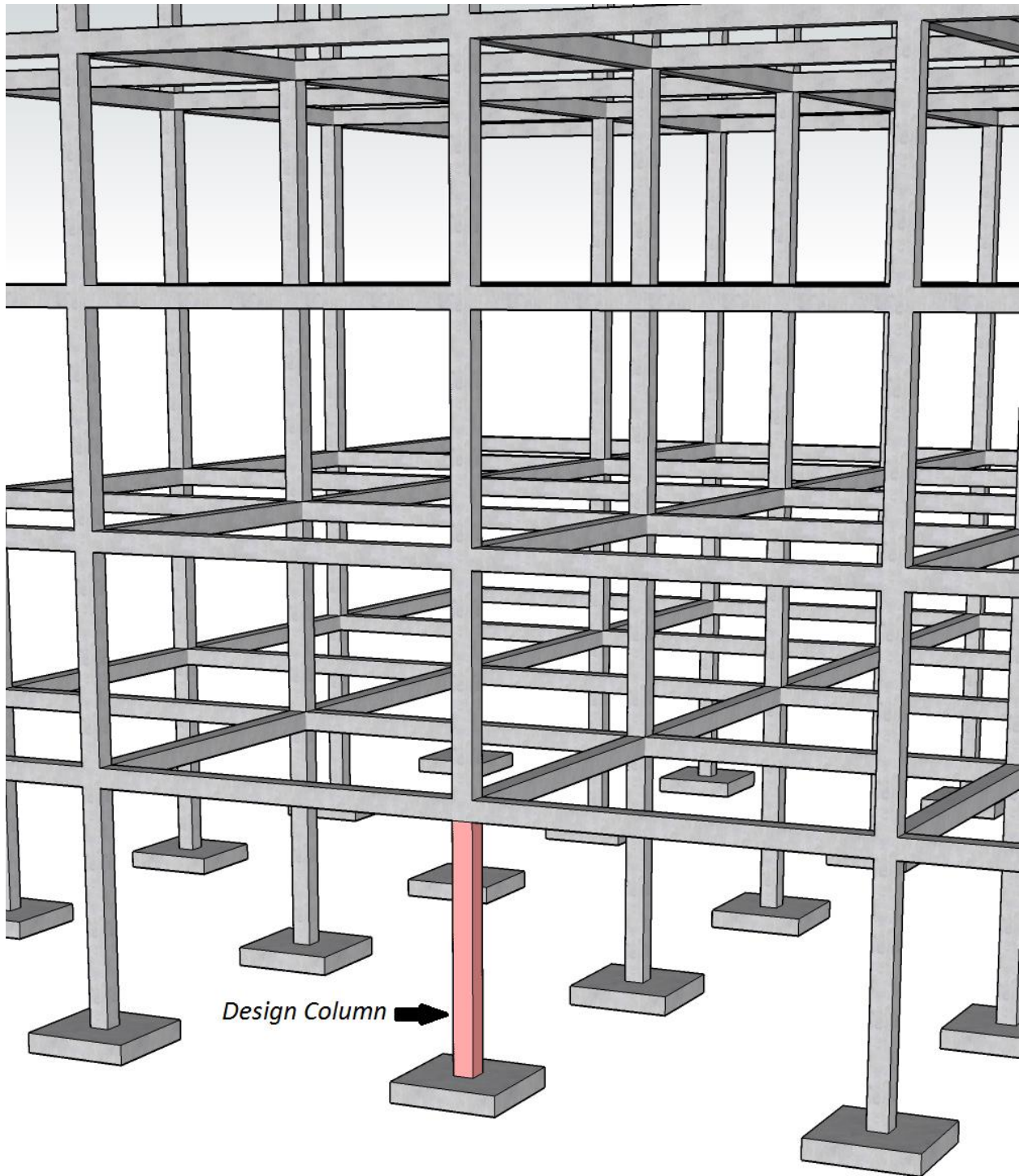**Slender Column Design in Non-Sway Frame - Moment Magnification Method (ACI 318-19)**

Design Column →

Slender Column Design in Non-Sway Frame - Moment Magnification Method (ACI 318-19)

Evaluate slenderness effects for columns in a non-sway multistory reinforced concrete frame by determining the adequacy of the square tied column shown below, which is an exterior first floor column. The design forces obtained from a first-order analysis are provided in the design data section below. The story height is 12 ft. it is assumed that the frame is braced sufficiently to prevent relative translation of its joints. Assume 40% of the factored axial load is sustained. Compare the calculated results with the values presented in the Reference and with exact values from spColumn engineering software program from StructurePoint.
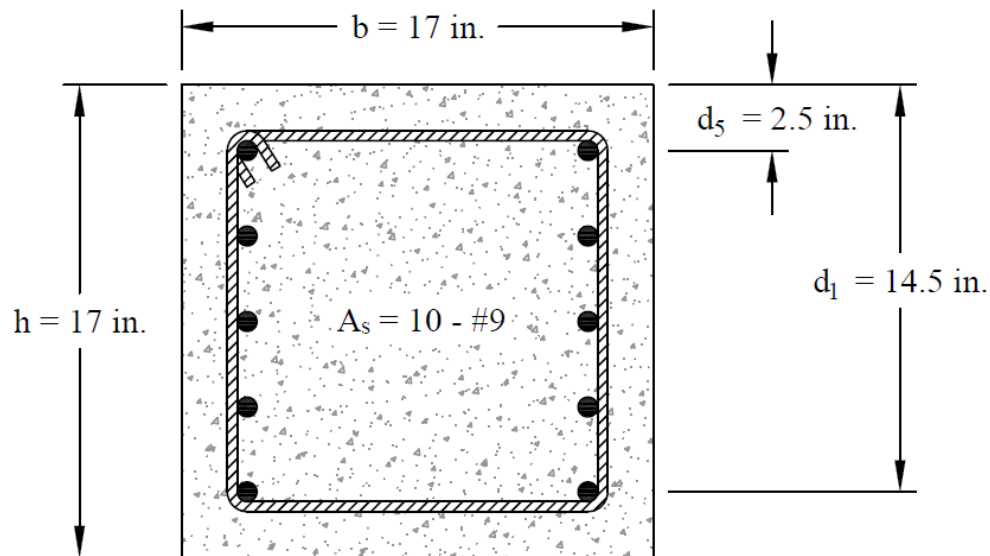

Figure 1 – Reinforced Concrete Column Cross-Section

Contents

Code

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

References

- Reinforced Concrete Design, 9th Edition, 2021, Pincheira J. et. al., Oxford University Press, Example 13.17.3.

