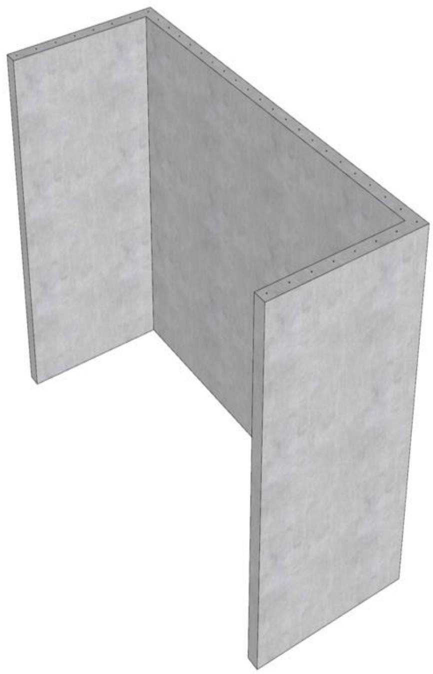
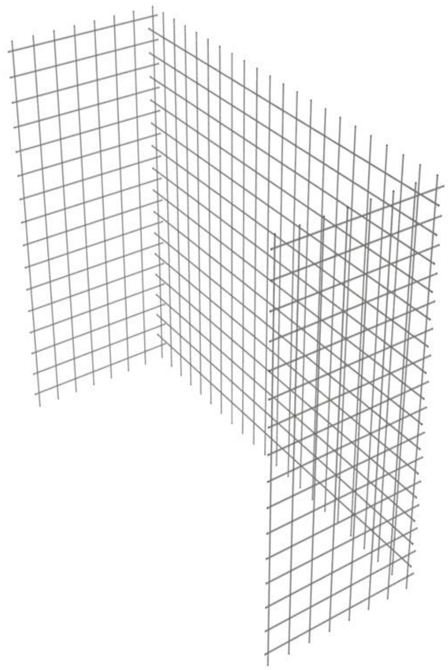
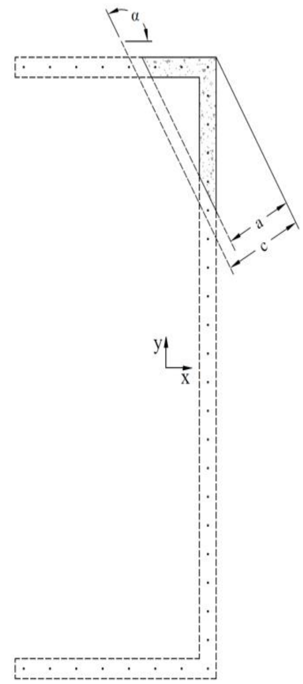
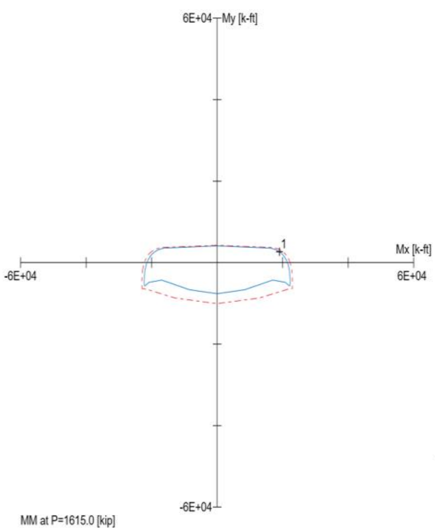
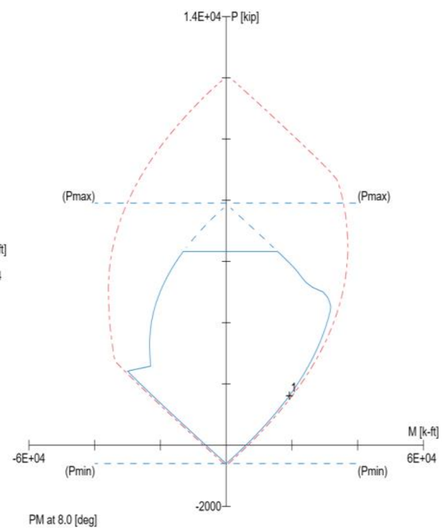
Biaxial Bending Interaction Diagrams for C-Shaped Concrete Core Wall Design (ACI 318-19)

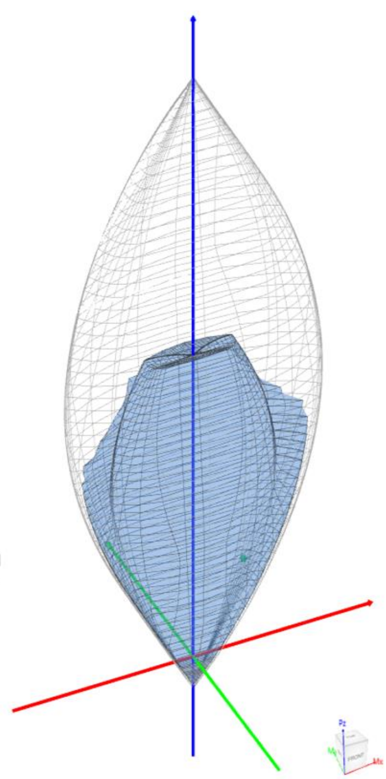












Biaxial Bending Interaction Diagrams for C-Shaped Concrete Core Wall Design (ACI 318-19)

Biaxial bending of columns and walls occurs when the loading causes bending simultaneously about both principal axes. Columns and walls exposed to known moments about each axis simultaneously should be designed for biaxial bending and axial load.

A uniaxial interaction diagram defines the load-moment strength along a single plane of a section under an axial load P and a uniaxial moment M. The biaxial bending resistance of an axially loaded column or wall can be represented schematically as a surface formed by a series of uniaxial interaction curves drawn radially from the P axis. Data for these intermediate curves are obtained by varying the angle of the neutral axis (for assumed strain configurations) with respect to the major axes.

The difficulty associated with the determination of the strength of reinforced columns or walls subjected to combined axial load and biaxial bending is primarily an arithmetic one. The bending resistance of an axially loaded column or wall about a particular skewed axis is determined through iterations involving simple but lengthy calculations. These extensive calculations are compounded when optimization of the reinforcement or cross-section is sought.

This example demonstrates the determination of the design axial load capacity, ϕP_n, and the design ϕM_{nx} and ϕM_{ny} moments corresponding to the following case: The neutral axis depth of 36.12 in., at an angle of 120º counterclockwise from the x-axis of the cross section. The figure below shows the reinforced concrete C-shaped core wall cross section in consideration. The calculated values of the wall axial strength and biaxial bending strength are compared with the exact values from spColumn engineering software program from StructurePoint. The steps to develop the three-dimensional failure surface (interaction diagram) using spColumn will be shown in detail as well.

This core has been extracted from the complete design example presented in Chapter 6 of "Simplified Design of Reinforced Concrete Buildings" book to provide lateral support of a multi-story building. Additional background information about the building geometry and loads can be found in the reference.

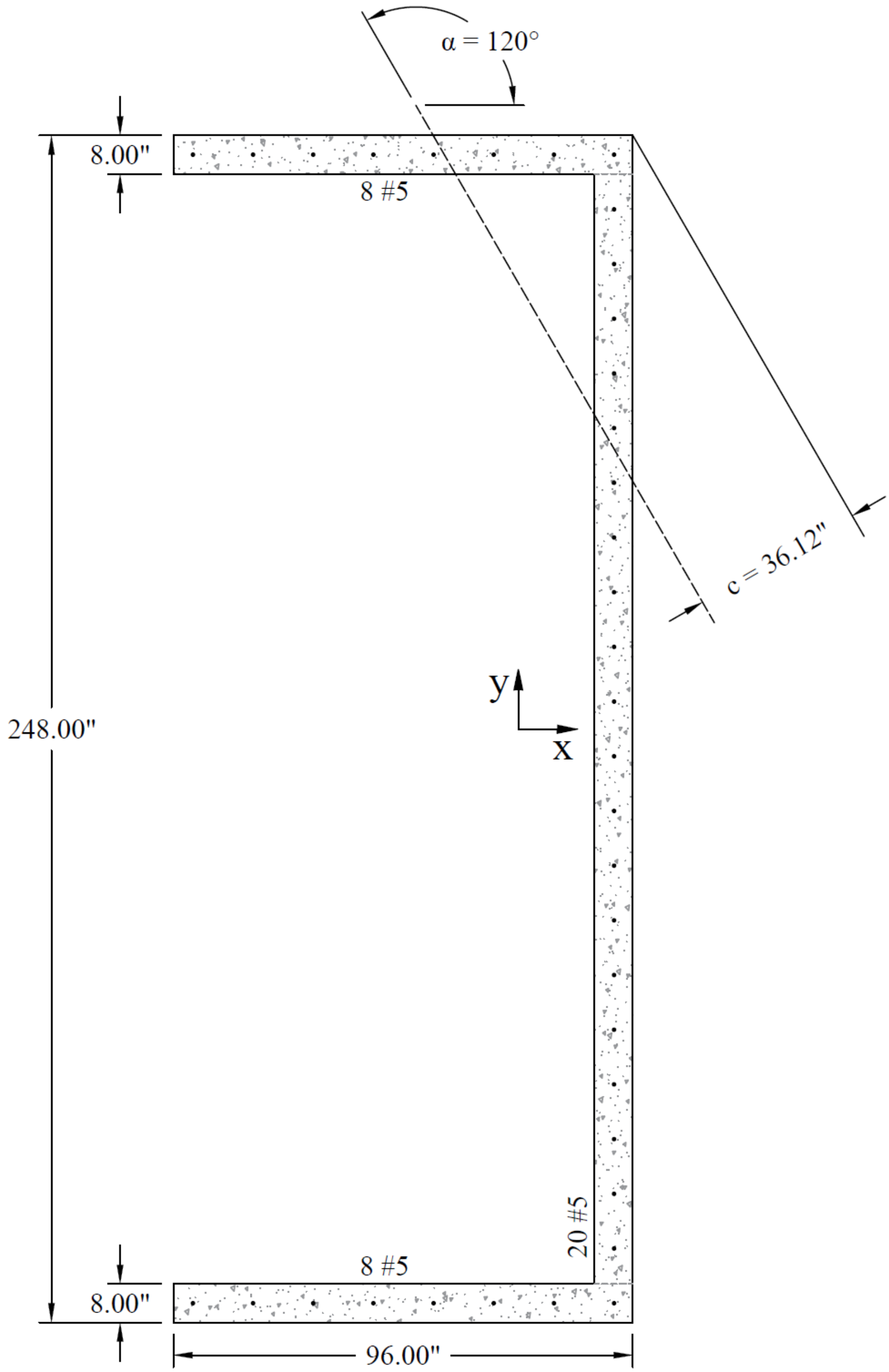


Figure 1 – Reinforced Concrete C-Shaped Core Wall Cross-Section

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1. C-Shaped Core Wall Biaxial Strength Calculations 3
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Code

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

References

- Notes on ACI 318-11 Building Code Requirements for Structural Concrete, Twelfth Edition, 2013 Portland Cement Association
- Simplified Design of Reinforced Concrete Buildings, Fourth Edition, 2011 Portland Cement Association
- spColumn Engineering Software Program Manual v10.00, STRUCTUREPOINT, 2021
- "Biaxial Bending Interaction Diagrams for Square Reinforced Concrete Column Design (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022
- "Biaxial Bending Interaction Diagrams for Rectangular Reinforced Concrete Column Design (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022
- "Biaxial Bending Interaction Diagrams for Spiral Reinforced Circular Concrete Column Design (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022
- "Manual Design Procedure for Columns and Walls with Biaxial Bending (ACI 318-11/14/19)" Design Example, STRUCTUREPOINT, 2022

Design Data

f_c' = 4000 psi

f_y = 60000 psi

Wall geometry and reinforcement locations are shown in following figure.

