \varepsilon_y \rightarrow \text{reinforcement has yielded} \rightarrow f_{s,i} = f_y \\ \varepsilon_{s,i} < \varepsilon_y \rightarrow \text{reinforcement has not yielded} \rightarrow f_{s,i} = \varepsilon_{s,i} \times E_s \end{Bmatrix}$$

If the reinforcement layer is located within the depth of the equivalent rectangular stress block (a), it is necessary to subtract $0.85f_c'$ from $f_{s,i}$ before computing $F_{s,i}$ since the area of the reinforcement in this layer has been included in the area used to compute C_c.

$$if \begin{Bmatrix} d_i < a \rightarrow F_{s,i} = \left(f_{s,i} - 0.85 f_c^{'} \right) \times A_{s,i} \\ d_i > a \rightarrow F_{s,i} = f_{s,i} \times A_{s,i} \end{Bmatrix}$$

The force developed in the reinforcement layer ($F_{s,i}$) is considered as compression force ($C_{s,i}$) if the effective depth of this steel layer (d_i) is less than c (the distance from the fiber of maximum compressive strain to the neutral axis), otherwise it is considered as tension force ($T_{s,i}$).

6.3. ϕP_n and ϕM_n

Using values from the next Table:

$$P_n = C_c + \sum_{i=1}^{16} C_{s,i} - \sum_{i=1}^{16} T_{s,i} \simeq 0 \text{ kip}$$

The assumption that c = 3.67 in. is correct

$$M_n = C_c \times \left(\bar{y} - \bar{y}_p \right) + \sum_{i=1}^{16} C_{s,i} \times \left(\bar{y} - d_i \right) + \sum_{i=1}^{16} T_{s,i} \times \left(d_i - \bar{y} \right) = -14{,}804.25 \text{ kip-ft}$$

$$\phi M_n = 0.9 \times -14{,}804.25 = -13{,}323.82 \text{ kip-ft}$$

Table 7 - Axial and Moment Capacity for the Sixth Control Point								
Layer	A_s/bar, in^2	# of bars	d, in	ε_s, in./in.	$f_{s,i}$, ksi	$C_{s,i}$, kip	$T_{s,i}$, kip	$M_{n,i}$, kip-ft
1	0.31	8	2.0	-0.00136	39.55	-85.4	0.0	-768.88
2	0.31	8	10.0	0.00518	60.00	0.0	148.8	1240.00
3	0.31	4	26.0	0.01827	60.00	0.0	74.4	520.80
4	0.31	4	42.0	0.03136	60.00	0.0	74.4	421.60
5	0.31	4	58.0	0.04445	60.00	0.0	74.4	322.40
6	0.31	4	74.0	0.05754	60.00	0.0	74.4	223.20
7	0.31	4	90.0	0.07063	60.00	0.0	74.4	124.00
8	0.31	8	106.0	0.08372	60.00	0.0	148.8	49.60
9	0.31	8	114.0	0.09027	60.00	0.0	148.8	-49.60
10	0.31	4	130.0	0.10336	60.00	0.0	74.4	-124.00
11	0.31	4	146.0	0.11645	60.00	0.0	74.4	-223.20
12	0.31	4	162.0	0.12954	60.00	0.0	74.4	-322.40
13	0.31	4	178.0	0.14263	60.00	0.0	74.4	-421.60
14	0.31	4	194.0	0.15572	60.00	0.0	74.4	-520.80
15	0.31	8	210.0	0.16881	60.00	0.0	148.8	-1240.00
16	0.31	8	218.0	0.17535	60.00	0.0	148.8	-1339.20
Concrete	---	$\bar{y}_p$ =	1.38	---	---	-1402.6	0.0	-12696.17
					P_n, kip	0.0	M_n, kip-ft	-14804.25

7. Maximum Tension

The final loading case to be considered is concentric axial tension. The strength under maximum axial tension is computed by assuming that the section is completely cracked through and subjected to a uniform strain greater than or equal to the yield strain in tension. The axial tensile strength under such a loading is equal to the yield strength of the reinforcement in tension.

7.1. P_{nt} and ϕP_{nt}

$$P_{nt} = f_y \times A_{st} = 60,000 \times 27.28 = 1,636.8 \text{ kip}$$ ***ACI 318-14 (22.4.3.1)***

Where:

$$A_{st} = \# \text{ of bars} \times A_{s/bar} = 55 \times 0.31 = 27.28 \text{ in.}^2$$

$$\phi = 0.9$$ ***ACI 318-14 (Table 21.2.2)***

$$\phi P_{nt} = 0.90 \times 1,636.8 = 1,473.1 \text{kip}$$

7.2. M_n and ϕM_n

Since the section is regular about the x-axis, the moment capacity associated with the maximum axial tensile strength is equal to zero.

$$M_n = 0.00 \text{ kip-ft}$$

$$\phi M_n = 0.9 \times 0.00 = 0.00 \text{kip-ft}$$

As a summary, the following table shows the values for the control points necessary to create the interaction diagram for the core wall investigated in this example (when the moment is applied about the positive x-axis):

Table 8 - Control Points (Moment Applied about the X-Axis)

Control Point	ϕP_n, kip	ϕM_n, kip-ft	c, in	$\varepsilon_{s,16}$, in.in.	ϕ
Maximum Compression	27,546.5	0.00	---	---	0.65
Allowable Compression	22,037.2	---	---	---	0.65
$f_s = 0.0$	19,649.0	58,973.68	218.00	0.00000	0.65
$f_s = 0.5f_y$	16,070.9	69,162.09	162.10	0.00103	0.65
Balanced Point	10,830.7	70,187.59	129.02	0.00207	0.65
Tension Control	10,582.1	87,591.95	81.75	0.00500	0.90
Pure Bending	0.0	13,323.82	3.67	0.17535	0.90
Maximum Tension	1,473.1	0.00	---	---	0.90

8. Core Wall Interaction Diagram - spColumn Software

spColumn is a StructurePoint software program that performs the strength analysis of reinforced concrete sections conforming to the provisions of the Strength Design Method and Unified Design Provisions with all conditions of strength satisfying the applicable conditions of equilibrium and strain compatibility. For this core wall section, investigation mode was used with no loads (the program will only report control points) using ACI 318-14. The model editor in spColumn was used to model the section including multiple openings for elevator banks, place the steel reinforcing bars, and define the concrete cover. These steps illustrate handling of irregular shapes and unusual and/or complicated bar arrangements often found in building shear and core walls.