- spColumn Engineering Software Program Manual v10.00, STRUCTUREPOINT, 2021

- "Slenderness Effects for Concrete Columns in Sway Frame – Moment Magnification Method (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022

- "Slender Concrete Column Design in Sway Frames – Moment Magnification Method (ACI 318-19)" Design Example, STRUCTUREPOINT, 2022

Design Data

Concrete: $f_c' = 3000$ psi

Steel: $f_y = 60000$ psi

Beams: h = 24 in., b = 14 in., l = 30 ft

Columns: h = 17 in., b = 17 in. H = 12 ft

factored design forces obtained from first-order analysis from the reference:

Table 1 - Column factored loads			
Load Case	Axial Load, kip	Bending Moment, kip-ft	
		Top	Bottom
Factored Load	525[*]	105	0
[*] Assume 40% of the axial load is sustained			

1. Slenderness Effects and Sway or Non-sway Frame Designation

Columns and stories in structures are considered as non-sway frames if the stability index for the story (Q) does not exceed 0.05.   ***ACI 318-19 (6.6.4.3)***

The reference assumed that the frame is a non-sway frame since Q value is less than 0.05.

2. Determine Slenderness Effects

The reference decided to be consistent with the more conservative procedure provided by ***ACI 318-19 (6.6.4.4.3)*** by taking k value equals to 1.0. However, the k value, in this example, is calculated based on the exact procedure for illustration purposes.

$$I_{column} = 0.7 \times \frac{c^4}{12} = 0.7 \times \frac{17^4}{12} = 4872 \text{ in.}^4$$ ***ACI 318-19 (6.6.3.1.1(a))***

$$I_{beam} = 0.35 \times \frac{b \times h^3}{12} = 0.35 \times \frac{14 \times 24^3}{12} = 5645 \text{ in.}^4$$ ***ACI 318-19 (6.6.3.1.1(a))***

$$E = 57,000 \times \sqrt{f_c'} = 57,000 \times \sqrt{3000} = 3122 \text{ ksi}$$ ***ACI 318-19 (19.2.2.1.b)***

For columns:

$$\frac{E \times I_{column}}{l_c} = \frac{3122 \times 4872}{12 \times 12} = 8.8 \times 10^3 \text{ kip.ft}$$

For beams framing into the columns:

$$\frac{E \times I_{beam}}{l_b} = \frac{3122 \times 5645}{30 \times 12} = 4.08 \times 10^3 \text{ kip.ft}$$

$$\Psi_A = \frac{\left(\sum \dfrac{EI}{l_c}\right)_{columns}}{\left(\sum \dfrac{EI}{l}\right)_{beams}} = \frac{2 \times 8.80 \times 10^3}{4.08 \times 10^3} = 4.32$$ ***ACI 318-19 (Figure R6.2.5.1)***

$\Psi_B = \infty$ (Column was assumed hinged at base)   ***ACI 318-19 (Figure R6.2.5.1)***

Using Figure R6.2.5.1 from ACI 318-19 → $k = 0.959$ as shown in the figure below for the exterior columns with one beam framing into them in the directions of analysis.
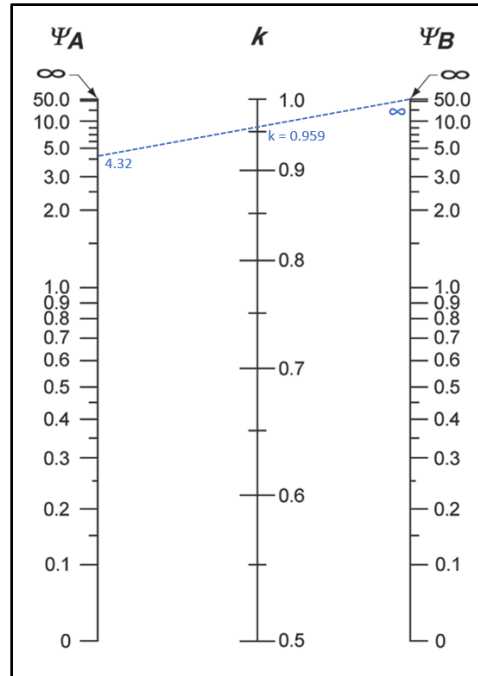
Figure 2 – Effective Length Factor (k) (Non-Sway Frame)

ACI 318-19 allows to neglect the slenderness in a non-sway frame if:

$$\frac{k \times l_u}{r} \leq 34 + 12\left(\frac{M_1}{M_2}\right)$$

ACI 318-19 (6.2.5.1b)

Where:

$$r = \text{radius of gyration} = (a) \sqrt{\frac{I_g}{A_g}} \quad or \quad (b)\ 0.3 \times c_1$$

ACI 318-19 (6.2.5.2)

$$r = \sqrt{\frac{I_g}{A_g}} = \sqrt{\frac{c^2}{12}} = \sqrt{\frac{17^2}{12}} = 4.91\,\text{in.}$$

$$\frac{0.959 \times (12 \times 12 - 24)}{4.91} = 23.45 < 34 - 12\left(\frac{0}{105}\right) = 34 \qquad \therefore \text{slenderness can be neglected.}$$

Even though it is not required to consider slenderness effects for this column, the moment magnification method will be shown for illustration.

3. Moment Magnification – Non-Sway Frame

$$M_{c2} = \delta M_2$$

ACI 318-19 (6.6.4.5.1)

Where:

$$\delta = \text{magnification factor} = \frac{C_m}{1 - \dfrac{P_u}{0.75P_c}} \geq 1.0$$

ACI 318-19 (6.6.4.5.2)

$$P_c = \frac{\pi^2 (EI)_{eff}}{(kl_u)^2}$$

ACI 318-19 (6.6.4.4.2)

Where:

$$(EI)_{eff} = \begin{cases} \text{(a)} & \dfrac{0.4 E_c I_g}{1 + \beta_{dns}} \\[2mm] \text{(b)} & \dfrac{0.2 E_c I_g + E_s I_{se}}{1 + \beta_{dns}} \\[2mm] \text{(c)} & \dfrac{E_c I}{1 + \beta_{dns}} \end{cases}$$

ACI 318-19 (6.6.4.4.4)

There are three options for calculating the effective flexural stiffness of slender concrete columns $(EI)_{eff}$. The second equation provides accurate representation of the reinforcement in the section and will be used in this example and is also used by the solver in spColumn. Further comparison of the available options is provided in "Effective Flexural Stiffness for Critical Buckling Load of Concrete Columns" technical note.

3.1. Calculation of Critical Load (P_c)

$$I_{column} = \frac{c^4}{12} = \frac{17^4}{12} = 6960 \text{ in.}^4$$

ACI 318-19 (Table 6.6.3.1.1(a))

$$E_c = 57,000 \times \sqrt{f_c'} = 57,000 \times \sqrt{3000} = 3122 \text{ ksi}$$

ACI 318-19 (19.2.2.1.b)

β_{dns} is the ratio of maximum factored sustained axial load to maximum factored axial load associated with the same load combination.