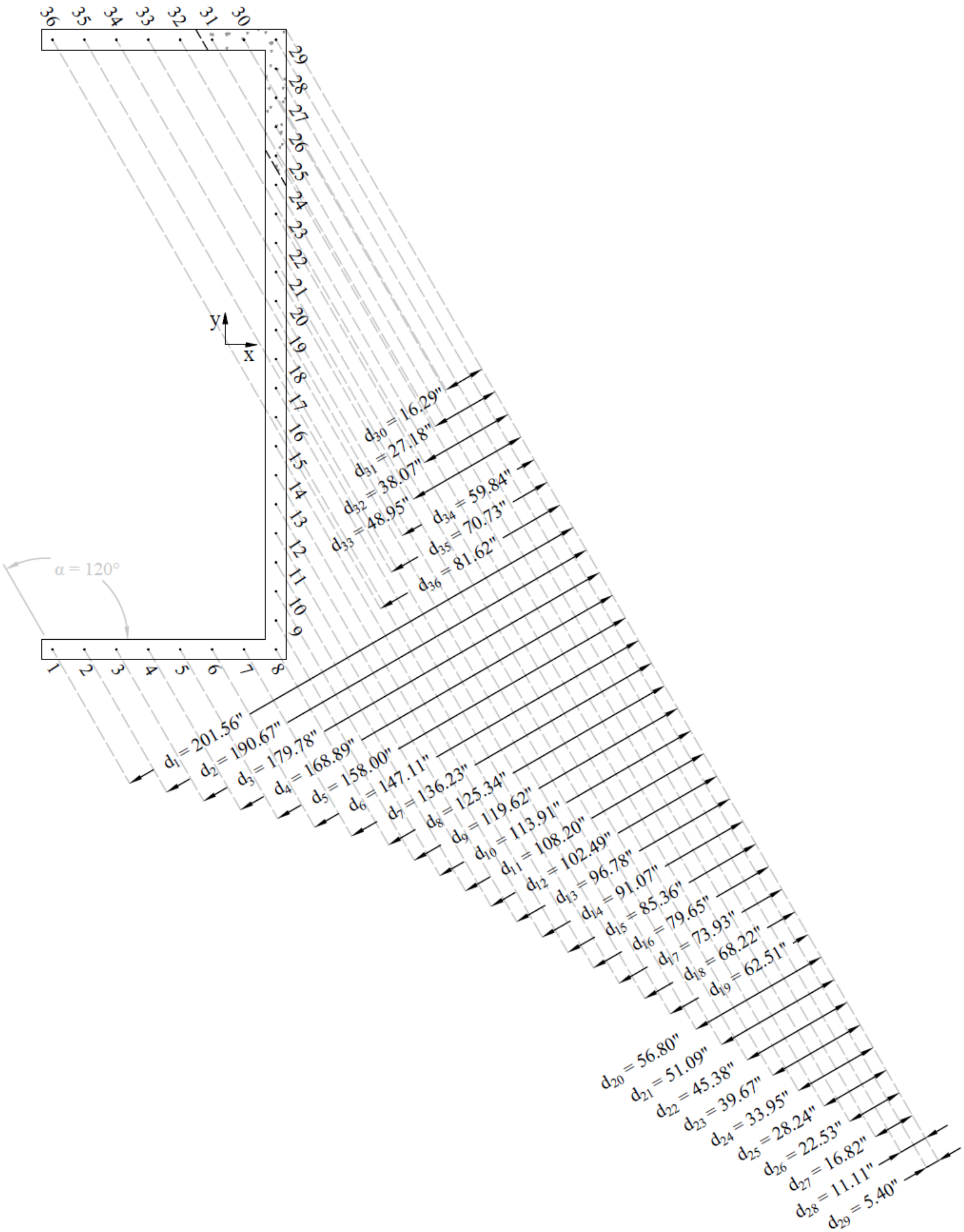


Figure 2 – C-Shaped Core Wall Cross-Section and Reinforcement Locations

Solution

In a reinforced concrete column or wall, the determination of the nominal axial load capacity, P_n, and the nominal M_{nx} and M_{ny} moments involves a trial-and-error process for calculating the neutral axis depth and angle α. In this example, the neutral axis depth and angle are provided as an input (c = 36.12 in. and an angle of α = 120º) for illustration.

The steps to calculate biaxial flexural strength of a reinforced concrete column or wall for a given nominal axial strength and moment ratio of biaxial bending moments is discussed in details in "Biaxial Bending Interaction Diagrams for Rectangular Reinforced Concrete Column Design (ACI 318-19)" design example.

1. C-Shaped Core Wall Biaxial Strength Calculations

The following three figures display the section's strain diagram, internal forces and the corresponding moment arms in the necessary nomenclature to prepare for the strength calculations of each of the following:

- Design Axial Strength (ϕP_n)

  Figure 3 shows the strain diagram for the reinforcement and concrete based on the neutral axis location and angle values provided. The internal forces for the reinforcement and concrete compression block are calculated based on the strain values. ϕ is calculated based on the strain in the extreme tension reinforcement layer.

- Design Flexural Strength (ϕM_{nx})

  The flexural strength ϕM_{nx} can be calculated using force values and moment arms from the x-axis (r_y) as shown in Figure 4.

- Design Flexural Strength (ϕM_{ny})

  The flexural strength ϕM_{ny} can be calculated using force values and moment arms from the y-axis (r_x) as shown in Figure 5.