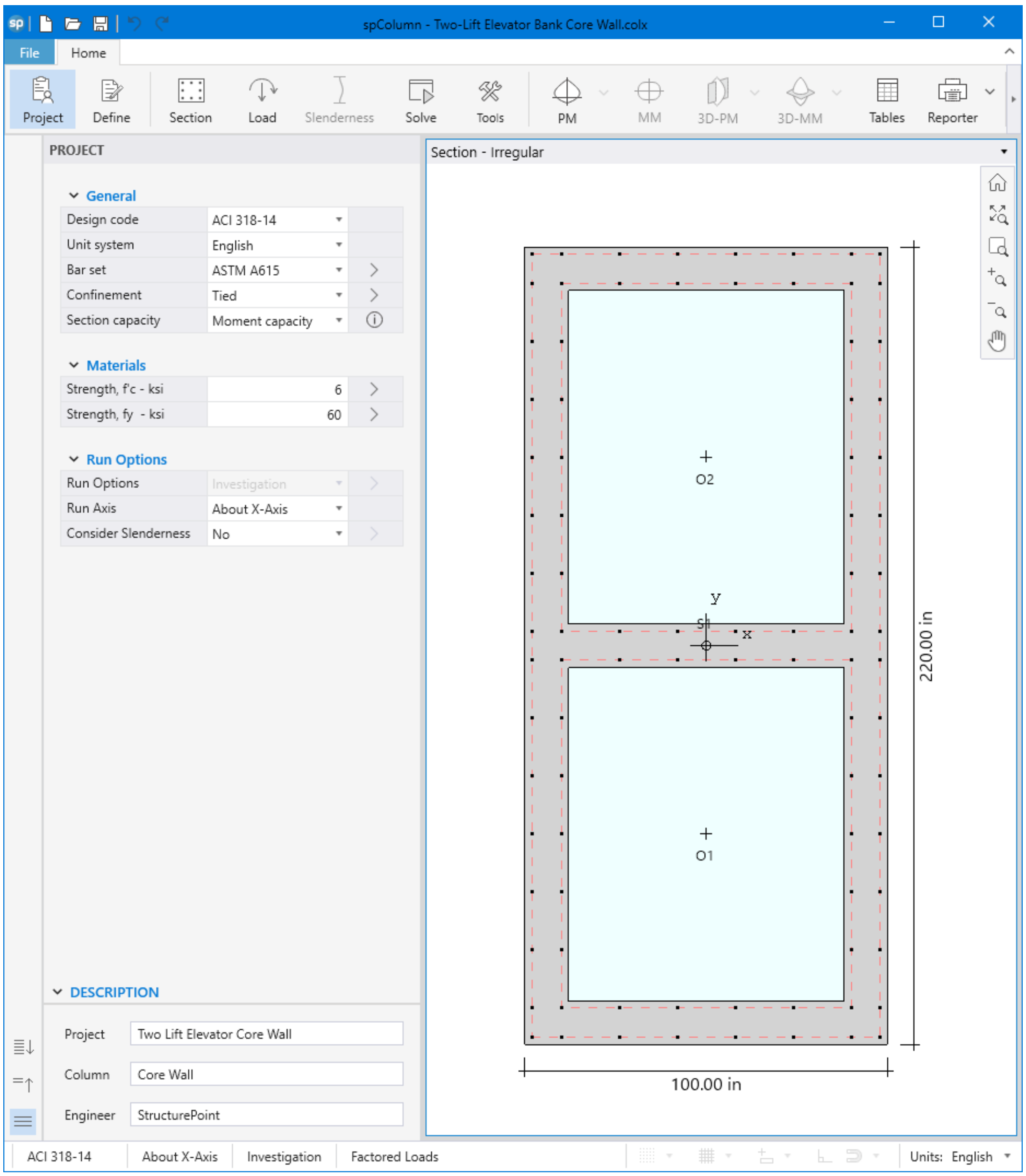


Figure 8 – Core Wall spColumn Modeling

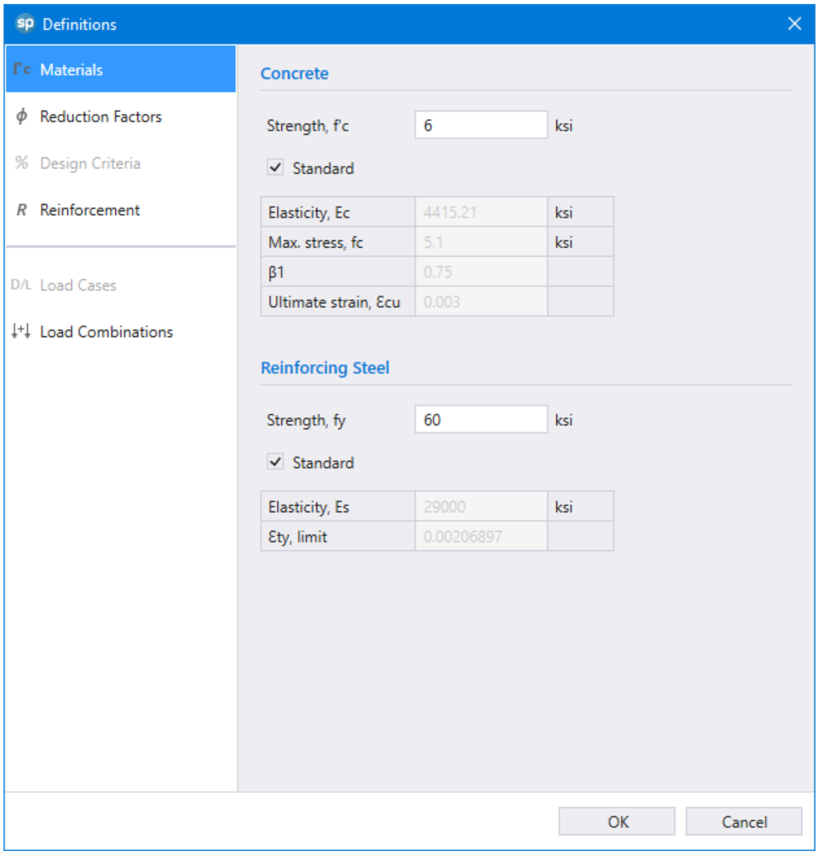


Figure 9 – Defining Material Properties - spColumn

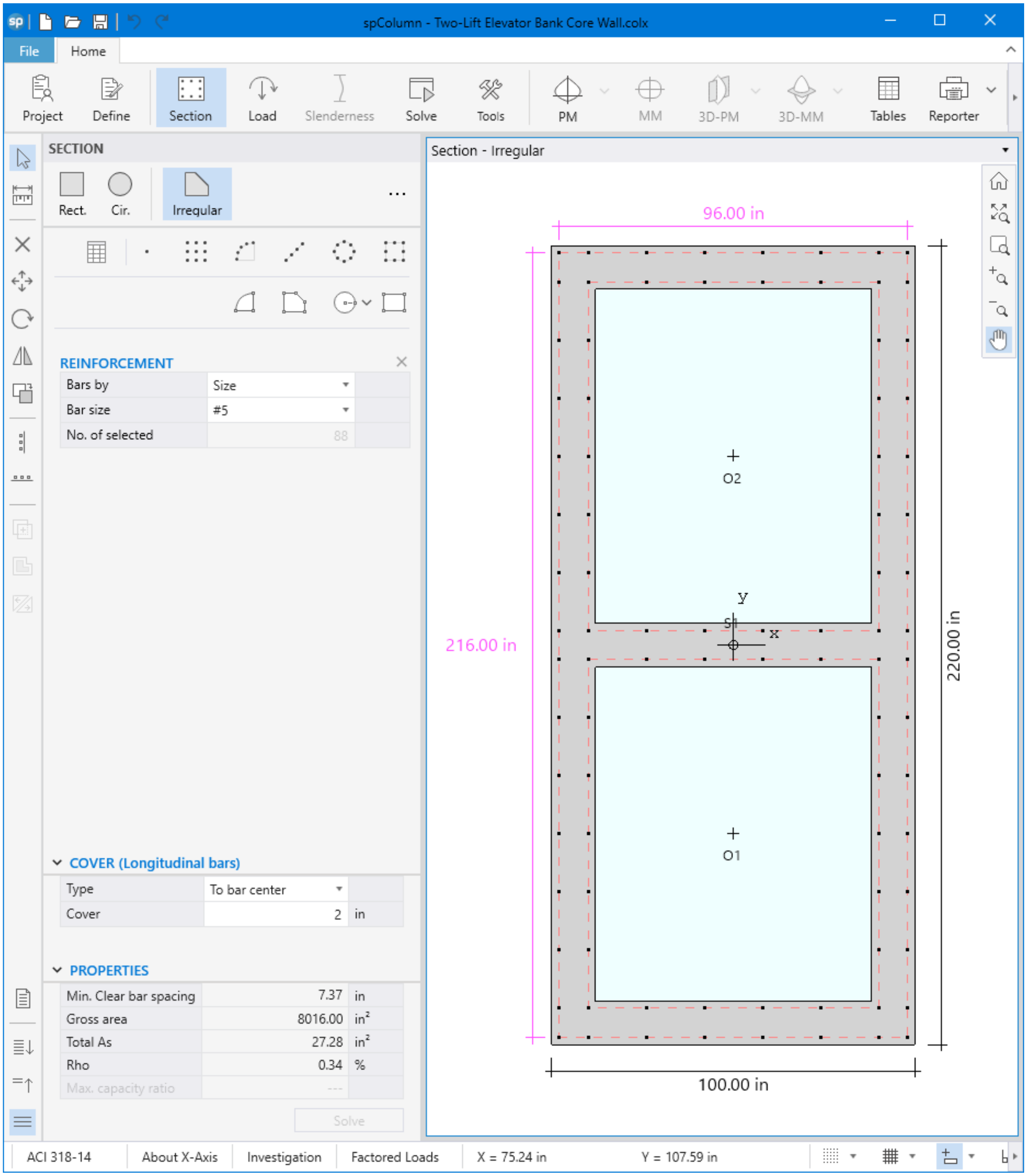


Figure 10 – Core Wall in spColumn Model Editor

Alternatively, the section, openings, and reinforcement arrangement can be imported to spColumn as an AutoCad file (.dxf). The following figure shows the section being imported to spColumn directly from AutoCad.

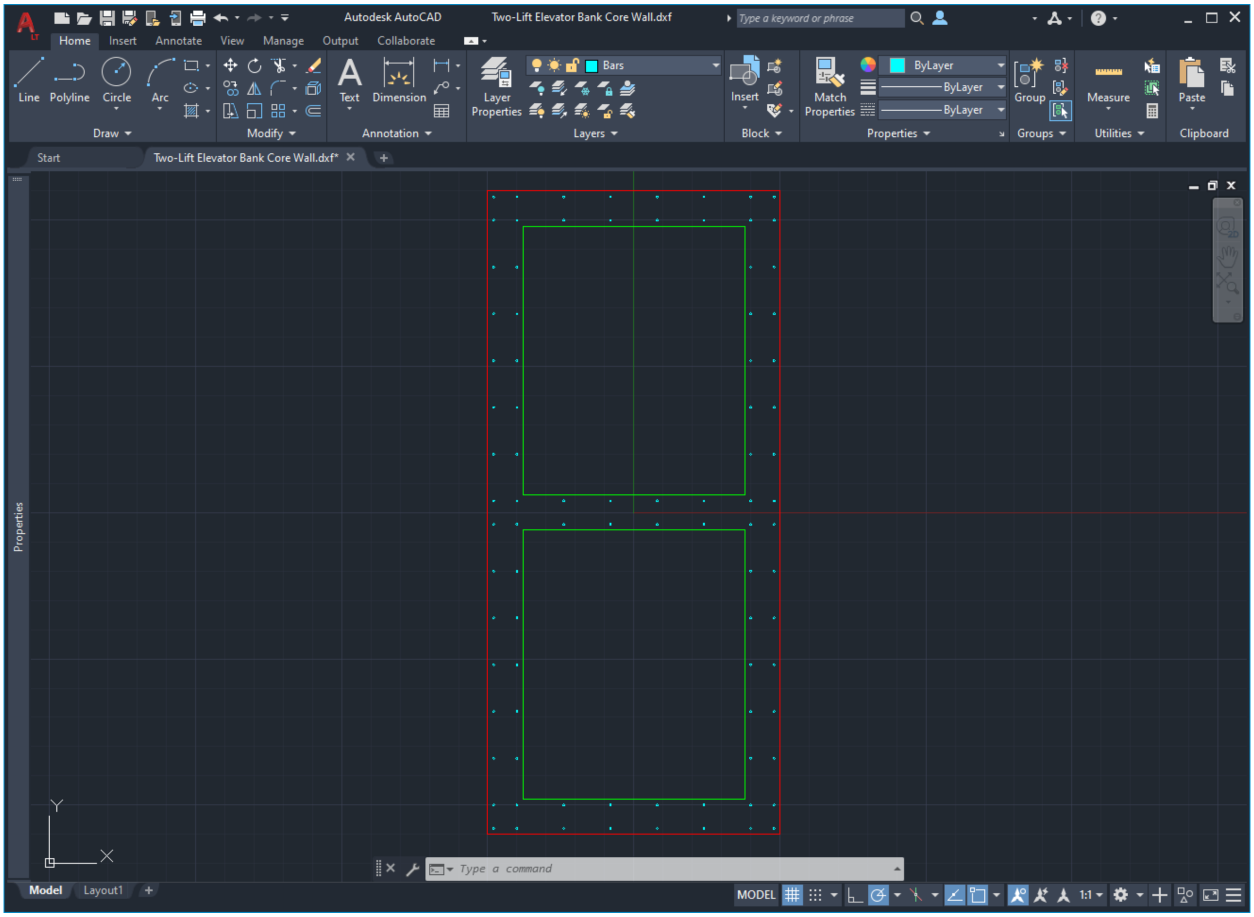


Figure 11 – Core wall Section Using AutoCad (.dxf file)

The following shows the P-M interaction diagram and input/output report generated by spColumn for the core wall.

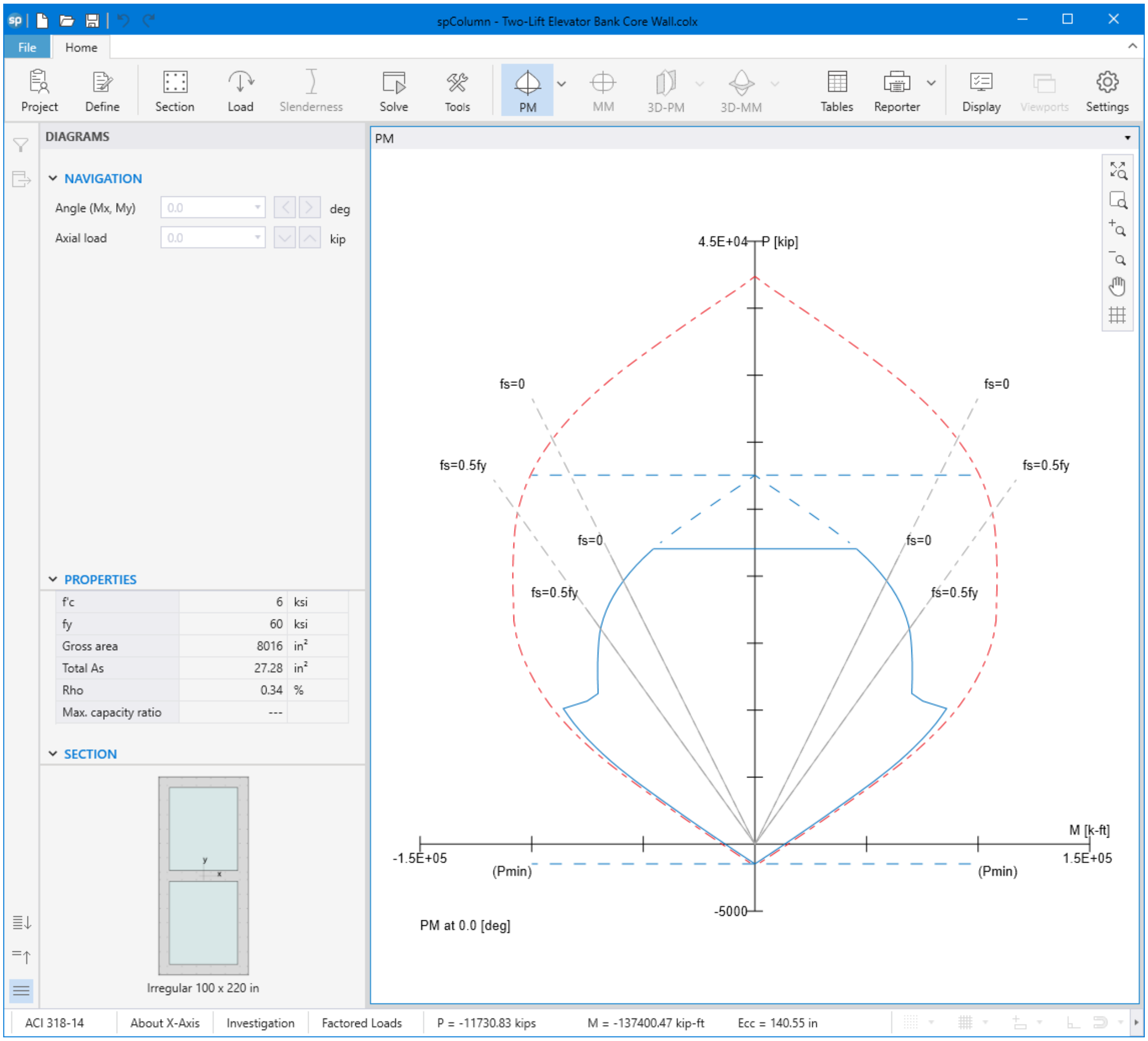
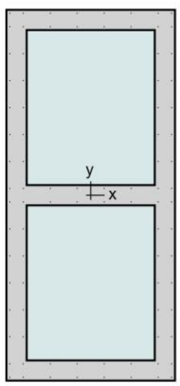