ACI 318-19 (6.6.4.4.4)

In this example, it is assumed that 40% of the factored axial load is sustained.

$$\beta_{dns} = \frac{P_{u,sustained}}{P_u} = \frac{0.4 \times P_u}{P_u} = 0.40 < 1.00 \quad \rightarrow \quad \therefore \beta_{dns} = 0.40$$

With 10-#9 reinforcement equally distributed on two sides and 17 in. x 17 in. column section → $I_{se} = 360$ in.^{4}.

$$(EI)_{eff} = \frac{0.2 E_c I_g + E_s I_{se}}{1 + \beta_{dns}}$$

ACI 318-19 (6.6.4.4.4(b))

$$\left(EI\right)_{eff} = \frac{0.2 \times 3122 \times 6960 + 29,000 \times 360}{1+0.4} = 10.56 \times 10^6 \text{ kip-in.}^2$$

$$P_c = \frac{\pi^2 \times 10.56 \times 10^6}{\left(0.959 \times (12-2) \times 12\right)^2} = 7871 \text{ kip}$$

3.2. Calculation of Magnified Moment (M_c)

$$C_m = 0.6 - 0.4 \frac{M_1}{M_2} \qquad\qquad\qquad \textit{\textbf{ACI 318-19 (6.6.4.5.3a)}}$$

$$C_m = 0.6 - 0.4 \left(\frac{0}{105}\right) = 0.6$$

$$\delta = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \geq 1.0 \qquad\qquad\qquad \textit{\textbf{ACI 318-19 (6.6.4.5.2)}}$$

$$\delta = \frac{0.6}{1 - \dfrac{525}{0.75 \times 7871}} = 0.66 < 1.00 \rightarrow \delta = 1.00$$

$$M_{min} = P_u \left(0.6 + 0.03h\right) \qquad\qquad\qquad \textit{\textbf{ACI 318-19 (6.6.4.5.4)}}$$

Where P_u = 525 kip, and h = the section dimension in the direction being considered = 17 in.

$$M_{min} = 525 \left(\frac{0.6 + 0.03 \times 17}{12}\right) = 48.56 \text{ kip.ft}$$

$$M_2 = 105 \text{ kip.ft} > M_{2,min} = 48.56 \text{ kip.ft} \rightarrow M_2 = 105 \text{ kip.ft} \qquad\qquad \textit{\textbf{ACI 318-19 (6.6.4.5.4)}}$$

$$M_{c2} = \delta M_2 \qquad\qquad\qquad \textit{\textbf{ACI 318-19 (6.6.4.5.1)}}$$

$$M_{c2} = 1.00 \times 105 = 105 \text{ kip.ft}$$

4. Column Design

Based on the factored axial loads and magnified moments considering slenderness effects, the capacity of the assumed column section (17 in. × 17 in. with 10 – #9 bars distributed on two sides) will be checked and confirmed to finalize the design. A column interaction diagram will be generated using strain compatibility analysis, the detailed procedure to develop column interaction diagram can be found in "Interaction Diagram - Tied Reinforced Concrete Column Design Strength (ACI 318-19)" example.
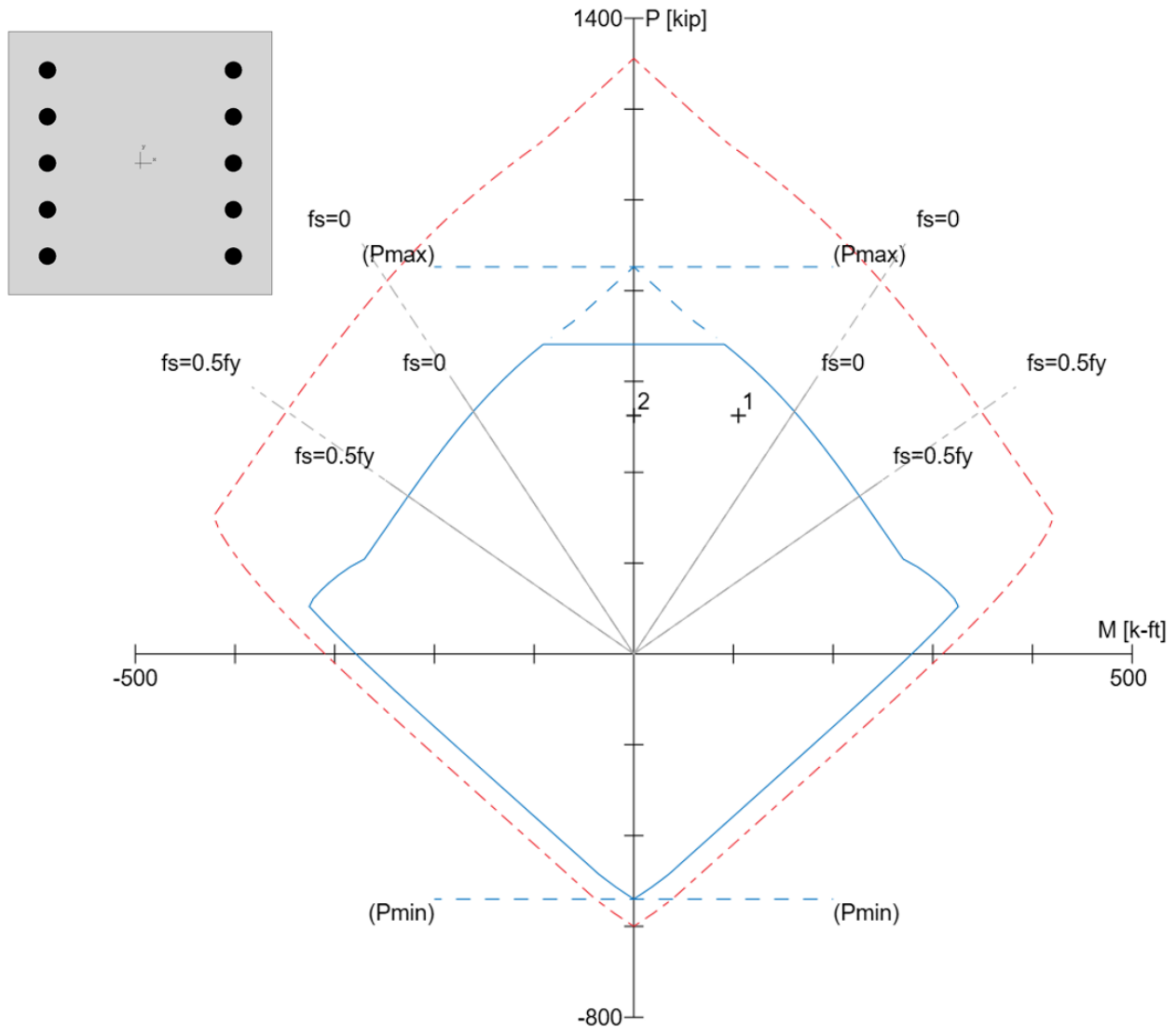
Figure 3 – Column Interaction Diagram