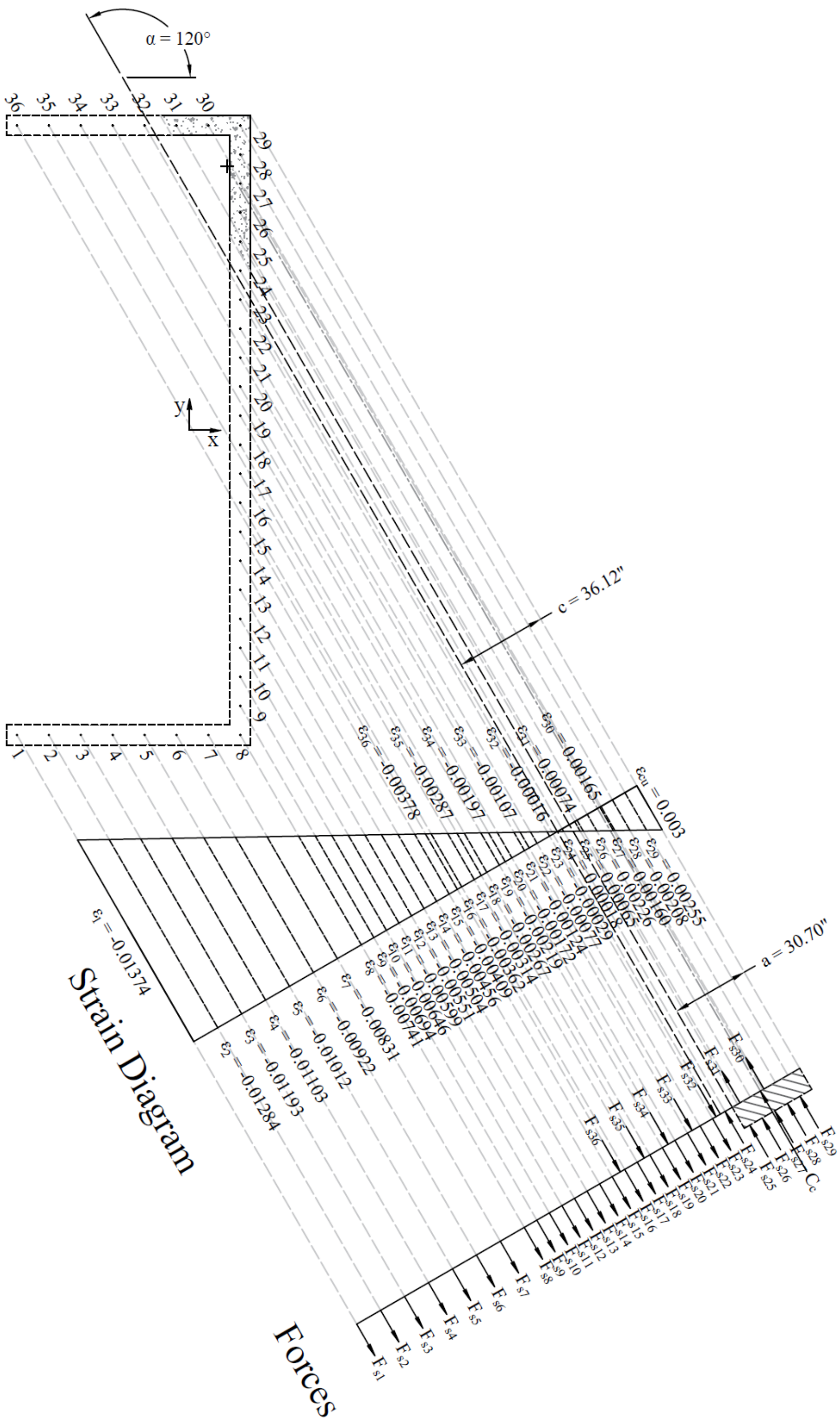


Figure 3 – Design Axial Strength (ϕP_n) Calculations

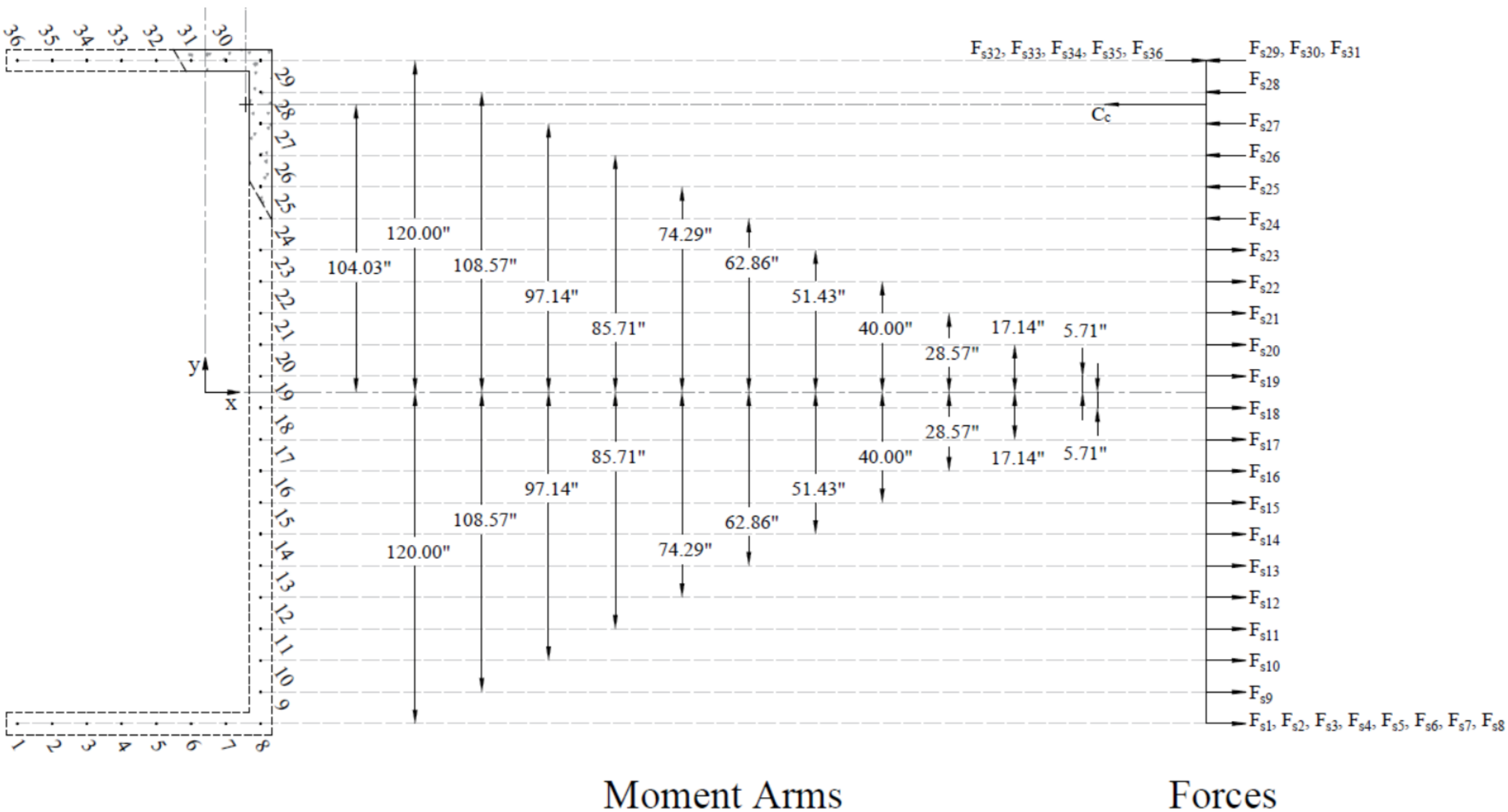


Figure 4 – Design Flexural Strength (ϕM_{nx}) Calculations

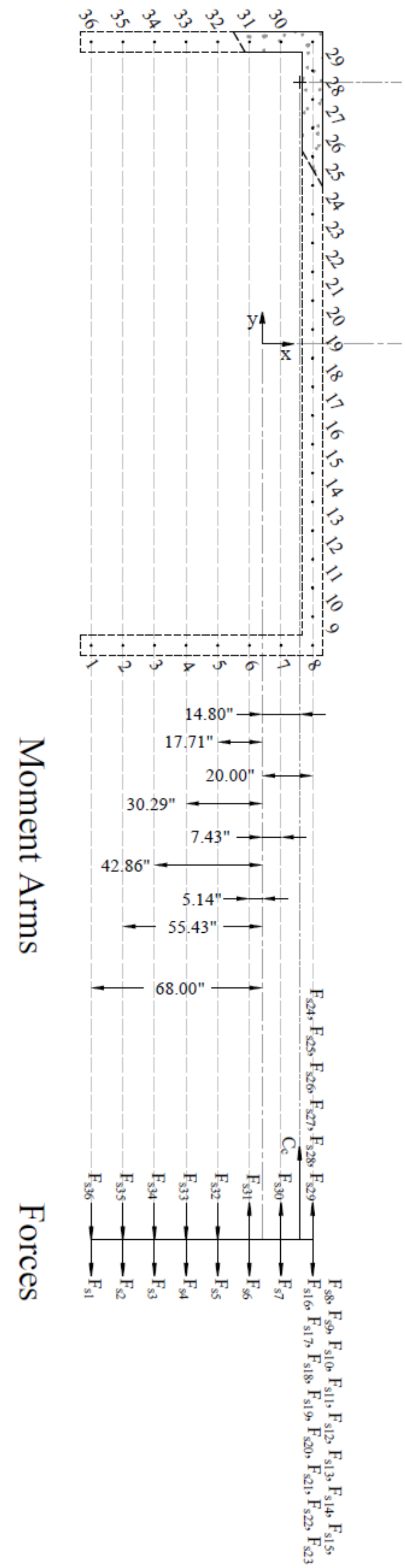


Figure 5 – Design Flexural Strength (ϕM_{ny}) Calculations

1.1. Neutral Axis Location and Concrete Compression Force

The trial-and-error process for calculating the neutral axis depth and angle α is not required in this example since these values are given (c = 36.12 in. and α = 120°). Where *c* is the distance from the fiber of maximum compressive strain to the neutral axis and α is the angle of the neutral axis.

ACI 318-19 (22.2.2.4.2)

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

$$\varepsilon_{s1} = (c - d_1) \times \frac{\varepsilon_{cu}}{c} = (36.12 - 201.56) \times \frac{0.003}{36.12} = -0.01374 \text{ (Tension)} > \varepsilon_y \rightarrow \text{reinforcement has yielded}$$

$$\varepsilon_{s1} > 0.005$$

$$\therefore \phi = 0.90$$ ***ACI 318-19 (Table 21.2.2)***

$$a = \beta_1 \times c = 0.85 \times 36.12 = 30.70 \text{ in.}$$ ***ACI 318-19 (22.2.2.4.1)***

$$\varepsilon_{cu} = 0.003$$ ***ACI 318-19 (22.2.2.1)***

Where:

a = Depth of equivalent rectangular stress block

$$\beta_1 = 0.85 - \frac{0.05 \times (f'_c - 4000)}{1000} = 0.85 - \frac{0.05 \times (4000 - 4000)}{1000} = 0.85$$ ***ACI 318-19 (Table 22.2.2.4.3)***

$$C_c = 0.85 \times f'_c \times A_{comp} = 0.85 \times 4000 \times 637.12 = 2166.22 \text{ kip (Compression)}$$ ***ACI 318-19 (22.2.2.4.1)***

Calculate the area of the section subject to compression and its centroid by examining the four sub segments as shown in the following figure:

$$A_1 = \frac{13.87 \times 8.00}{2} = 55.48 \text{ in.}^2 \qquad A_2 = 39.57 \times 8.00 = 316.56 \text{ in.}^2$$

$$A_3 = 8.00 \times (35.45 - 4.62) = 246.64 \text{ in.}^2 \qquad A_4 = \frac{8.00 \times 4.62}{2} = 18.48 \text{ in.}^2$$

$$\overline{x_1} = \frac{8.00}{3} = 2.67 \text{ in.} \qquad \overline{x_2} = \frac{8.00}{2} = 4.00 \text{ in.}$$

$$\overline{x_3} = \frac{(35.45 - 4.62)}{2} = 15.42 \text{ in.} \qquad \overline{x_4} = (35.45 - 4.62) + \frac{4.62}{3} = 32.37 \text{ in.}$$

$$\overline{y_1} = 8.00 + 39.57 + \frac{13.87}{3} = 52.19 \text{ in.} \qquad \overline{y_2} = 8.00 + \frac{39.57}{2} = 27.79 \text{ in.}$$

$$\overline{y_3} = \frac{8.00}{2} = 4.00 \text{ in.} \qquad \overline{y_4} = \frac{8.00}{3} = 2.67 \text{ in.}$$

$$A_{comp} = A_1 + A_2 + A_3 + A_4 = 637.12 \text{ in.}^2$$

$$\bar{x} = \left(\frac{A_1 \times \overline{x_1} + A_2 \times \overline{x_2} + A_3 \times \overline{x_3} + A_4 \times \overline{x_4}}{A_1 + A_2 + A_3 + A_4} \right) - \overline{X} = 14.80 \text{ in.}$$

$$\bar{y} = \left(\frac{A_1 \times \overline{y_1} + A_2 \times \overline{y_2} + A_3 \times \overline{y_3} + A_4 \times \overline{y_4}}{A_1 + A_2 + A_3 + A_4} \right) - \overline{Y} = 104.03 \text{ in.}$$

Note that $\overline{X}$ and $\overline{Y}$ are the coordinates of the centroid of the entire cross-section (uncracked core wall section).

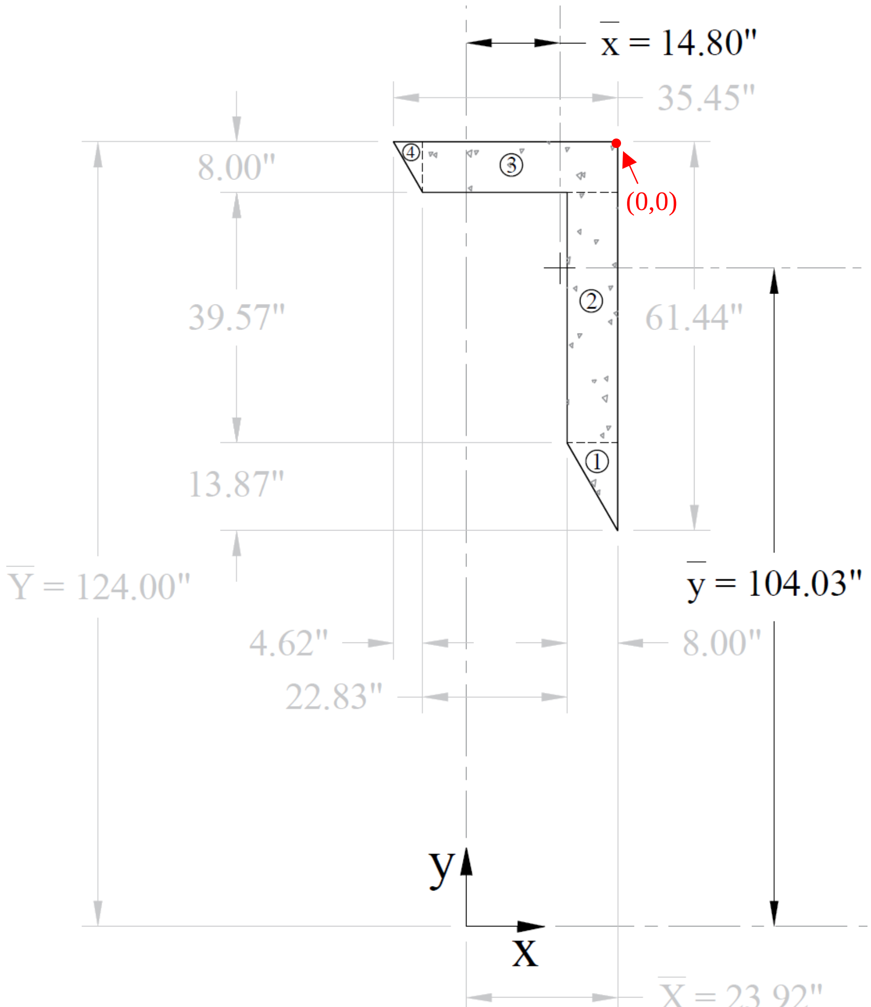


Figure 6 – Cracked Concrete Wall Section Centroid Calculations

1.2. Determination of Reinforcement Strains and Forces

The following shows the calculations of forces in the reinforcement layers with the extreme tension (at bar 1) and extreme compression (at bar 29) strains. The calculations for the rest of layers are shown the table at the end of this section.

For extreme tension reinforcement layer (at bar 1):

$$\varepsilon_{s1} = -0.01374 \text{ (Tension)} < \varepsilon_y \rightarrow \text{reinforcement has yielded}$$

$$\therefore f_{s1} = f_y = -60000 \text{ psi}$$

$$\text{F}_{s1} = f_{s1} \times A_{s1} = -60000 \times (1 \times 0.31) = -18.60 \text{ kip} \quad \text{(Tension)}$$

For extreme compression reinforcement layer (at bar 29):

$$\varepsilon_{s29} = (c - d_{29}) \times \frac{\varepsilon_{cu}}{c} = (36.12 - 5.40) \times \frac{0.003}{36.12} = 0.00255 \text{ (Compression)} > \varepsilon_y \rightarrow \text{reinforcement has yielded}$$

$$\therefore f_{s29} = f_y = 60000 \text{ psi}$$

The area of the reinforcement in this layer is included in the area used to compute C_c (a = 30.70 in. > d_{29} = 5.40 in.). As a result, it is necessary to subtract $0.85 f_c'$ from f_{s29} before computing F_{s29}:

$$\text{F}_{s29} = f_{s29} \times A_{s29} = (60000 - 0.85 \times 4000) \times (1 \times 0.31) = 17.55 \text{ kip} \quad \text{(Compression)}$$

The same procedure shown above can be repeated to calculate the forces in the remaining reinforcement locations, results are summarized in the table shown in the next page.

1.3. Calculation of ϕP_n, ϕM_{nx} and ϕM_{ny}

$$P_n = C_c + \sum F_s$$ (+) = Compression (-) = Tension

$$\phi P_n = \phi \times P_n = 0.65 \times P_n$$

$$M_{ny} = C_c \times \left(\frac{b}{2} - \bar{x}_c \right) + \sum_{i=1}^{n=36} \left(F_{si} \times \left(\frac{b}{2} - x_i \right) \right)$$ (+) = Counter Clockwise (-) = Clockwise

$$\phi M_{ny} = \phi \times M_{ny} = 0.65 \times M_{ny}$$

$$M_{nx} = C_c \times \left(\frac{h}{2} - \bar{y}_c \right) + \sum_{i=1}^{n=36} \left(F_{si} \times \left(\frac{h}{2} - y_i \right) \right)$$ (+) = Counter Clockwise (-) = Clockwise