Figure 12 – Core Wall P-M Interaction Diagram about the X-Axis (spColumn)

spColumn v10.00 (TM) - Beta 1
Computer program for the Strength Design of Reinforced Concrete Sections



STRUCTUREPOINT - spColumn v10.00 (TM) - Beta 1
Licensed to: StructurePoint, LLC. License ID: 00000-0000000-4-23D9E-23D9E
C:\StructurePoint\Two-Lift Elevator Bank Core Wall.colx

Page | 2
7/29/2021
12:25 PM

Contents

List of Figures

1. General Information

File Name	C:\Struc...\Two-Lift Elevator Bank Core Wall.colx
Project	Two Lift Elevator Core Wall
Column	Core Wall
Engineer	StructurePoint
Code	ACI 318-14
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - 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	60	ksi
E_s	29000	ksi
ε_{yt}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	8016	in²
I_x	4.10572e+007	in⁴
I_y	1.16024e+007	in⁴
r_x	71.5675	in
r_y	38.0447	in
X_o	0	in
Y_o	0	in

3.2. Section Figure

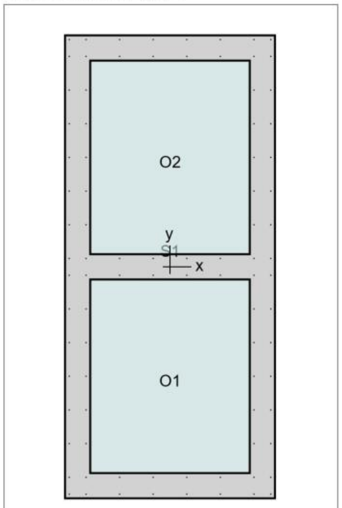


Figure 1: Column section

3.3. Solids

3.3.1. S1

Points	X	Y	Points	X	Y	Points	X	Y
	in	in		in	in		in	in
1	-50.0	-110.0	2	50.0	-110.0	3	50.0	110.0
4	-50.0	110.0						

3.4. Openings

3.4.1. O1

Points	X	Y	Points	X	Y	Points	X	Y
	in	in		in	in		in	in
1	-38.0	-98.0	2	38.0	-98.0	3	38.0	-6.0
4	-38.0	-6.0						

3.4.2. O2

Points	X	Y	Points	X	Y	Points	X	Y
	in	in		in	in		in	in
1	-38.0	6.0	2	38.0	6.0	3	38.0	98.0
4	-38.0	98.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in²		in	in²		in	in²
#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

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in²		in	in²		in	in²
#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	27.28 in²
Rho	0.34 %
Minimum clear spacing	7.37 in

(Note: Rho < 0.50%)

4.4. Bars Provided

Area	X	Y	Area	X	Y	Area	X	Y
in²	in	in	in²	in	in	in²	in	in
0.31	-48.0	-108.0	0.31	-40.0	-108.0	0.31	-48.0	-100.0
0.31	-40.0	-100.0	0.31	-24.0	-100.0	0.31	-24.0	-108.0
0.31	-8.0	-100.0	0.31	-8.0	-108.0	0.31	8.0	-100.0
0.31	8.0	-108.0	0.31	24.0	-100.0	0.31	24.0	-108.0
0.31	40.0	-100.0	0.31	40.0	-108.0	0.31	48.0	-108.0
0.31	48.0	-100.0	0.31	-48.0	-84.0	0.31	-40.0	-84.0
0.31	-48.0	-68.0	0.31	-40.0	-68.0	0.31	-48.0	-52.0
0.31	-40.0	-52.0	0.31	-48.0	-36.0	0.31	-40.0	-36.0
0.31	-48.0	-20.0	0.31	-40.0	-20.0	0.31	40.0	-84.0
0.31	48.0	-84.0	0.31	40.0	-68.0	0.31	48.0	-68.0
0.31	40.0	-52.0	0.31	48.0	-52.0	0.31	40.0	-36.0
0.31	48.0	-36.0	0.31	40.0	-20.0	0.31	48.0	-20.0
0.31	-48.0	108.0	0.31	-40.0	108.0	0.31	-48.0	100.0
0.31	-40.0	100.0	0.31	-24.0	100.0	0.31	-24.0	108.0
0.31	-8.0	100.0	0.31	-8.0	108.0	0.31	8.0	100.0
0.31	8.0	108.0	0.31	24.0	100.0	0.31	24.0	108.0
0.31	40.0	100.0	0.31	40.0	108.0	0.31	48.0	108.0
0.31	48.0	100.0	0.31	-48.0	84.0	0.31	-40.0	84.0
0.31	-48.0	68.0	0.31	-40.0	68.0	0.31	-48.0	52.0
0.31	-40.0	52.0	0.31	-48.0	36.0	0.31	-40.0	36.0
0.31	-48.0	20.0	0.31	-40.0	20.0	0.31	-48.0	4.0
0.31	-40.0	4.0	0.31	-48.0	-4.0	0.31	-40.0	-4.0

Area	X	Y	Area	X	Y	Area	X	Y
in^2	in	in	in^2	in	in	in^2	in	in
0.31	40.0	84.0	0.31	48.0	84.0	0.31	40.0	68.0
0.31	48.0	68.0	0.31	40.0	52.0	0.31	48.0	52.0
0.31	40.0	36.0	0.31	48.0	36.0	0.31	40.0	20.0
0.31	48.0	20.0	0.31	48.0	4.0	0.31	40.0	4.0
0.31	48.0	-4.0	0.31	40.0	-4.0	0.31	-24.0	-4.0
0.31	-24.0	4.0	0.31	-8.0	-4.0	0.31	-8.0	4.0
0.31	8.0	-4.0	0.31	8.0	4.0	0.31	24.0	-4.0
0.31	24.0	4.0						