5. Column Design - 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 column section, investigation mode is used, service loads are defined, and slenderness effects are considered using ACI 318-19 provisions.
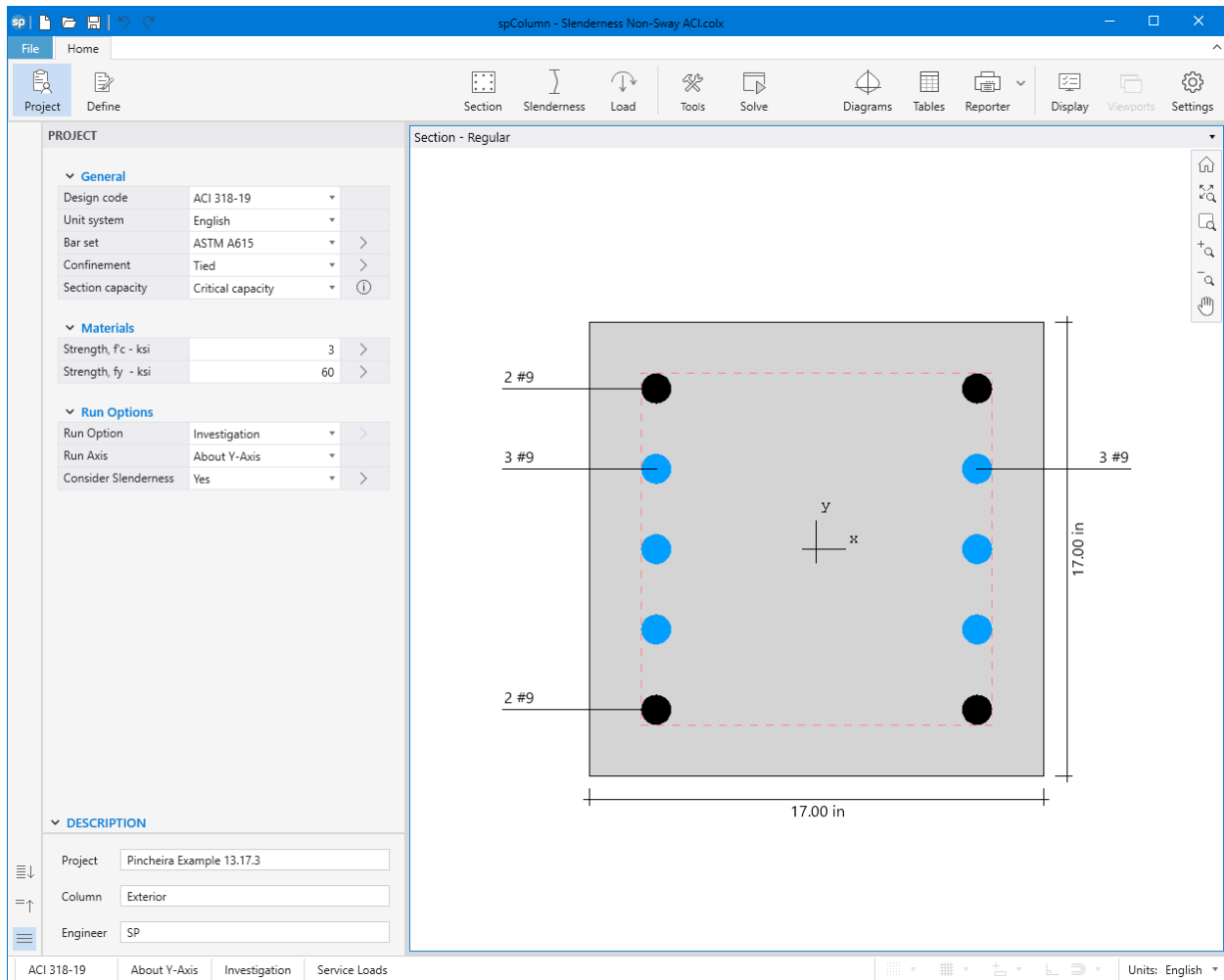