$$\phi M_{nx} = \phi \times M_{nx} = 0.65 \times M_{nx}$$

Table 1 - Strains, internal force resultants and Moments									
Location	d, in.	ε, in./in.	f_s, psi	F_s, kip	C_c, kip	r_x, in.	M_y, kip-ft	r_y, in.	M_x, kip-ft
Concrete	---	0.003	---	---	2166.2	14.8	2671.6	104.0	18778.4
Bar 1	201.56	-0.01374	-60000.0	-18.6	---	-68.0	105.4	-120.0	186.0
Bar 2	190.67	-0.01284	-60000.0	-18.6	---	-55.4	85.9	-120.0	186.0
Bar 3	179.78	-0.01193	-60000.0	-18.6	---	-42.9	66.4	-120.0	186.0
Bar 4	168.89	-0.01103	-60000.0	-18.6	---	-30.3	46.9	-120.0	186.0
Bar 5	158.00	-0.01012	-60000.0	-18.6	---	-17.7	27.5	-120.0	186.0
Bar 6	147.11	-0.00922	-60000.0	-18.6	---	-5.2	8.0	-120.0	186.0
Bar 7	136.23	-0.00831	-60000.0	-18.6	---	7.4	-11.5	-120.0	186.0
Bar 8	125.34	-0.00741	-60000.0	-18.6	---	20.0	-31.0	-120.0	186.0
Bar 9	119.62	-0.00694	-60000.0	-18.6	---	20.0	-31.0	-108.6	168.3
Bar 10	113.91	-0.00646	-60000.0	-18.6	---	20.0	-31.0	-97.1	150.6
Bar 11	108.20	-0.00599	-60000.0	-18.6	---	20.0	-31.0	-84.7	131.3
Bar 12	102.49	-0.00551	-60000.0	-18.6	---	20.0	-31.0	-74.3	115.1
Bar 13	96.78	-0.00504	-60000.0	-18.6	---	20.0	-31.0	-63.2	98.0
Bar 14	91.07	-0.00456	-60000.0	-18.6	---	20.0	-31.0	-51.4	79.7
Bar 15	85.36	-0.00409	-60000.0	-18.6	---	20.0	-31.0	-40.0	62.0
Bar 16	79.65	-0.00362	-60000.0	-18.6	---	20.0	-31.0	-28.6	44.3
Bar 17	73.93	-0.00314	-60000.0	-18.6	---	20.0	-31.0	-17.1	26.6
Bar 18	68.22	-0.00267	-60000.0	-18.6	---	20.0	-31.0	-5.7	8.9
Bar 19	62.51	-0.00219	-60000.0	-18.6	---	20.0	-31.0	5.7	-8.9
Bar 20	56.80	-0.00172	-49810.6	-15.4	---	20.0	-25.7	17.1	-22.1
Bar 21	51.09	-0.00124	-36057.3	-11.2		20.0	-18.6	28.6	-26.6
Bar 22	45.38	-0.00077	-22304.0	-6.9	---	20.0	-11.5	40.0	-23.0
Bar 23	39.67	-0.00029	-8550.7	-2.7	---	20.0	-4.4	51.4	-11.4
Bar 24	33.95	0.00018	5226.7	1.6	---	20.0	2.7	62.9	8.5
Bar 25*	28.24	0.00065	18980.0	4.8	---	20.0	8.1	74.3	29.9
Bar 26*	22.53	0.00113	32733.4	9.1	---	20.0	15.2	85.7	64.9
Bar 27*	16.82	0.00160	46486.7	13.4	---	20.0	22.3	97.1	108.1
Bar 28*	11.11	0.00208	60000.0	17.6	---	20.0	29.3	108.6	158.8
Bar 29*	5.40	0.00255	60000.0	17.6	---	20.0	29.3	120.0	175.5
Bar 30*	16.29	0.00165	47763.3	13.8	---	7.4	8.5	120.0	137.5
Bar 31*	27.18	0.00074	21533.2	5.6	---	-5.2	-2.4	120.0	56.2
Bar 32	38.07	-0.00016	-4696.8	-1.5	---	-17.7	2.2	120.0	-14.9
Bar 33	48.95	-0.00107	-30902.8	-9.6	---	-30.3	24.2	120.0	-95.8
Bar 34	59.84	-0.00197	-57132.9	-17.7	---	-42.9	63.3	120.0	-177.1
Bar 35	70.73	-0.00287	-60000.0	-18.6	---	-55.4	85.9	120.0	-186.0
Bar 36	81.62	-0.00378	-60000.0	-18.6	---	-68.0	105.4	120.0	-186.0
Axial Force and Biaxial			P_n, kip		1794.06	M_{ny}, kip-ft	2961.61	M_{nx}, kip-ft	21139.26
Bending Moments Capacities			ϕP_n, kip		1614.65	ϕM_{ny}, kip-ft	2665.45	ϕM_{nx}, kip-ft	19025.34

* The area of the reinforcement in this layer has been included in the area used to compute C_c. As a result, $0.85f_c'$ is subtracted from f_s in the computation of F_s.

2. C-Shaped Core Wall Biaxial Bending 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 biaxial runs, the values of maximum compressive axial load capacity and maximum tensile load capacity are computed. These two values set the range within which the moment capacities are computed for a predetermined number of axial load values. For each level of axial load, the section is rotated in 10-degree increments from 0 degrees to 360 degrees and the M_x and M_y moment capacities are computed. Thus, for each level of axial load, an M_x-M_y contour is developed. Repeating this for the entire range of axial loads, the three-dimensional failure surface is computed. A three-dimensional visualization of the resulting entire nominal and factored failure surface is provided to support enhanced understanding of the section capacity.

The **"biaxial"** feature allows the user to investigate the P-M interaction diagrams, the M_x-M_y moment contour plots, as well as the 3D failure surface for irregular shaped column, beam, and wall sections quickly, simply, and accurately.

spColumn model editor can be used to handle irregular shapes and unusual bar arrangement like the C-shaped wall section illustrated in this design example. Alternatively, the section can be imported to spColumn from an AutoCad model using DXF file format. For this core wall section, we ran in investigation mode with **"biaxial"** option for "Run Axis" using the ACI 318-19.