5. Control Points

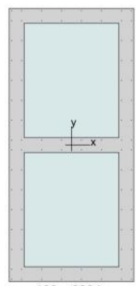
About	Point	P	X-Moment	Y-Moment	NA Depth	d_t Depth	ε_t	ϕ
		kip	k-ft	k-ft	in	in		
X	@ Max compression	27546.5	0.00	0.00	702.44	218.00	-0.00207	0.65000
X	@ Allowable comp.	22037.2	45554.40	0.01	256.29	218.00	-0.00045	0.65000
X	@ f_s = 0.0	19649.0	58973.67	-0.01	218.00	218.00	0.00000	0.65000
X	@ f_s = 0.5 f_y	16070.9	69161.98	-0.01	162.10	218.00	0.00103	0.65000
X	@ Balanced point	10830.7	70187.57	-0.01	129.02	218.00	0.00207	0.65000
X	@ Tension control	10582.1	87591.84	0.01	81.75	218.00	0.00500	0.90000
X	@ Pure bending	0.0	13323.82	0.00	3.67	218.00	0.17535	0.90000
X	@ Max tension	-1473.1	0.00	0.00	0.00	218.00	9.99999	0.90000
-X	@ Max compression	27546.5	0.01	0.00	702.44	218.00	-0.00207	0.65000
-X	@ Allowable comp.	22037.2	-45554.43	-0.02	256.29	218.00	-0.00045	0.65000
-X	@ f_s = 0.0	19649.0	-58973.66	0.01	218.00	218.00	0.00000	0.65000
-X	@ f_s = 0.5 f_y	16070.9	-69161.98	-0.01	162.10	218.00	0.00103	0.65000
-X	@ Balanced point	10830.7	-70187.57	0.01	129.02	218.00	0.00207	0.65000
-X	@ Tension control	10582.1	-87591.84	0.00	81.75	218.00	0.00500	0.90000
-X	@ Pure bending	0.0	-13323.82	-0.01	3.67	218.00	0.17535	0.90000
-X	@ Max tension	-1473.1	0.00	0.00	0.00	218.00	9.99999	0.90000

STRUCTUREPOINT - spColumn v10.00 (TM) - Beta 1
Licensed to: StructurePoint, LLC. License ID: 00000-0000000-4-23D9E-23D9E
C:\StructurePoint\Two-Lift Elevator Bank Core Wall.colx

Page | 7
7/29/2021
12:25 PM

6. Diagrams

6.1. PM at θ=0 [deg]



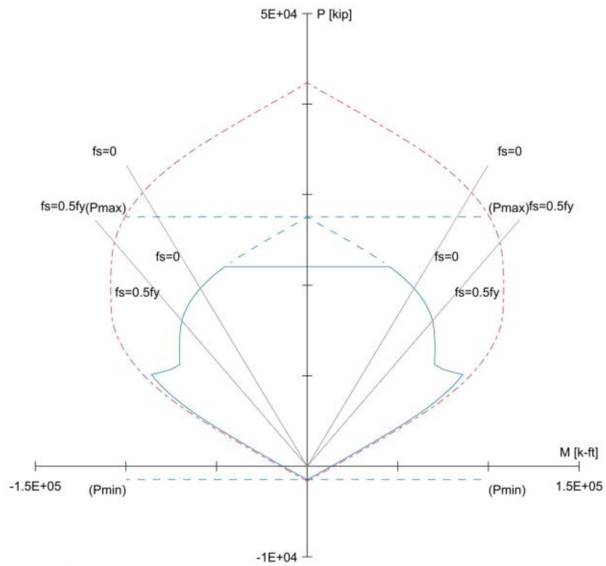
100 x 220 in

General Information	
Project	Two Lift El...or Core Wall
Column	Core Wall
Engineer	StructurePoint
Code	ACI 318-14
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	6 ksi
E_c	4415.21 ksi
f_y	60 ksi
E_s	29000 ksi

Section	
Type	Irregular
A_g	8016 in²
I_x	4.10572e+007 in⁴
I_y	1.16024e+007 in⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	27.28 in²
Rho	0.34 %
Min. clear spacing	7.37 in



PM at 0.0 [deg]

9. Summary and Comparison of Design Results

Table 9 - Comparison of Results (Moment about X-Axis)				
Support	ϕP_n, kip		ϕM_n, kip-ft	
	Hand	spColumn	Hand	spColumn
Max compression	27,546.5	27,546.5	0.00	0.00
Allowable compression	22,037.2	22,037.2	---	---
f_s = 0.0	19,649.0	19,649.0	58,973.68	58,973.67
f_s = 0.5 f_y	16,070.9	16,070.9	69,162.09	69,161.98
Balanced point	10,830.7	10,830.7	70,187.59	70,187.57
Tension control	10,582.1	10,582.1	87,591.95	87,591.84
Pure bending	0.0	0.0	13,323.82	13,323.82
Max tension	1,473.1	1,473.1	0.00	0.00

In all of the hand calculations in this example and illustrated above, the results are in precise agreement with the automated exact results obtained from the spColumn program.

10. Conclusions & Observations

The analysis of the reinforced concrete section performed by spColumn conforms to the provisions of the Strength Design Method and Unified Design Provisions with all conditions of strength satisfying the applicable conditions of equilibrium and strain compatibility.

In the calculation shown above a P-M interaction diagram was generated with moments about the X-Axis. Since the section and reinforcement distribution are not symmetrical, a different P-M interaction diagram is required for the other orthogonal direction (where moments are about the Y-Axis) (The following Figures illustrate the two conditions for the case where $f_s = f_y$).

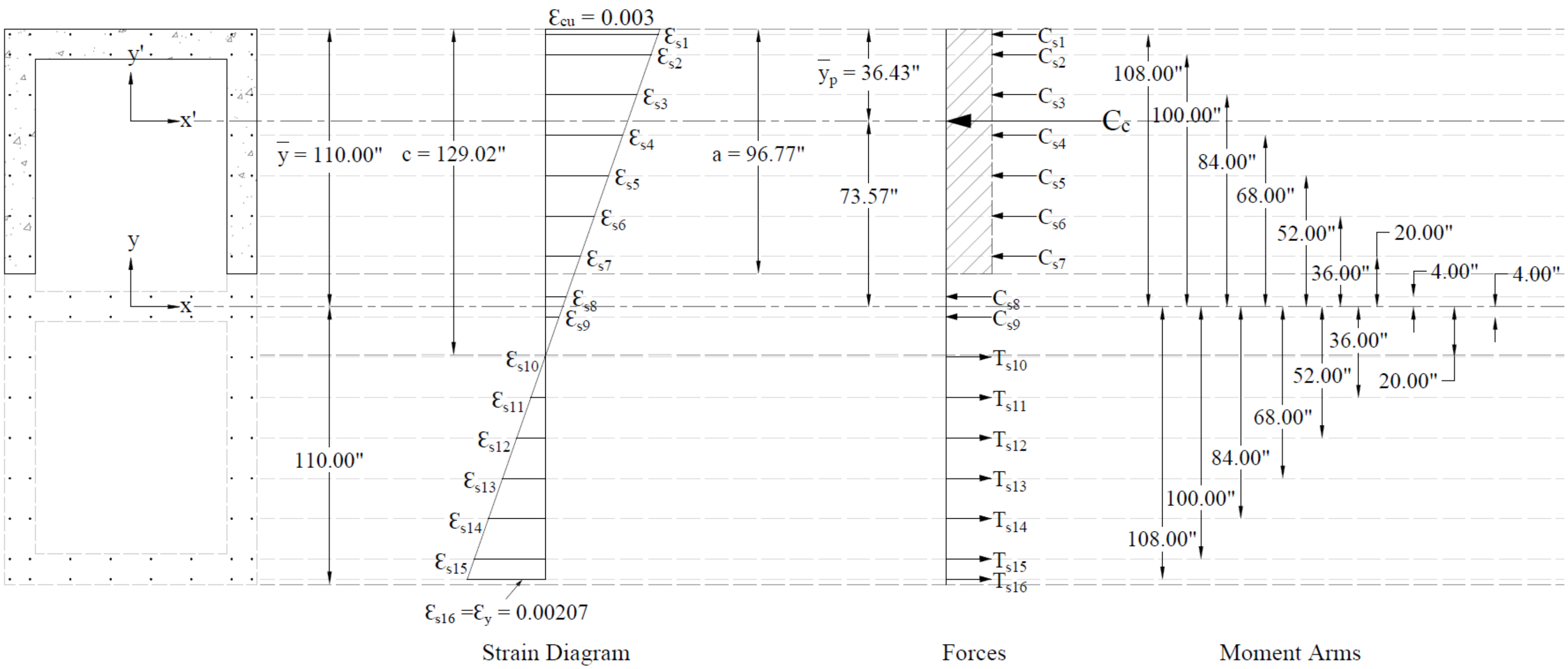


Figure 13 – Strains, Forces, and Moment Arms ($f_s = f_y$ Moments About X-Axis)