Figure 4 – spColumn Interface
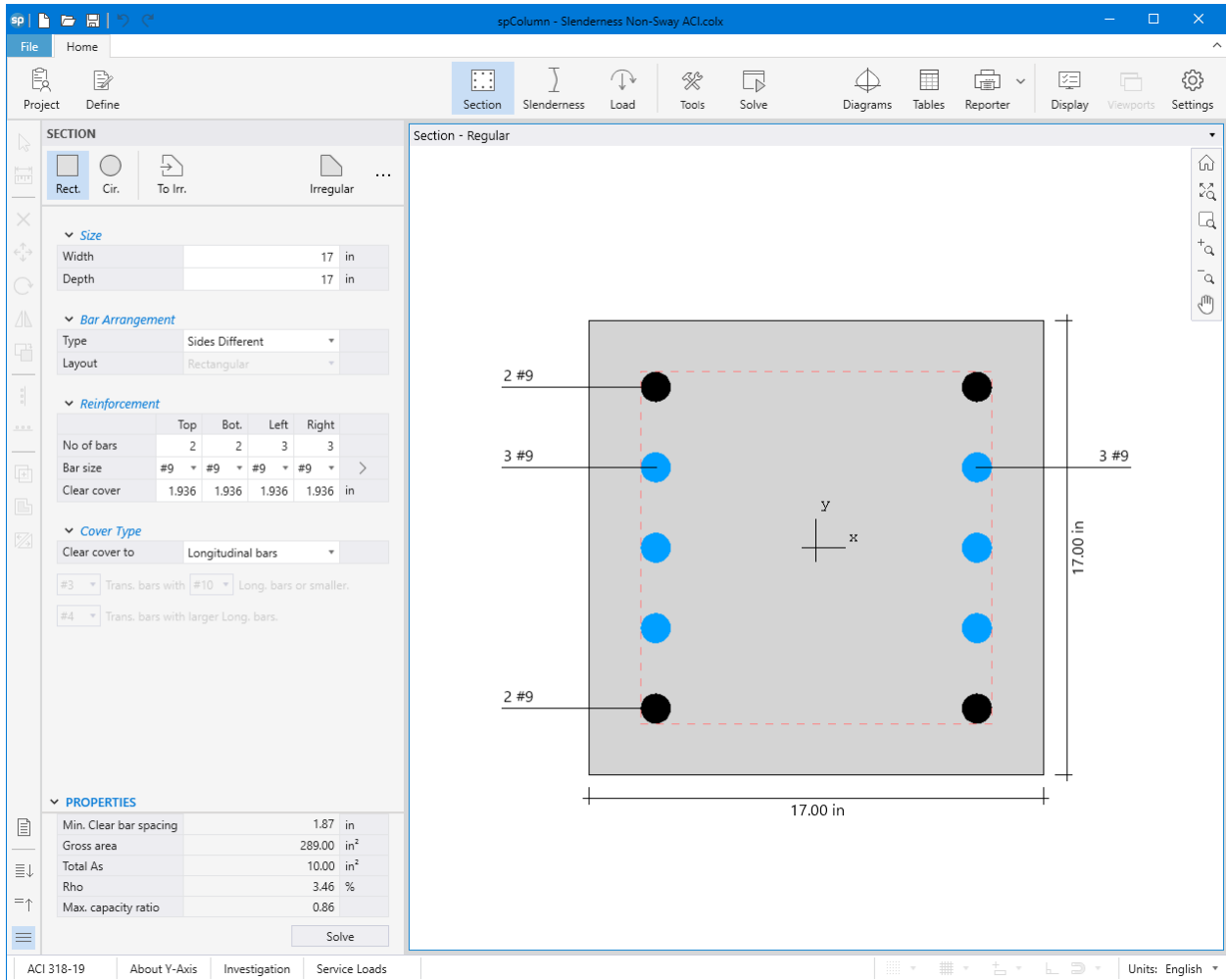
Figure 5 – spColumn Model Editor
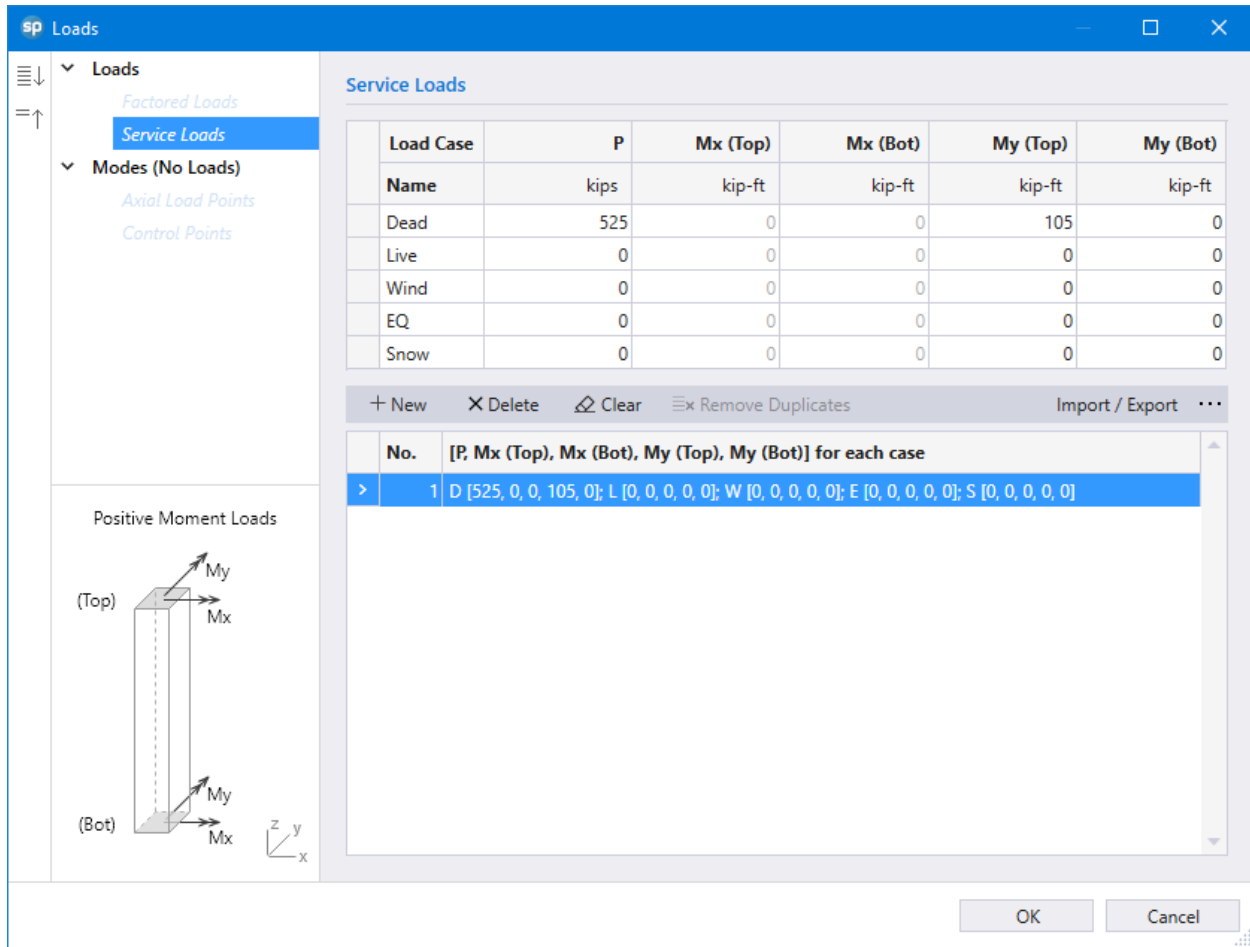
Figure 6 – Defining Loads / Modes (spColumn)
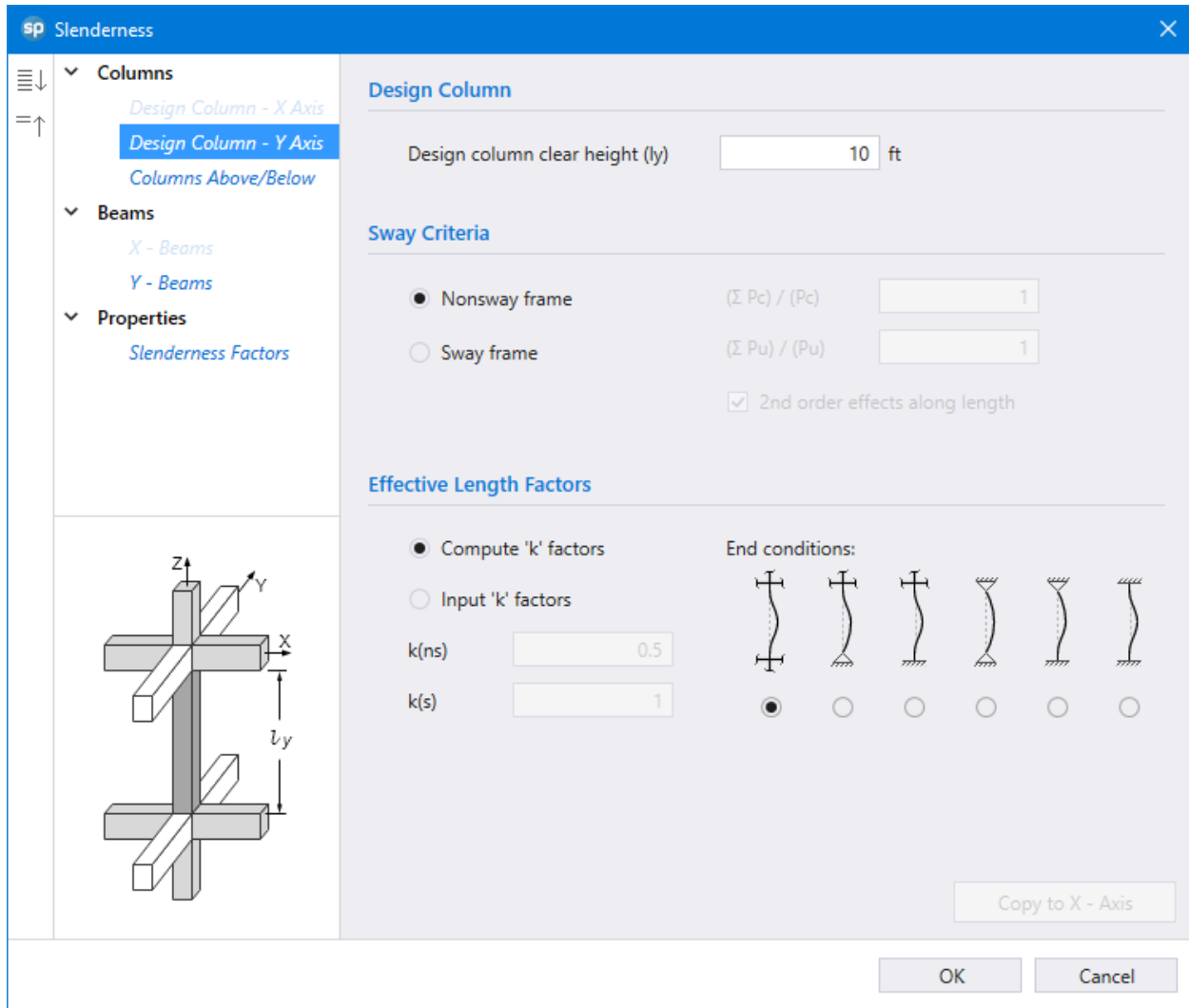