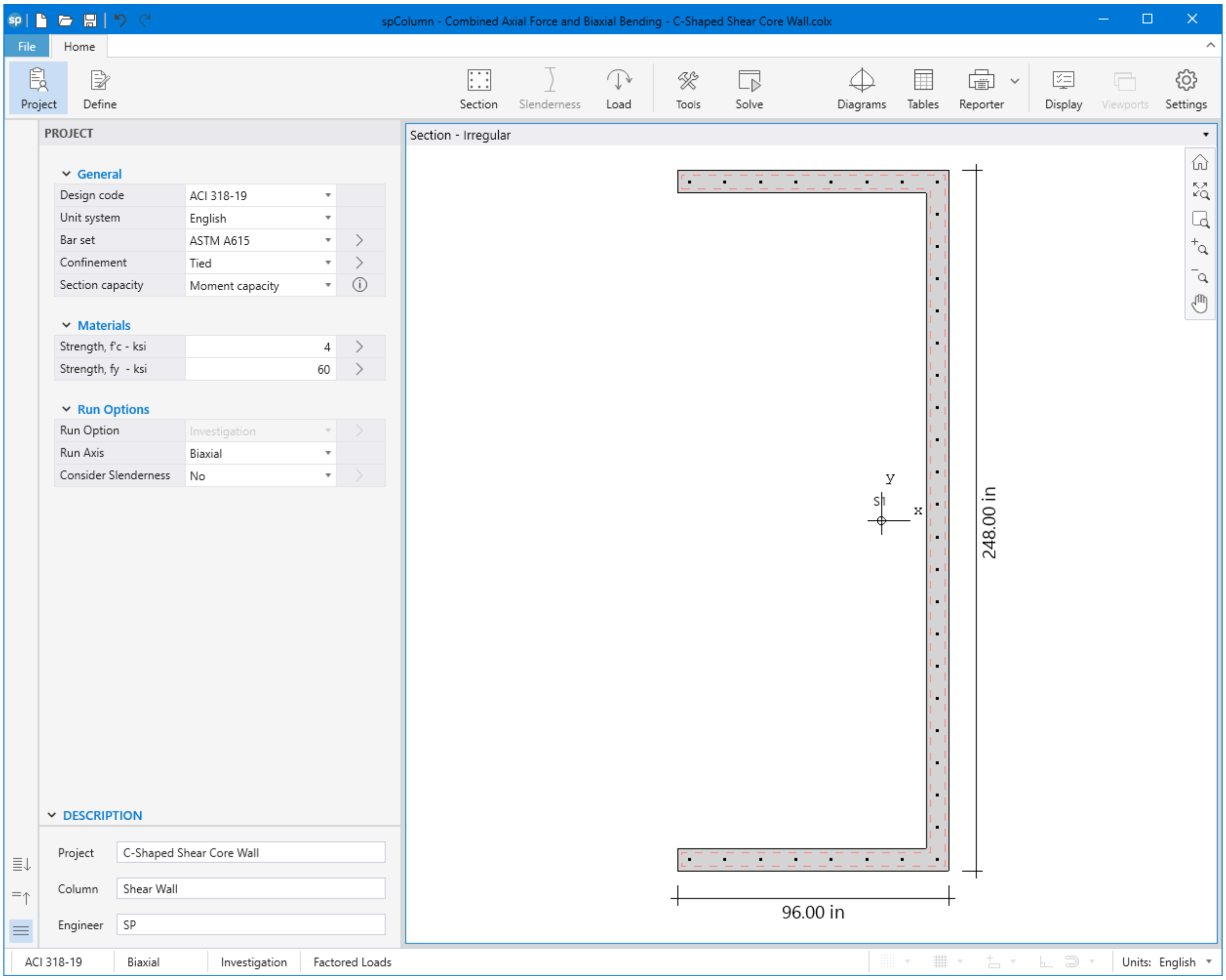


Figure 7 – spColumn Interface

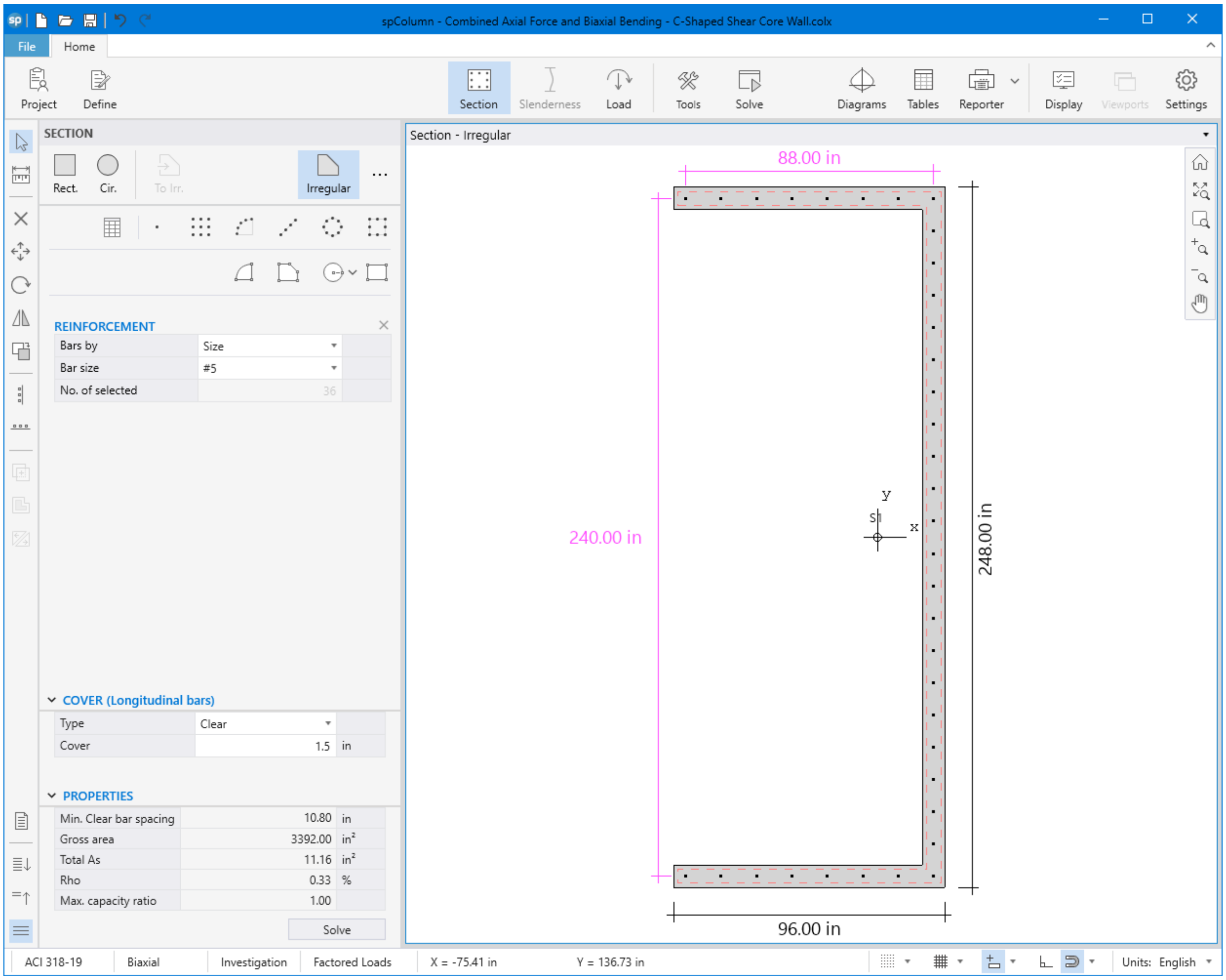


Figure 8 – spColumn Model Editor

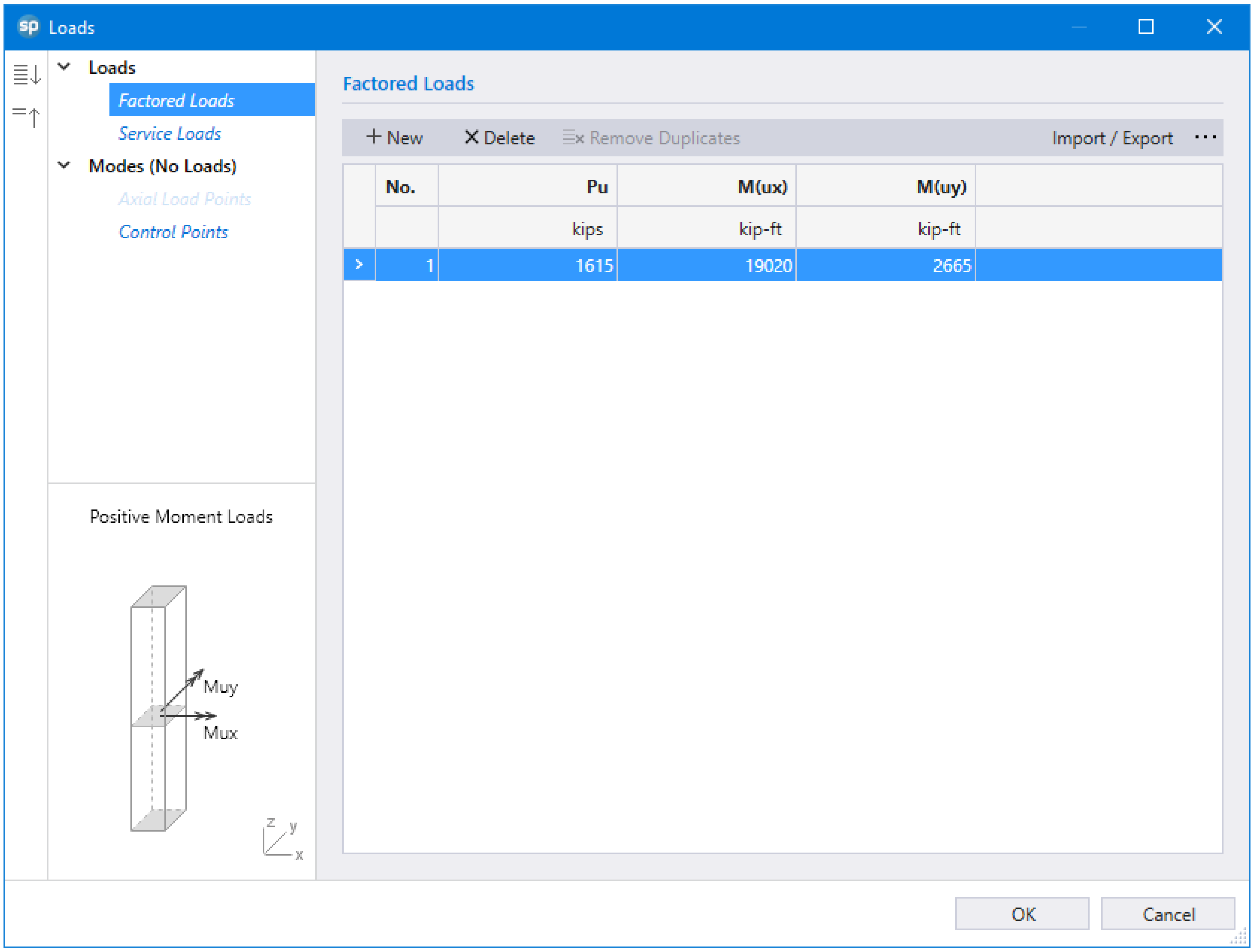


Figure 9 – Defining Loads / Modes (spColumn)