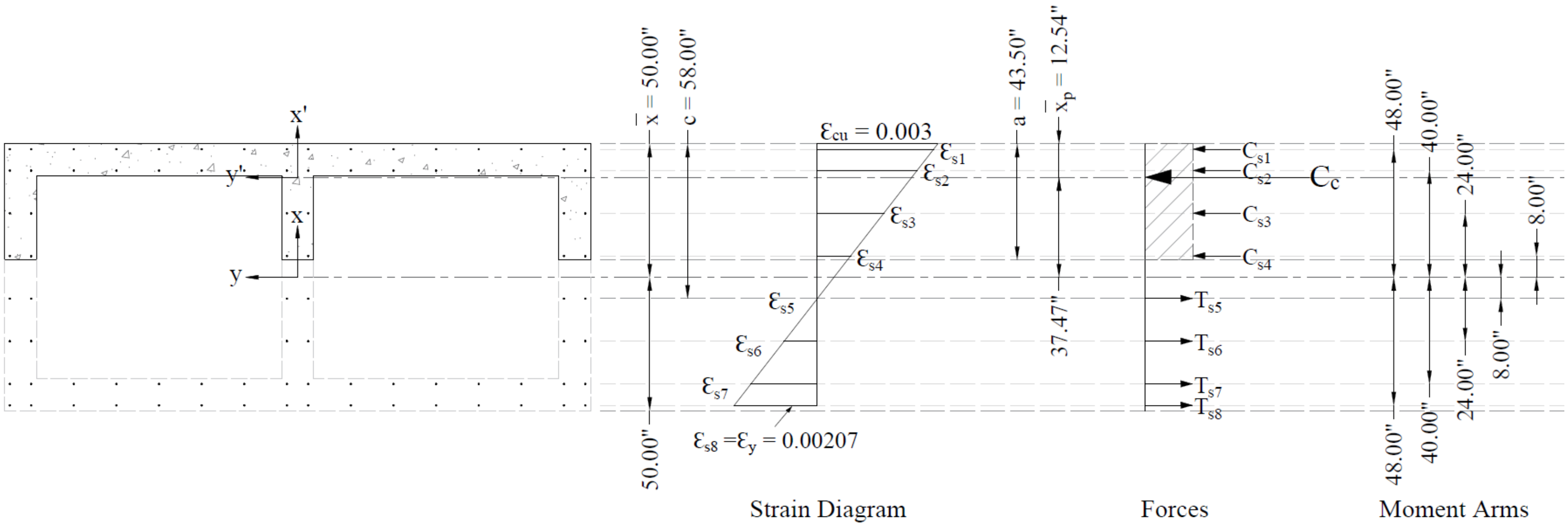
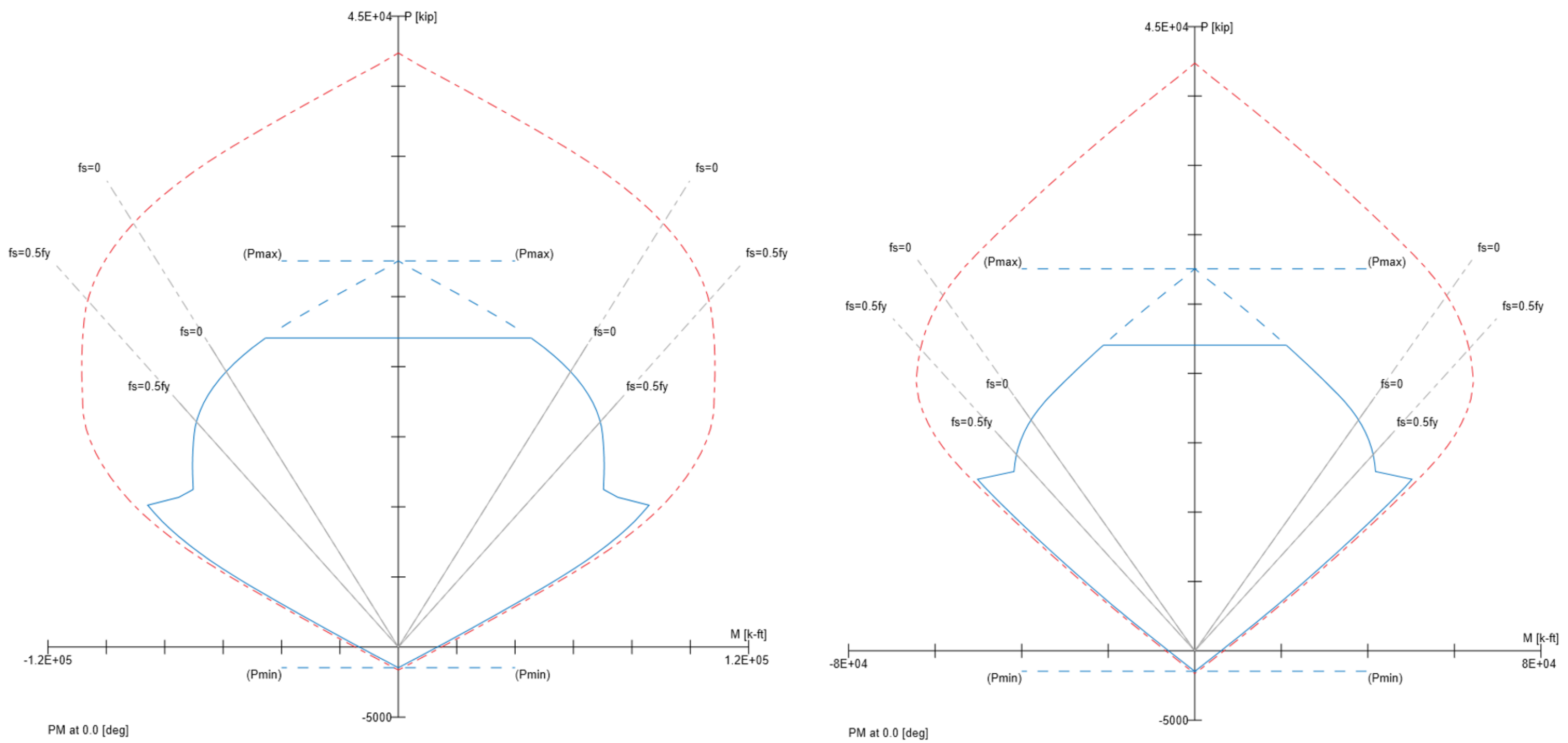


Figure 14 – Strains, Forces, and Moment Arms (f_s = f_y Moments About Y-Axis)

When running about the y-axis in spColumn, 8 layers of reinforcement are participating, instead of 16 layers of reinforcement when running about x-axis, resulting in a completely different P-M interaction diagram as shown in the following spColumn output. The P-M diagrams about x-axis and y-axis are symmetrical since the section is also symmetrical.