Figure 7 – Defining Slenderness (spColumn)
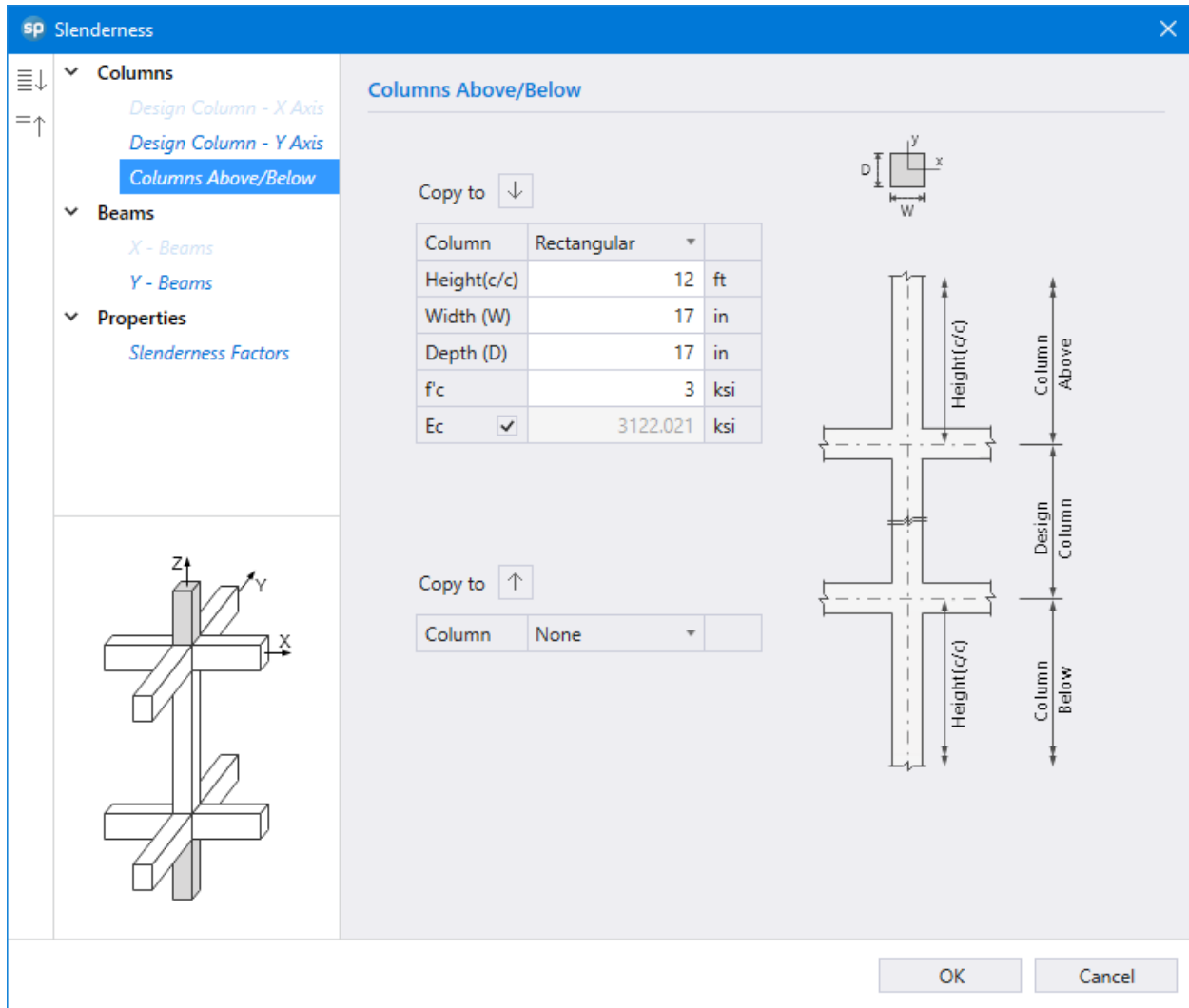
Figure 8 – Defining Columns Above / Below (spColumn)
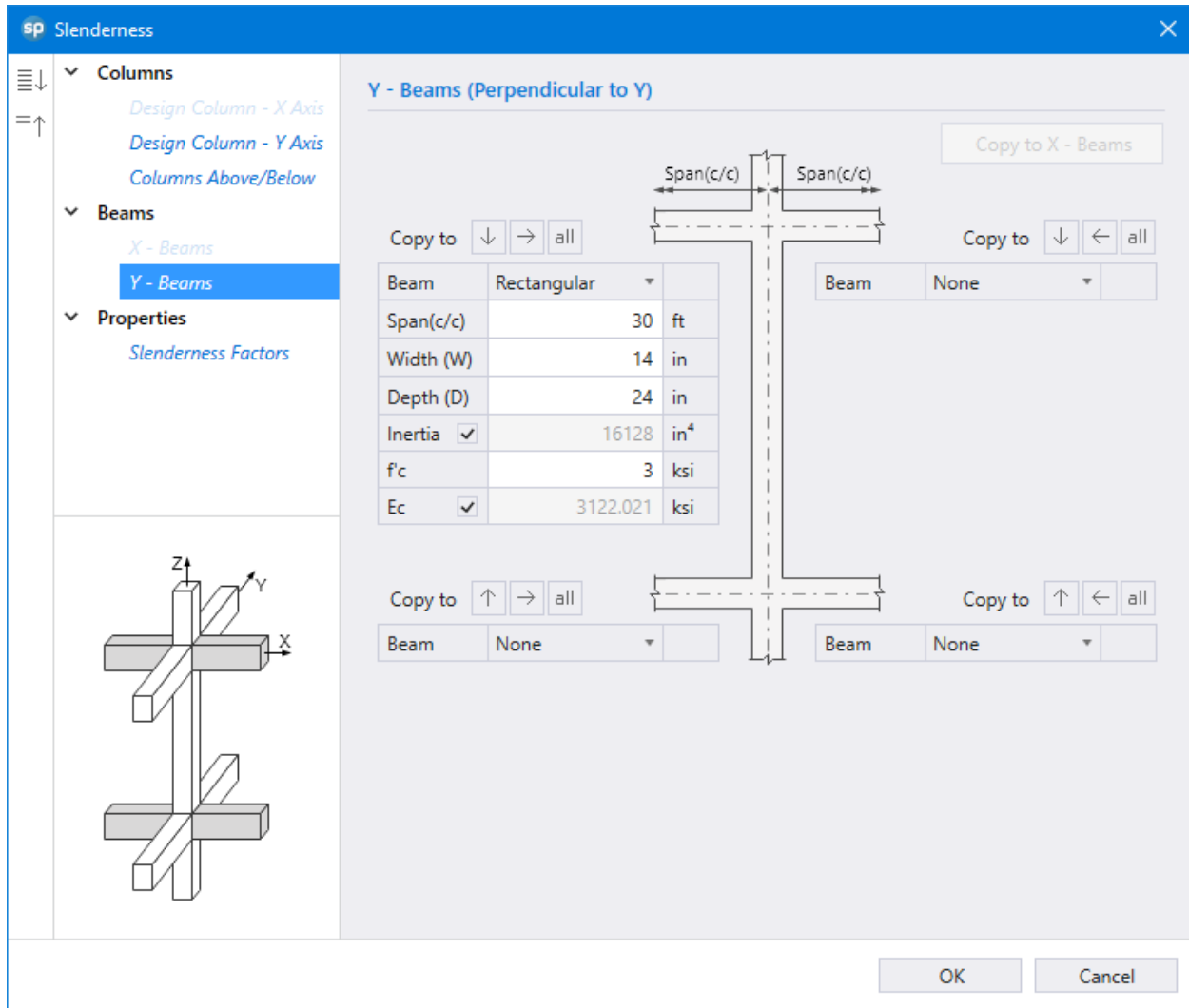
Figure 9 – Defining Beams in Y - Direction (spColumn)