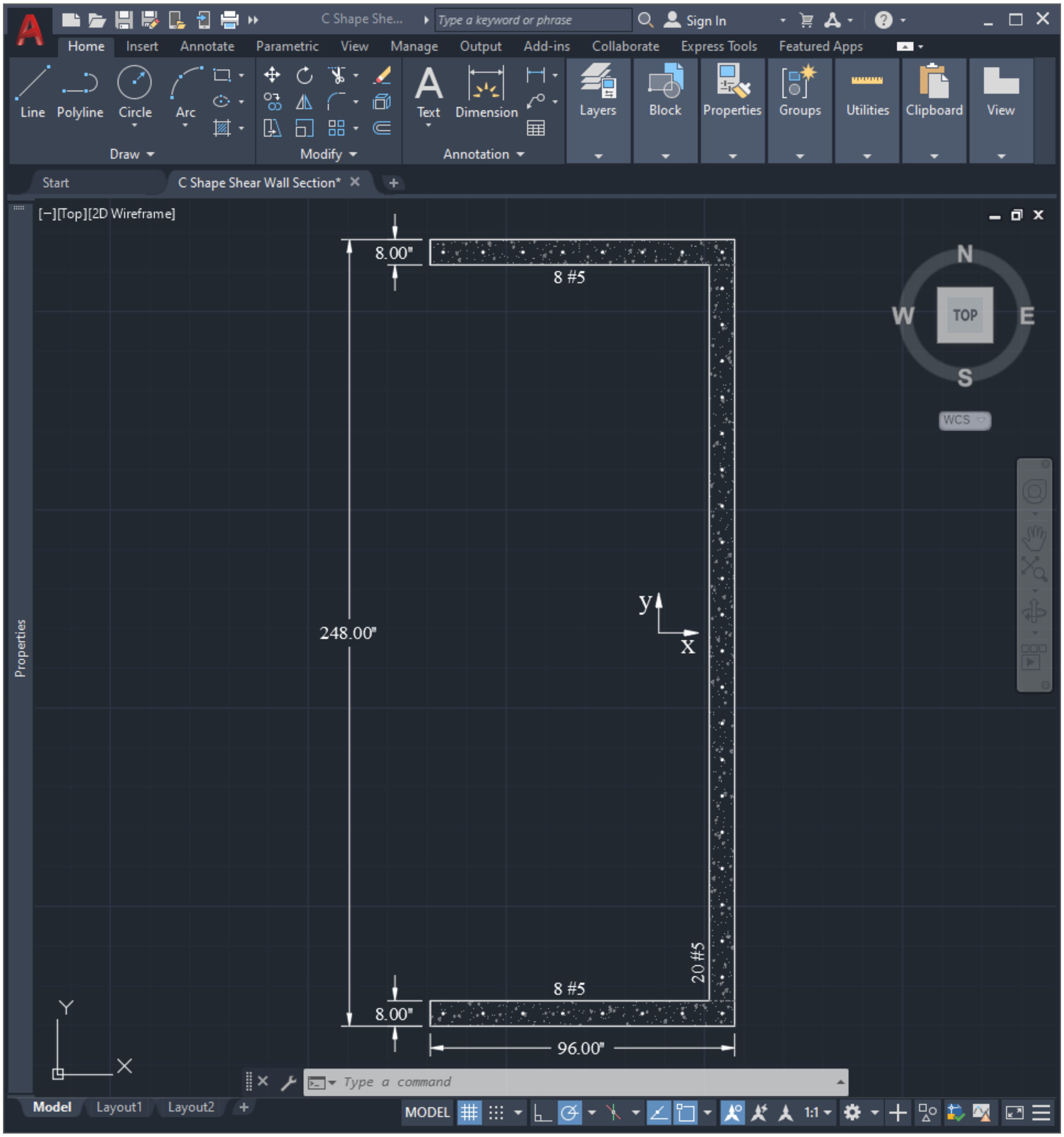


Figure 10 – spColumn DXF file Import from AutoCad

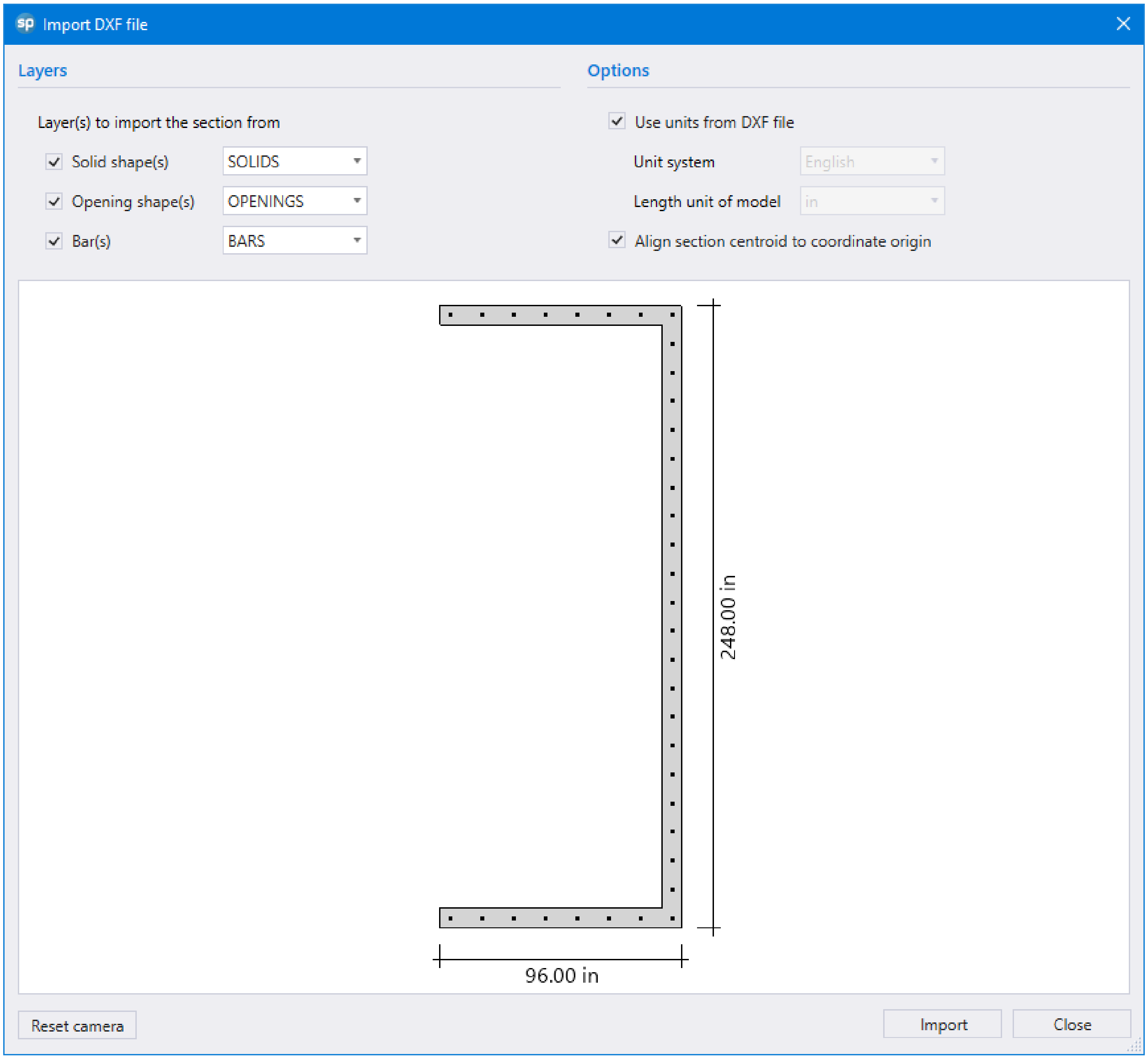


Figure 11 – Importing Wall Section from DXF file to spColumn

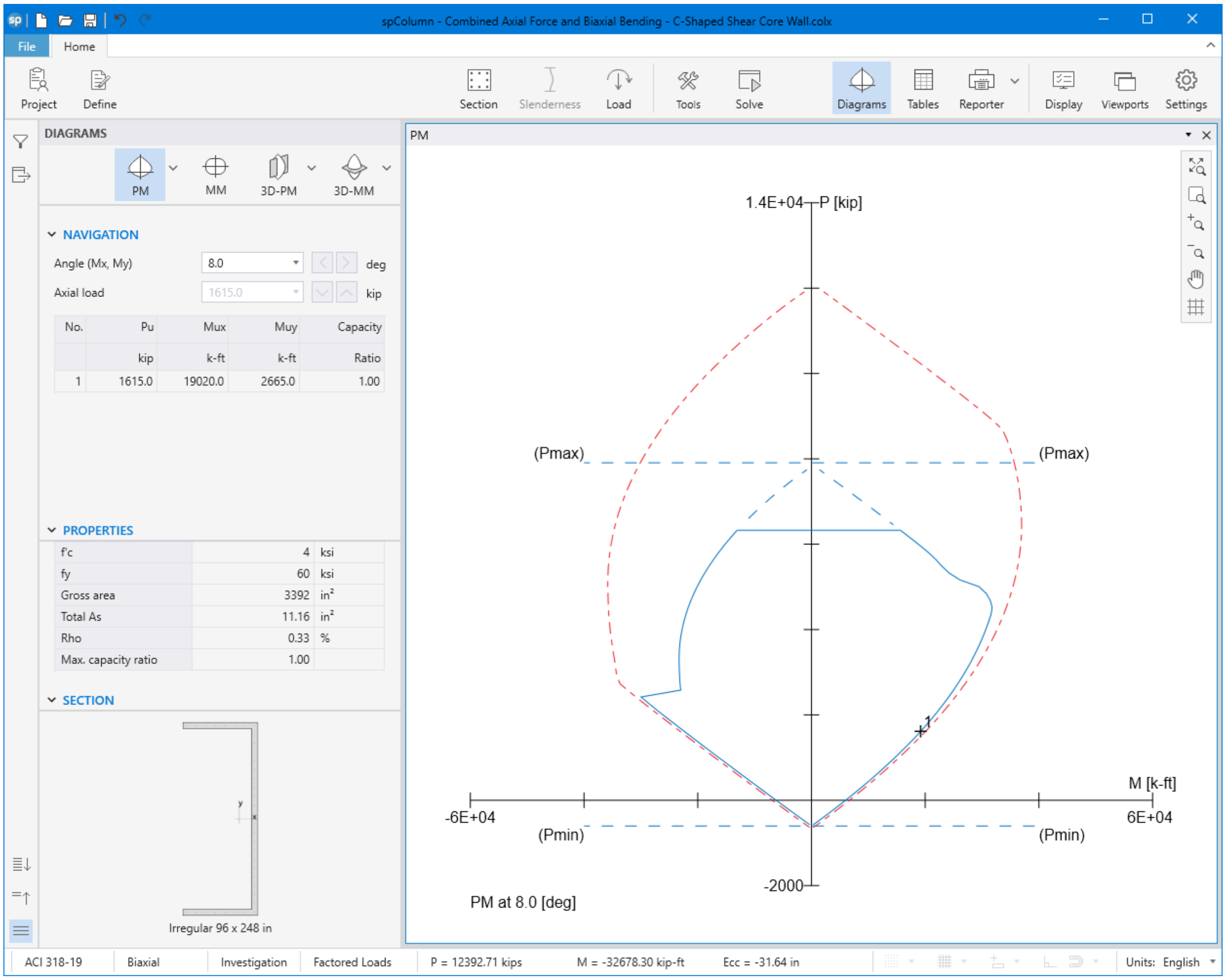
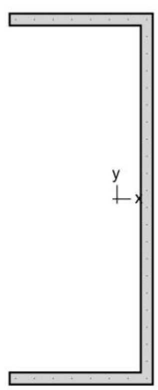


Figure 12 – Core Wall Interaction Diagram at 8º (spColumn)

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



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List of Figures

Figure 1: Column section 4

1. General Information

File Name	...\Combined Axial Force and Biaxial Bending -...
Project	C-Shaped Shear Core Wall
Column	Shear Wall
Engineer	SP
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	4	ksi
E_c	3605	ksi
f_c	3.4	ksi
ε_u	0.003	in/in
β_1	0.85	

2.2. Steel

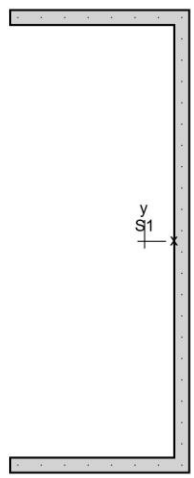
Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{ty}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	3392	in²
I_x	3.04514e+007	in⁴
I_y	2.81666e+006	in⁴
r_x	94.7492	in
r_y	28.8164	in
X_o	0	in
Y_o	0	in

3.2. Section Figure



Irregular 96 x 248 in   0.33% reinf.

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	-72.1	-124.0	2	-72.1	-116.0	3	15.9	-116.0
4	15.9	116.0	5	-72.1	116.0	6	-72.1	124.0
7	23.9	124.0	8	23.9	-124.0			

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
#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	11.16 in²
Rho	0.33 %
Minimum clear spacing	10.80 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	-68.0	120.0	0.31	-55.4	120.0	0.31	-42.9	120.0
0.31	-30.3	120.0	0.31	-17.7	120.0	0.31	-5.1	120.0
0.31	7.4	120.0	0.31	20.0	120.0	0.31	20.0	108.6
0.31	20.0	97.1	0.31	20.0	85.7	0.31	20.0	74.3
0.31	20.0	62.9	0.31	20.0	51.4	0.31	20.0	40.0
0.31	20.0	28.6	0.31	20.0	17.1	0.31	20.0	5.7
0.31	20.0	-5.7	0.31	20.0	-17.1	0.31	20.0	-28.6
0.31	20.0	-40.0	0.31	20.0	-51.4	0.31	20.0	-62.9
0.31	20.0	-74.3	0.31	20.0	-85.7	0.31	20.0	-97.1
0.31	20.0	-108.6	0.31	20.0	-120.0	0.31	7.4	-120.0
0.31	-5.1	-120.0	0.31	-17.7	-120.0	0.31	-30.3	-120.0
0.31	-42.9	-120.0	0.31	-55.4	-120.0	0.31	-68.0	-120.0

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	7906.9	0.00	15.21	786.22	244.00	-0.00207	0.65000	
X	@ Allowable comp.	6325.5	15600.15	2963.39	283.73	244.00	-0.00042	0.65000	
X	@ f_s = 0.0	5461.0	23072.38	2510.95	244.00	244.00	0.00000	0.65000	
X	@ f_s = 0.5 f_y	4418.2	28203.11	956.23	181.44	244.00	0.00103	0.65000	
X	@ Balanced point	3756.7	29554.26	34.44	144.41	244.00	0.00207	0.65000	
X	@ Tension control	3931.1	38847.58	-2062.80	90.72	244.00	0.00507	0.90000	*
X	@ Pure bending	0.0	6175.09	-1231.36	2.40	244.00	0.30236	0.90000	
X	@ Max tension	-602.6	0.02	-22.32	0.00	244.00	9.99999	0.90000	
Y	@ Max compression	7906.9	0.00	15.20	296.20	91.92	-0.00207	0.65000	
Y	@ f_s = 0.0	7204.8	0.00	3635.83	91.92	91.92	0.00000	0.65000	
Y	@ f_s = 0.5 f_y	6450.0	0.00	6415.76	68.35	91.92	0.00103	0.65000	
Y	@ Allowable comp.	6325.5	0.00	6764.13	64.58	91.92	0.00127	0.65000	
Y	@ Balanced point	5984.4	-0.01	7575.08	54.40	91.92	0.00207	0.65000	
Y	@ Tension control	7340.1	0.00	11685.62	34.18	91.92	0.00507	0.90000	*
Y	@ Pure bending	0.0	0.00	1159.23	0.93	91.92	0.29218	0.90000	
Y	@ Max tension	-602.6	0.00	-22.32	0.00	91.92	9.99999	0.90000	
-X	@ Max compression	7906.9	0.03	15.21	786.22	244.00	-0.00207	0.65000	
-X	@ Allowable comp.	6325.5	-15600.19	2963.40	283.73	244.00	-0.00042	0.65000	
-X	@ f_s = 0.0	5461.0	-23072.38	2510.95	244.00	244.00	0.00000	0.65000	
-X	@ f_s = 0.5 f_y	4418.2	-28203.10	956.23	181.44	244.00	0.00103	0.65000	
-X	@ Balanced point	3756.7	-29554.28	34.44	144.41	244.00	0.00207	0.65000	
-X	@ Tension control	3931.1	-38847.56	-2062.80	90.72	244.00	0.00507	0.90000	*

About	Point	P	X-Moment	Y-Moment	NA Depth	d_t Depth	ε_t	ϕ
		kip	k-ft	k-ft	in	in		
-X	@ Pure bending	0.0	-6175.10	-1231.37	2.40	244.00	0.30236	0.90000
-X	@ Max tension	-602.6	0.00	-22.32	0.00	244.00	9.99999	0.90000
-Y	@ Max compression	7906.9	0.00	15.21	296.69	92.08	-0.00207	0.65000
-Y	@ Allowable comp.	6325.5	-0.04	-2855.26	110.05	92.08	-0.00049	0.65000
-Y	@ f_s = 0.0	2879.4	-0.01	-7977.15	92.08	92.08	0.00000	0.65000
-Y	@ f_s = 0.5 f_y	2002.5	-0.01	-7936.62	68.47	92.08	0.00103	0.65000
-Y	@ Balanced point	1415.2	0.01	-7431.35	54.49	92.08	0.00207	0.65000
-Y	@ Tension control	1011.9	-0.01	-7691.36	34.23	92.08	0.00507	0.90000
-Y	@ Pure bending	0.0	-0.01	-3365.06	12.52	92.08	0.01907	0.90000
-Y	@ Max tension	-602.6	0.00	-22.32	0.00	92.08	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_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	NA Depth	ε_t	ϕ	Ratio
	kip	k-ft	k-ft	kip	k-ft	k-ft	in			
1	1615.00	19020.00	2665.00	1615.00	19027.49	2666.05	36.12	0.01374	0.900	1.00

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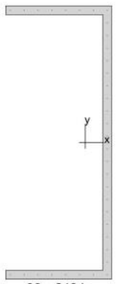
7. Diagrams

7.1. PM at θ=8 [deg]



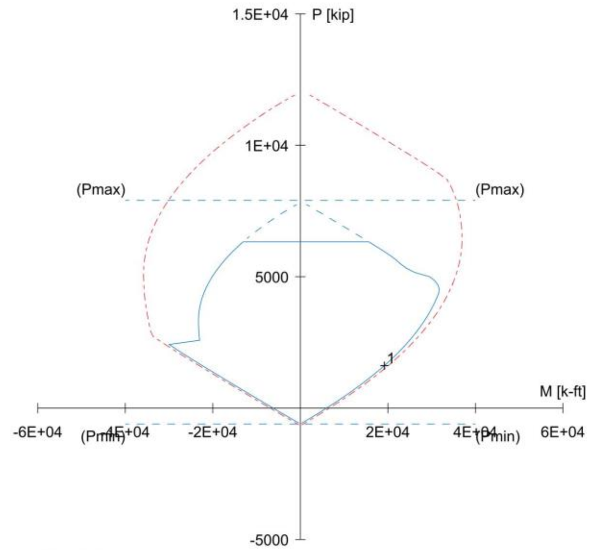
96 x 248 in

General Information	
Project	C-Shaped S... Core Wall
Column	Shear Wall
Engineer	SP
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	4 ksi
E_c	3605 ksi
f_y	60 ksi
E_s	29000 ksi

Section	
Type	Irregular
A_g	3392 in²
I_x	3.04514e+007 in⁴
I_y	2.81666e+006 in⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	11.16 in²
Rho	0.33 %
Min. clear spacing	10.80 in

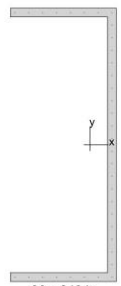


PM at 8.0 [deg]

No.	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	Capacity
	kip	k-ft	k-ft	kip	k-ft	k-ft	Ratio
1	1615.0	19020.0	2665.0	1615.00	19027.49	2666.05	1.00

Max. Capacity Ratio: 1.00

7.2. MM at P=1615 [kip]



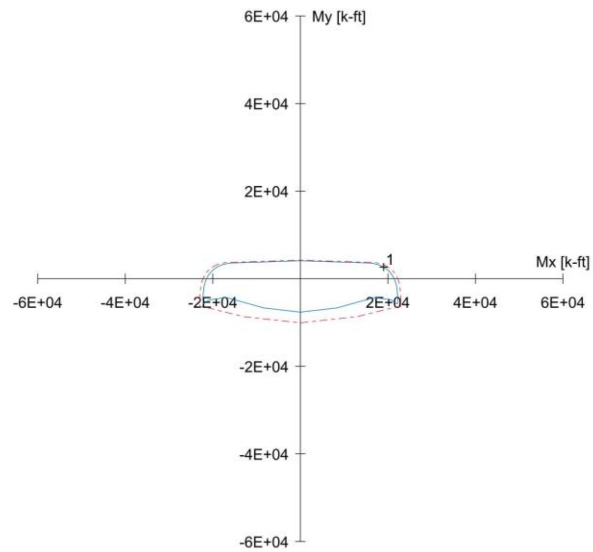
96 x 248 in

General Information	
Project	C-Shaped S... Core Wall
Column	Shear Wall
Engineer	SP
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	4 ksi
E_c	3605 ksi
f_y	60 ksi
E_s	29000 ksi

Section	
Type	Irregular
A_g	3392 in²
I_x	3.04514e+007 in⁴
I_y	2.81666e+006 in⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	11.16 in²
Rho	0.33 %
Min. clear spacing	10.80 in



MM at P=1615.0 [kip]

No.	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	Capacity
	kip	k-ft	k-ft	kip	k-ft	k-ft	Ratio
1	1615.0	19020.0	2665.0	1615.00	19027.49	2666.05	1.00

Max. Capacity Ratio: 1.00

3. Summary of Design Results

3.1. Comparison of Results by Method

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

Table 2 - Comparison of Results		
Parameter	Hand	spColumn
c, in.	36.12	36.12
α, degrees	120	120
d_1, in.	201.56	201.56
ε_{s1}, in./in.	0.01374	0.01374
ϕP_n, kip	1614.65	1615.00
ϕM_{nx}, kip-ft	19025.34	19027.49
ϕM_{ny}, kip-ft	2665.45	2666.05

3.2. spColumn Interaction Diagram Results Export

spColumn allows the user to export results data of the following:

1. The column section can be exported to a file in Drawing Exchange Format (DXF) format that is readable by most CAD programs like AutoCAD.
2. Loads data can be exported to a Delimited Text file (TXT).
3. Points from the (nominal or factored) interaction diagram or 3D failure surface can be exported to a Comma-Separated Values (CSV) file or to a Tab Delimited Text file (TXT). These files can be read by most spreadsheet and mathematical programs where data produced by spColumn can be further analyzed and processed as needed by the user. Coordinates of the points (P, M_x, M_y) are saved together with corresponding location of the neutral axis (depth and angle), maximum steel strain, and (for ACI code) strength reduction factor.