Interaction Diagram About the X-Axis

Interaction Diagram About the Y-Axis

Figure 15 – Comparison of Core Wall Interaction Diagrams about X-Axis and Y-Axis (spColumn)

In most building design calculations, such as the examples shown in the StructurePoint website, all building columns and walls are subjected to M_x and M_y due to lateral forces and unbalanced moments from both directions of analysis. This requires an evaluation of the column or wall P-M interaction diagram in two directions simultaneously (biaxial bending) instead of the uniaxial investigation illustrated here.

StucturePoint's spColumn program can also investigate column and wall sections in biaxial mode to produce the results shown in the following Figure for the wall section in this example. In biaxial run mode, M_x and M_y diagrams at each axial force level can be viewed in 2D and 3D views.

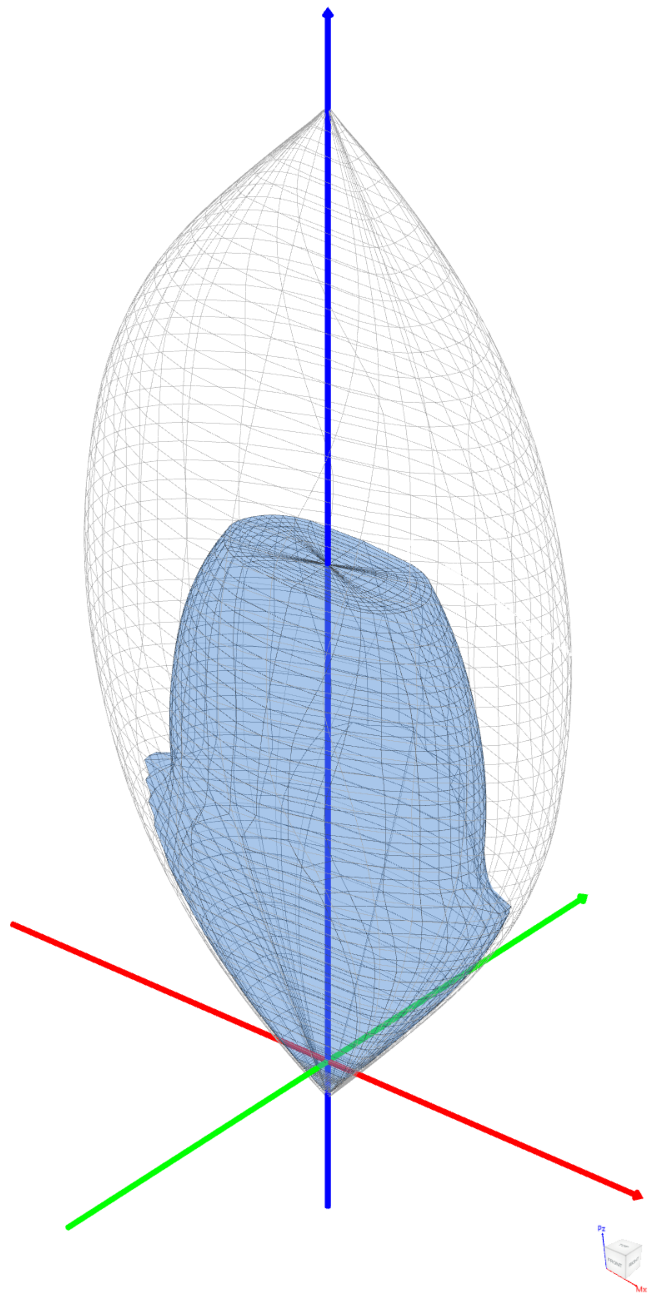


Figure 16 – Core Wall Nominal & Design 3D failure Surfaces (Biaxial) (spColumn)

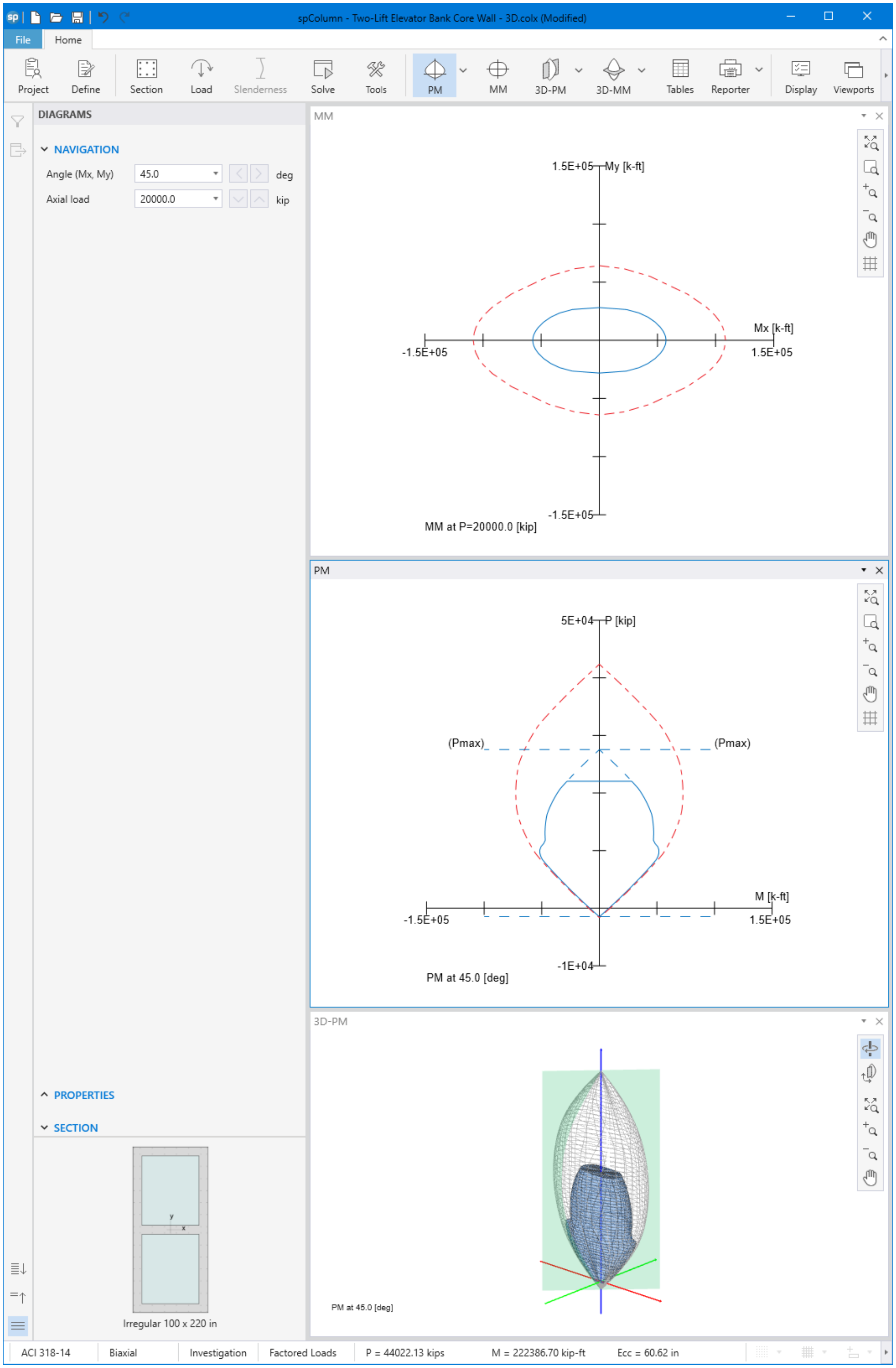


Figure 17 – Core Wall Interaction Diagram and 3D failure Surface Viewer (spColumn)