Figure 10 – Column Section Interaction Diagram about Y-Axis (spColumn)

sp**column** (TM)

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

File Name	E:\StructurePoin...\Slenderness Non-Sway ACI.colx
Project	Pincheira Example 13.17.3
Column	Exterior
Engineer	SP
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Y - axis
Slenderness	Considered
Column Type	Structural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	3 ksi
E_c	3122.02 ksi
f_c	2.55 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	Rectangular
Width	17 in
Depth	17 in
A_g	289 in²
I_x	6960.08 in⁴
I_y	6960.08 in⁴
r_x	4.90748 in
r_y	4.90748 in
X_o	0 in
Y_o	0 in

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3.2. Section Figure

Rectangular 17 x 17 in	3.46% reinf.
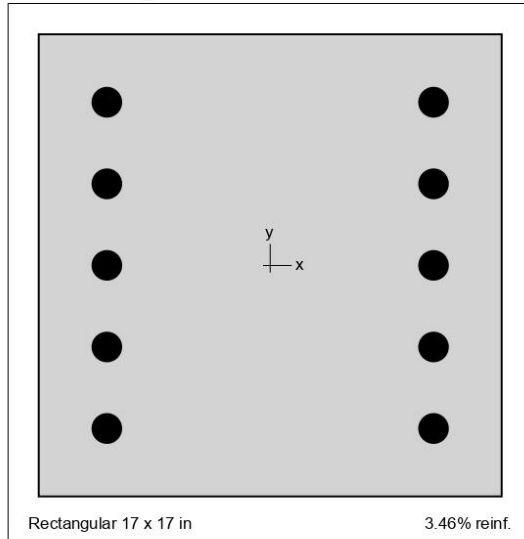
Figure 1: Column section

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	Sides different
Bar layout	Rectangular
Cover to	Longitudal bars
Clear cover	---
Bars	---

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Total steel area, A_s	10.00 in²
Rho	3.46 %
Minimum clear spacing	1.87 in

4.4. Bars Provided

		Bars	Clear cover
			in
Top	2	#9	1.936
Bottom	2	#9	1.936
Left	3	#9	1.936
Right	3	#9	1.936

5. Loading
5.1. Load Cases

Case	Type	Sustained Load
		%
A	Dead	40
B	Live	0
C	Wind	0
D	EQ	0
E	Snow	0

5.2. Load Combinations

Combination	Dead	Live	Wind	EQ	Snow
U1	1.000	0.000	0.000	0.000	0.000

5.3. Service Loads

No.	Load Case	Axial Load	Mx @ Top	Mx @ Bottom	My @ Top	My @ Bottom
		kip	k-ft	k-ft	k-ft	k-ft
1	Dead	525.00	105.00	0.00	105.00	0.00
1	Live	0.00	0.00	0.00	0.00	0.00
1	Wind	0.00	0.00	0.00	0.00	0.00
1	EQ	0.00	0.00	0.00	0.00	0.00
1	Snow	0.00	0.00	0.00	0.00	0.00

6. Slenderness
6.1. Sway Criteria

Y-Axis	Non-sway column

6.2. Columns

Column	Axis	Height	Width	Depth/Dia.	I	f'$_c$	E$_c$
		ft	in	in	in⁴	ksi	ksi
Design	Y	10	17	17	6960.08	3	3122.02
Above	Y	12	17	17	6960.08	3	3122.02
Below	Y	(no column specified...)					

6.3. Y - Beams

Beam	Length	Width	Depth	I	f'$_c$	E$_c$
	ft	in	in	in⁴	ksi	ksi
Above Left	30	14	24	16128	3	3122.02

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Beam	Length	Width	Depth	I	f'c	Ec
	ft	in	in	in⁴	ksi	ksi
Above Right	(no beam specified...)					
Below Left	(no beam specified...)					
Below Right	(no beam specified...)					

7. Moment Magnification

7.1. General Parameters

Factors	Code defaults
Stiffness reduction factor, ϕ_K	0.75
Cracked section coefficients, cI(beams)	0.35
Cracked section coefficients, cI(columns)	0.7
$0.2 E_c I_g + E_s I_{se}$ (Y-axis)	1.48e+007 kip-in²
Minimum eccentricity, $e_{y\,min}$	1.11 in

7.2. Effective Length Factors

Axis	Ψ_{top}	Ψ_{bottom}	k (Nonsway)	k (Sway)	kl_u/r
Y	4.512	999.000	0.960	(N/A)	23.48

7.3. Magnification Factors: Y - axis
* Slenderness need not be considered.

Load Combo	At Ends					Along Length					
	$\sum P_u$	P_c	$\sum P_c$	β_{ds}	δ_s	P_u	$k'l_u/r$	P_c	β_{dns}	C_m	δ
	kip	kip	kip			kip		kip			
1 U1	(N/A)	(N/A)	(N/A)	(N/A)	(N/A)	525.00	(N/A)	7850.31	0.400	(N/A)	(N/A) *

8. Factored Moments
NOTE: Each loading combination includes the following cases:
Top - At column top
Bot - At column bottom

8.1. Y - axis

Load Combo	1st Order					2nd Order		Ratio
	M_{ns}	M_s	M_u	M_{min}		M_l	M_c	2nd/1st
	k-ft	k-ft	k-ft	k-ft		k-ft	k-ft	
1 U1 Top	105.00	(N/A)	105.00	(N/A)	M_2=	(N/A)	(N/A)	(N/A)
1 U1 Bot	0.00	(N/A)	0.00	(N/A)	M_1=	(N/A)	(N/A)	(N/A)