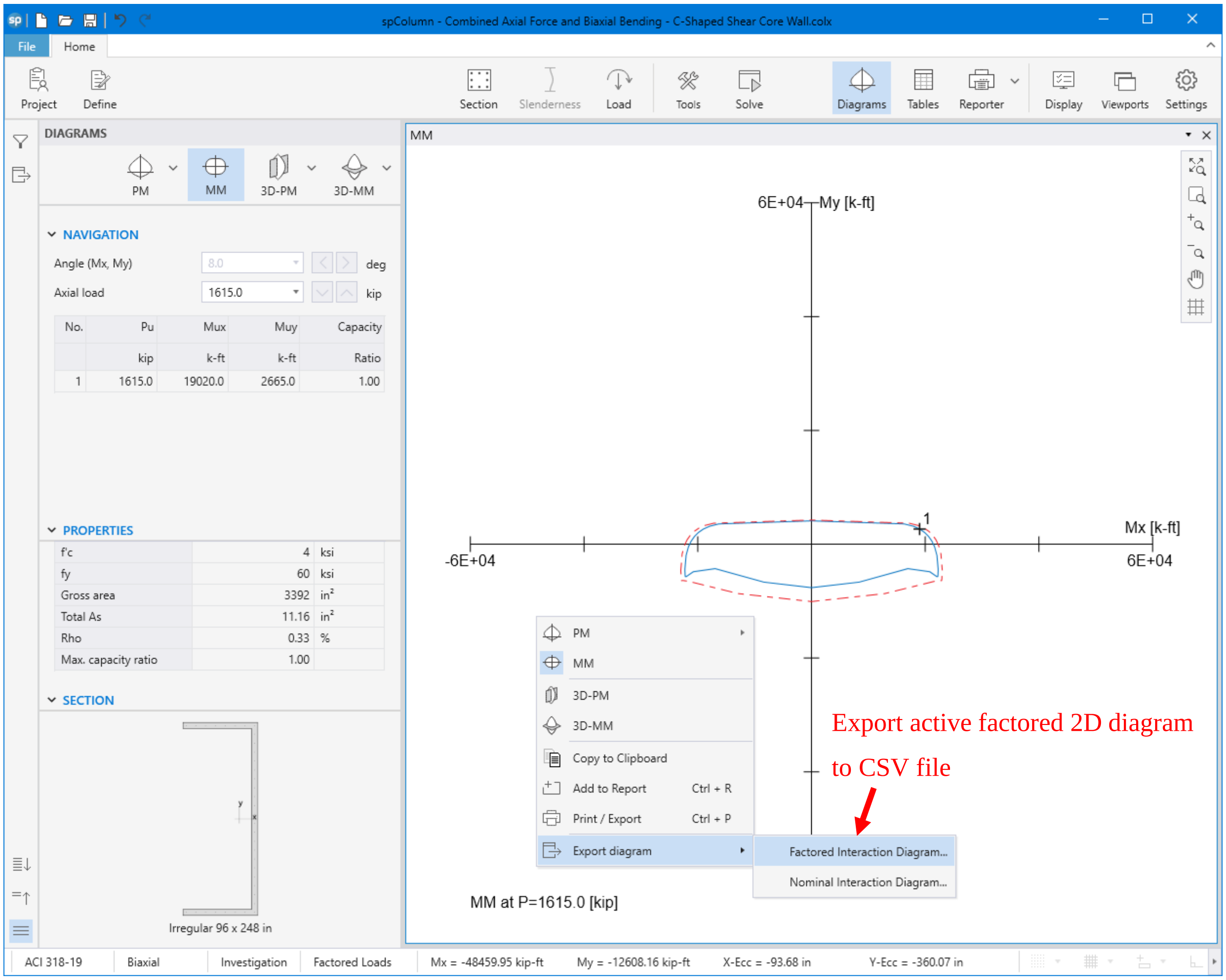


Figure 13 – Exporting Data to CSV file (spColumn)

Table 3 - ϕM_{nx} - ϕM_{ny} Diagram at ϕP_n = 1615 kip (Sample Results of spColumn Export)							
ϕP_n, kip	ϕM_{nx}, kip-ft	ϕM_{ny}, kip-ft	c, in.	α, degrees	d_1, in.	ε_1, in./in.	ϕ
1615	22277.1	-4470.2	7.99	0	244.00	0.08865	0.90
1615	22069.2	-1994.2	19.58	10	256.26	0.03627	0.90
1615	21651.6	-467.8	29.77	20	260.73	0.02327	0.90
1615	21150.3	571.6	36.28	30	257.27	0.01828	0.90
1615	20568.7	1389.4	39.46	40	246.00	0.01570	0.90
1615	19874.0	2079.6	39.40	50	227.26	0.01430	0.90
1615	19030.2	2664.7	36.13	60	201.61	0.01374	0.90
1615	17939.8	3160.1	29.48	70	169.83	0.01428	0.90
1615	16322.4	3567.3	19.19	80	132.90	0.01778	0.90
1615	0.0	4107.5	3.09	90	91.92	0.08624	0.90
1615	-16322.4	3567.3	19.19	100	132.90	0.01778	0.90
1615	-17939.8	3160.1	29.48	110	169.83	0.01428	0.90
1615	**-19030.2**	**2664.7**	**36.13**	**120**	**201.61**	**0.01374**	**0.90**
1615	-19874.0	2079.7	39.40	130	227.26	0.01430	0.90
1615	-20568.7	1389.4	39.46	140	246.00	0.01570	0.90
1615	-21150.3	571.6	36.28	150	257.27	0.01828	0.90
1615	-21651.6	-467.8	29.77	160	260.73	0.02327	0.90
1615	-22069.2	-1994.2	19.58	170	256.26	0.03627	0.90
1615	-22277.1	-4470.2	7.99	180	244.00	0.08865	0.90
1615	-22225.7	-5586.4	21.56	190	256.28	0.03267	0.90
1615	-22188.5	-5688.4	37.17	200	260.78	0.01805	0.90
1615	-22155.7	-5707.6	51.68	210	257.35	0.01194	0.90
1615	-22111.5	-5705.9	64.40	220	246.10	0.00846	0.90
1615	-22041.2	-5694.5	74.89	230	227.37	0.00611	0.90
1615	-20856.1	-4886.7	89.71	240	201.74	0.00375	0.79
1615	-16945.2	-4287.2	102.17	250	169.98	0.00199	0.65
1615	-8382.6	-6663.6	81.53	260	133.05	0.00190	0.65
1615	0.0	-7631.8	59.03	270	92.08	0.00168	0.65
1615	8382.6	-6663.6	81.53	280	133.05	0.00190	0.65
1615	16945.2	-4287.2	102.17	290	169.98	0.00199	0.65
1615	20856.1	-4886.7	89.71	300	201.74	0.00375	0.79
1615	22041.2	-5694.5	74.89	310	227.37	0.00611	0.90
1615	22111.5	-5705.9	64.40	320	246.10	0.00846	0.90
1615	22155.7	-5707.6	51.68	330	257.35	0.01194	0.90
1615	22188.5	-5688.4	37.17	340	260.78	0.01805	0.90
1615	22225.7	-5586.4	21.56	350	256.28	0.03267	0.90
1615	22277.1	-4470.2	7.99	0	244.00	0.08865	0.90

Other tables can be exported for other load points or for all load points as needed by the user.

4. 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 most building design calculations, such as the examples shown for flat plate or flat slab concrete floor systems, all building columns may be subjected to biaxial bending (M_x and M_y) due to lateral effects and unbalanced moments from both directions of analysis. This requires an investigation of the column P-M_x-M_y interaction diagram in two directions simultaneously (axial force interaction with biaxial bending).

This example shows the calculations needed to obtain one point on the three-dimensional failure surface (biaxial M_x-M_y interaction diagram). Generating the three-dimensional failure surface (interaction diagram) for a column or wall section subjected to a combined axial force and biaxial bending moments is tedious and challenging for engineers and the use of a computer aid can save time and eliminate errors. StucturePoint's spColumn program can, quickly, simply and accurately generate the 3D failure surface (interaction diagram) for all commonly encountered column, beam or wall sections in addition to complex and irregular cross-sections. Following figure shows the 3D representation of the complete Nominal and Factored failure surfaces for the C-shaped core wall in this example.

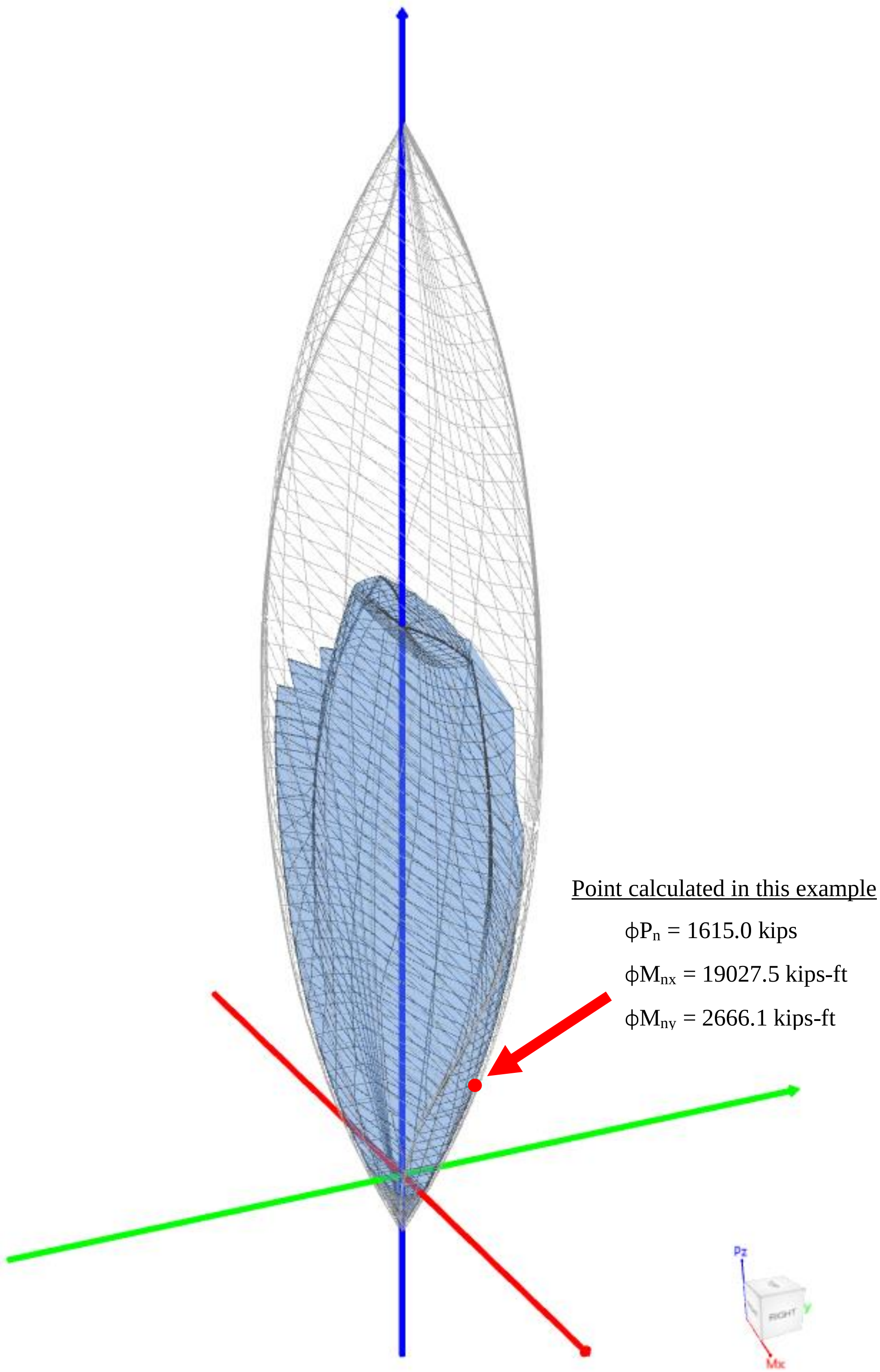


Figure 14 – Interaction Diagram in Two Directions (Biaxial) (spColumn)

The spColumn “Diagrams” module is a powerful tool especially for investigating interaction diagrams (failure surfaces) for columns and walls sections subjected to a combined axial force and biaxial bending moments. The module allows the user to view and analyze 2D interaction diagrams and contours along with 3D failure surfaces in a multi viewport environment. The following figure shows three views of:

1. M_x-M_y interaction diagram cut at axial load of 1615 kip in compression
2. P-M interaction diagram cut at angle of 8º
3. A 3D failure surface (interaction diagram) showing the points calculated in this example.