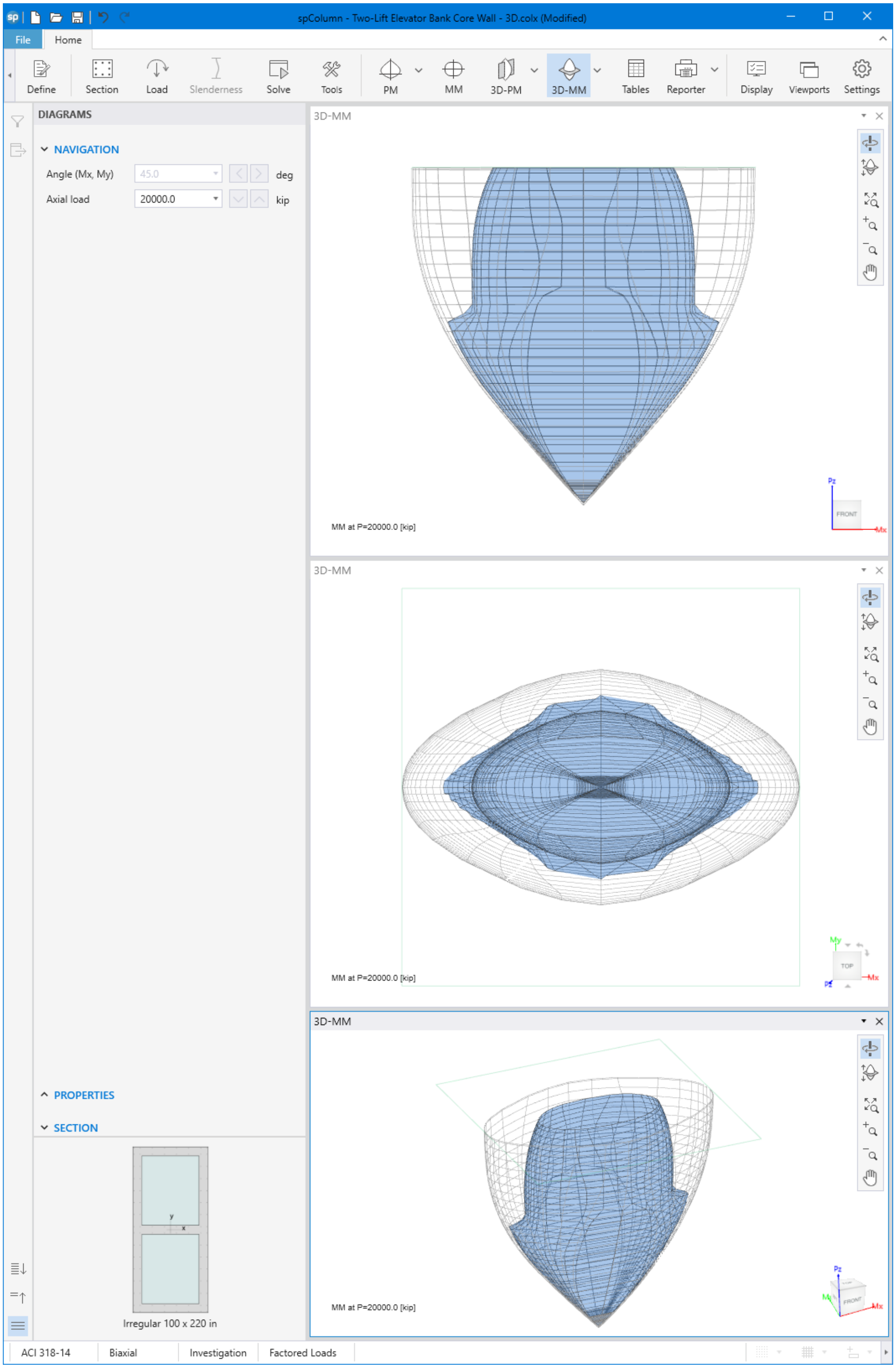


Figure 18 – Core Wall 3D Failure Surface with a Horizontal Plane Cut at P = 20,000 kip (spColumn)

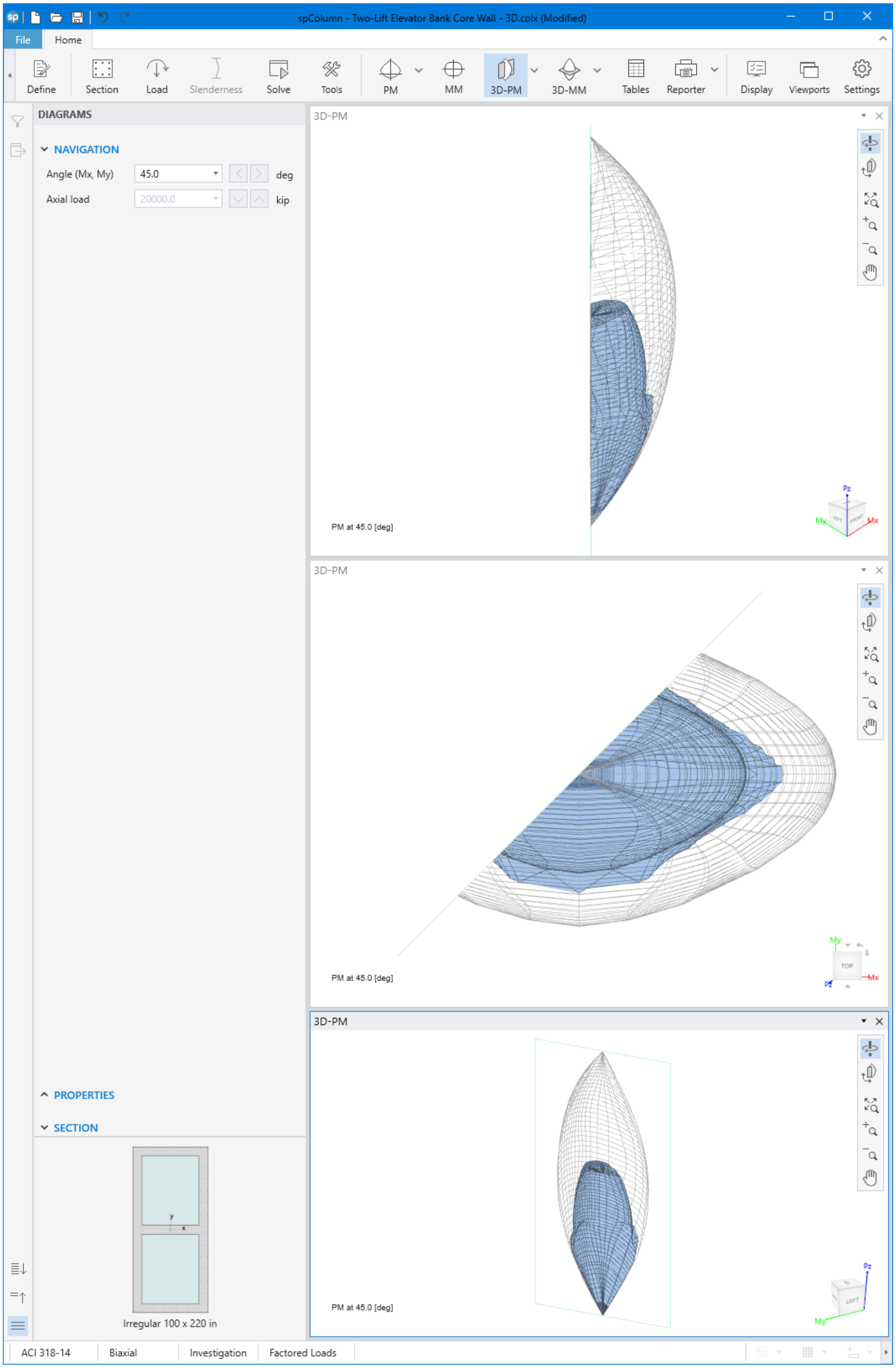


Figure 19 – Core Wall 3D Failure Surface with a Vertical Plane Cut at 45º (spColumn)