9. 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	852.4	0.00	0.00	46.72	14.50	-0.00207	0.65000
Y	@ Allowable comp.	682.0	0.00	90.87	18.48	14.50	-0.00065	0.65000
Y	@ $f_s = 0.0$	534.0	0.00	161.00	14.50	14.50	0.00000	0.65000
Y	@ $f_s = 0.5 f_y$	347.5	0.00	226.41	10.78	14.50	0.00103	0.65000
Y	@ Balanced point	197.3	0.00	273.98	8.58	14.50	0.00207	0.65000
Y	@ Tension control	107.3	0.00	326.74	5.39	14.50	0.00507	0.90000
Y	@ Pure bending	0.0	0.00	278.79	4.02	14.50	0.00782	0.90000
Y	@ Max tension	-540.0	0.00	0.00	0.00	14.50	9.99999	0.90000
-Y	@ Max compression	852.4	0.00	0.00	46.72	14.50	-0.00207	0.65000
-Y	@ Allowable comp.	682.0	0.00	-90.87	18.48	14.50	-0.00065	0.65000
-Y	@ $f_s = 0.0$	534.0	0.00	-161.00	14.50	14.50	0.00000	0.65000

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About Point	P	X-Moment	Y-Moment	NA Depth	d_t Depth	ε_t	ϕ
	kip	k-ft	k-ft	in	in		
-Y @ $f_s = 0.5\,f_y$	347.5	0.00	-226.41	10.78	14.50	0.00103	0.65000
-Y @ Balanced point	197.3	0.00	-273.98	8.58	14.50	0.00207	0.65000
-Y @ Tension control	107.3	0.00	-326.74	5.39	14.50	0.00507	0.90000
-Y @ Pure bending	0.0	0.00	-278.79	4.02	14.50	0.00782	0.90000
-Y @ Max tension	-540.0	0.00	0.00	0.00	14.50	9.99999	0.90000

10. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Critical Capacity" Method.
Each loading combination includes the following cases:
Top - At column top
Bot - At column bottom

No.	Load Combo			Demand		Capacity		Parameters at Capacity			Capacity Ratio
				P_u	M_{uy}	ϕP_n	ϕM_{ny}	NA Depth	ε_t	ϕ	
				kip	k-ft	kip	k-ft	in			
1	1	U1	Top	525.00	105.00	588.71	137.85	15.81	-0.00025	0.650	0.86
2	1	U1	Bot	525.00	0.00	681.95	0.00	18.48	-0.00065	0.650	0.77

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11. Diagrams
11.1. PM at θ=0 [deg]

17 x 17 in

General Information

Project	Pincheira ...ple 13.17.3
Column	Exterior
Engineer	SP
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Y - axis
Slenderness	Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials

f'$_c$	3	ksi
E$_c$	3122.02	ksi
f$_y$	60	ksi
E$_s$	29000	ksi

Section

Type	Rectangular	
Width	17	in
Depth	17	in
A$_g$	289	in^2
I$_x$	6960.08	in^4
I$_y$	6960.08	in^4

Reinforcement

Pattern	Sides different
Bar layout	Rectangular
Cover to	Longitudal bars
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A$_s$	10.00 in^2
Rho	3.46 %
Min. clear spacing	1.87 in

PM at 0.0 [deg]

No.	Load Combo			P$_u$ kip	M$_{uy}$ k-ft	φP$_n$ kip	φM$_{ny}$ k-ft	Capacity Ratio
1	1	U1	Top	525.0	105.0	588.71	137.85	0.86
2	1	U1	Bot	525.0	0.0	681.95	0.00	0.77

Max. Capacity Ratio: 0.86

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6. Summary and Comparison of Design Results

Analysis and design results from the hand calculations above are compared with the reference values and the exact values obtained from spColumn model.

	k (Note 3)	$(EI)_{eff}$, kip-in.^{2}	P_c, kip	P_u, kip	δ_{ns} (Note 4)	M_c, kip-ft
		Table 2 – Parameters for moment magnification of column in non-sway frame				
Reference	1.000	10.50×10^6	7200	525	0.66	105
Hand	0.959	10.56×10^6	7871	525	0.66	105
spColumn	0.960	10.57×10^6	7850	525	N/A	105

All the results of the hand calculations illustrated above are in precise agreement with the automated exact results obtained from the spColumn program.

The notes below are helpful to the spColumn user in creating the design model:

1. The reference used the larger of the two equations provided by ACI 318-19 (6.6.4.4.4) to calculate $(EI)_{eff}$ since both $(EI)_{eff}$ equations are lower bounds. However, the hand solution and spColumn use the first equation since it provides an estimate that is dependent on the reinforcement configuration provided in the section 2.

2. The reference used an approximate equation to calculate the radius of gyration (r) while the hand solution and spColumn use the exact equation to calculate r value.

3. The reference decided to use k = 1 in accordance with the more conservative procedure of ACI 318-19 (6.6.4.4.3). The hand solution and spColumn calculate the exact k value.

4. δ_{ns} in the three methods of solution shown above need not be calculated since the slenderness effects need not be considered. The reference and hand solution show this value for illustration purposes.

7. Conclusions & Observations

7.1. General 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 and includes slenderness effects using moment magnification method for sway and nonsway frames.

ACI 318 provides multiple options for calculating values of r and $(EI)_{eff}$ leading to variability in the determination of the adequacy of a column section. Engineers must exercise judgment in selecting suitable options to match their design condition. The spColumn program utilizes the exact methods whenever possible and allows user to override the calculated values with direct input based on their engineering judgment wherever it is permissible.

7.2. Design Column End Conditions in Slenderness Calculations

When the slenderness effects for a non-sway frame column is considered in creating a model using spColumn, the effective length factor can be input using one of the following three methods:

1. Entered manually
2. Computed by defining the properties of the columns and beams connected to the top and bottom of the design column
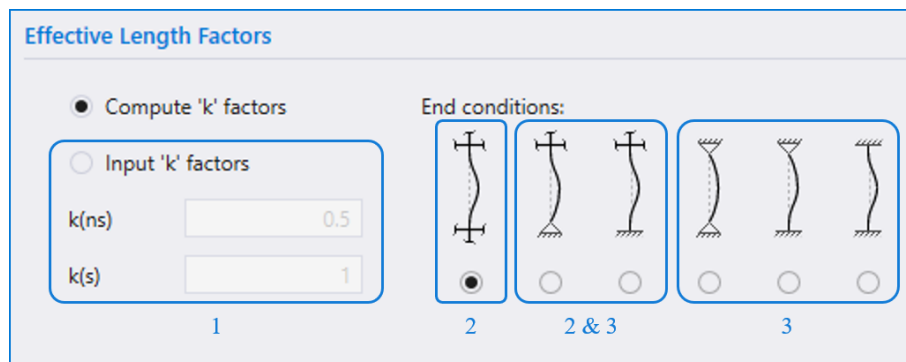3. Or simply computed by using the predefined end conditions offered by spColumn.

Figure 11 – Effective Length Factors & End Conditions (spColumn)