Figures 16 and 17 show 3D visualization of failure surface with a horizontal and vertical plane cut, respectively.

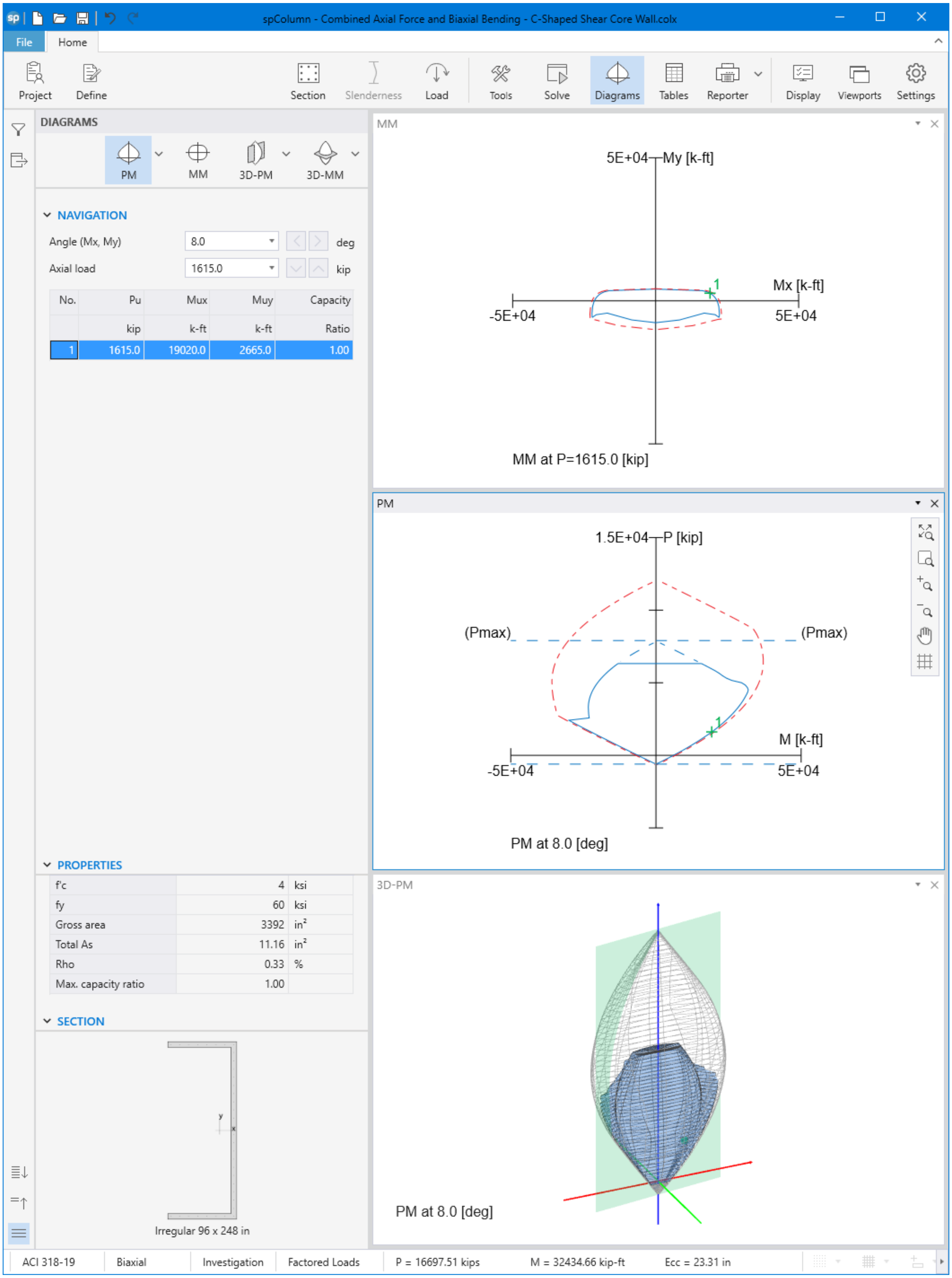


Figure 15 – Diagrams Module (spColumn)

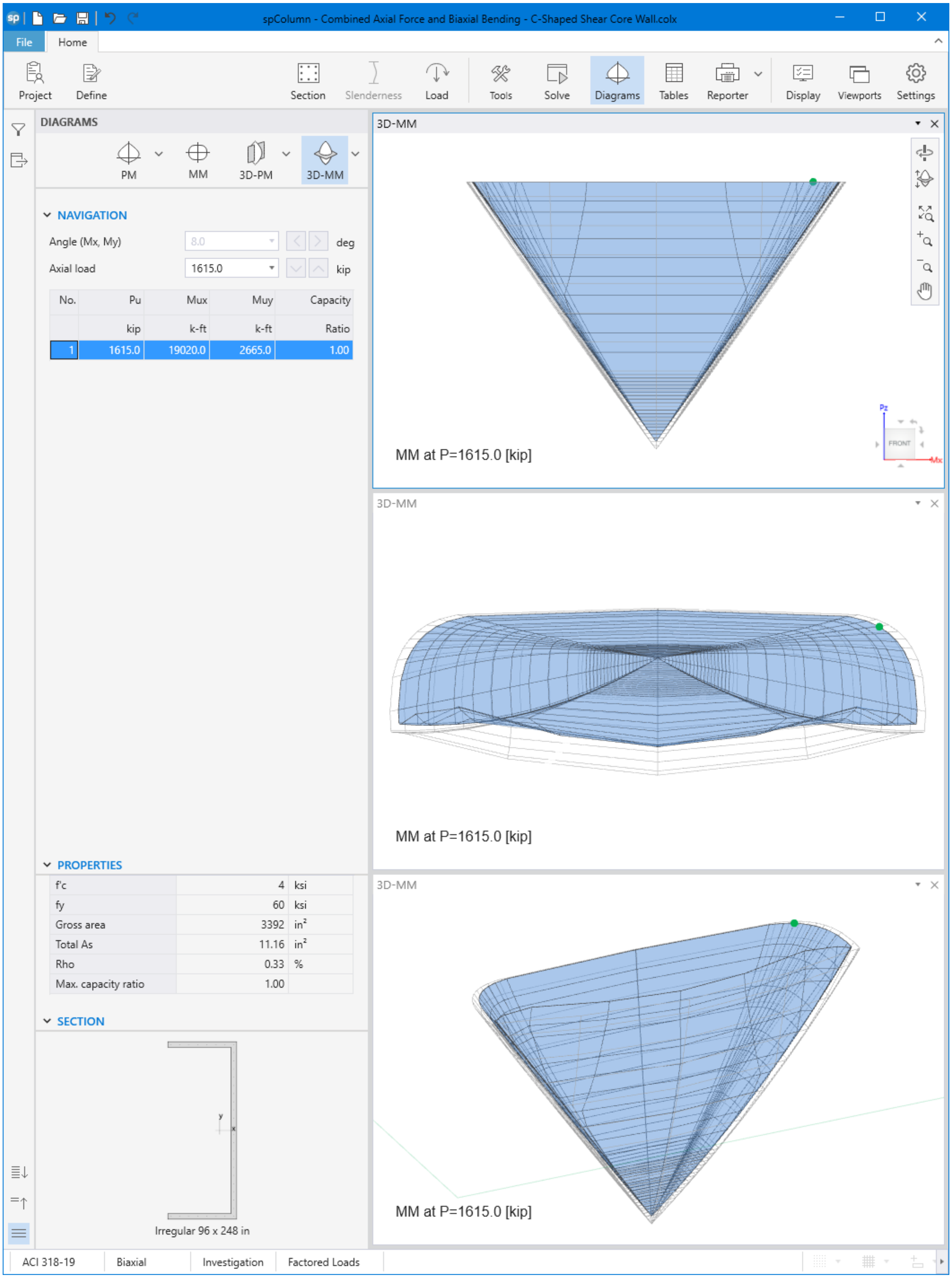


Figure 16 – 3D Failure Surface with a Horizontal Plane Cut at P = 1615 kip (spColumn)

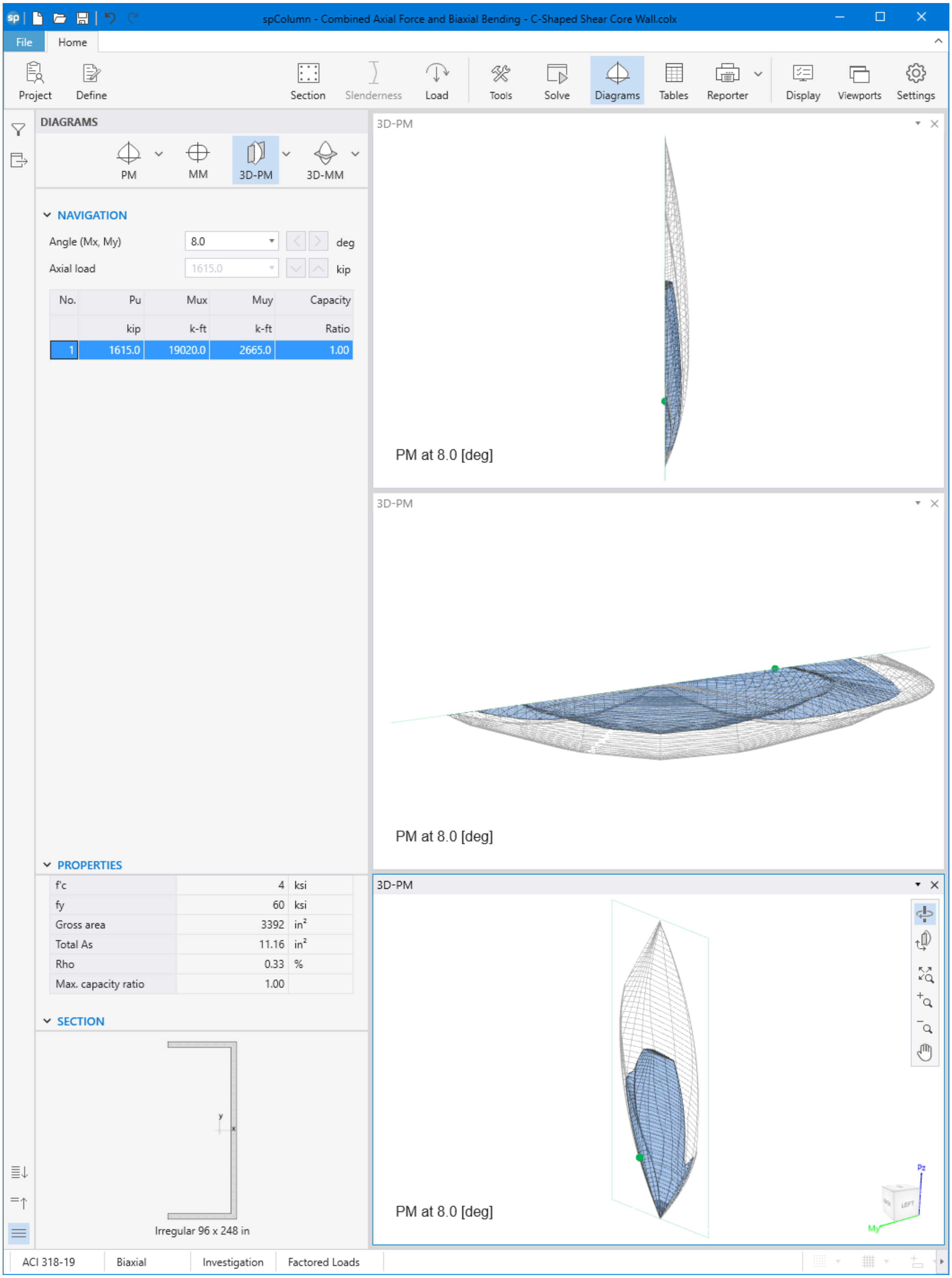


Figure 17 – 3D Failure Surface with a Vertical Plane Cut at 8º (spColumn)