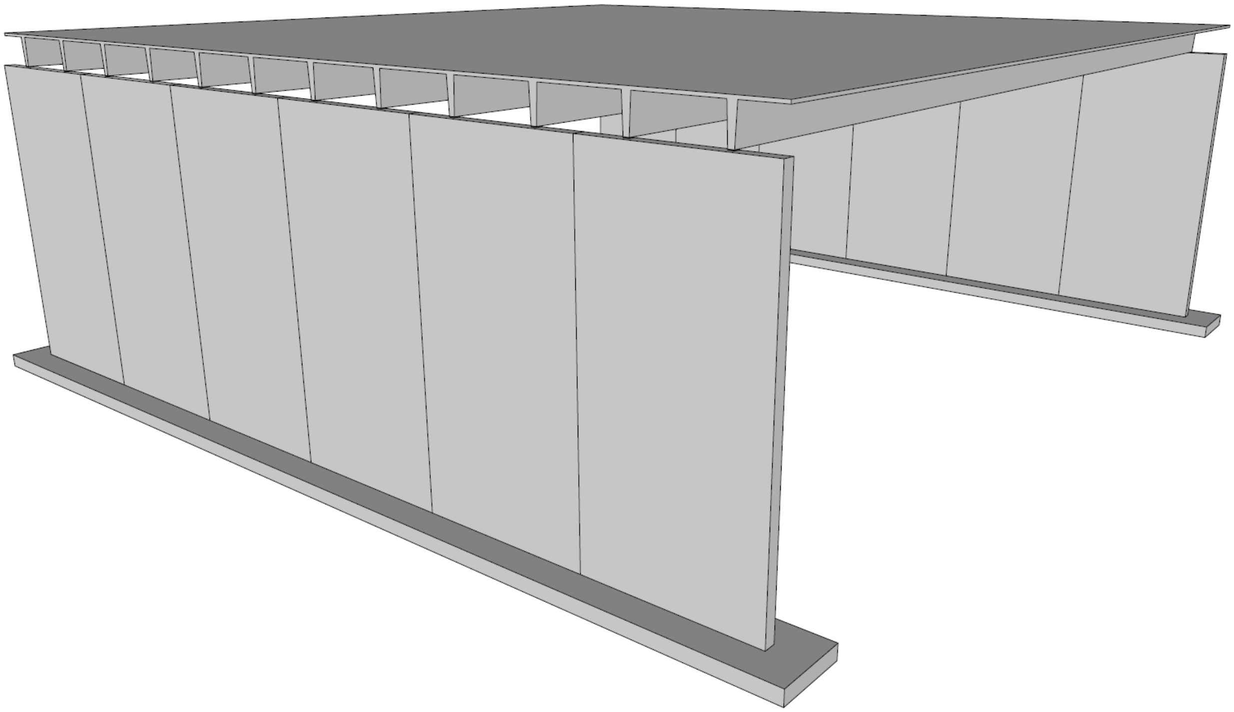
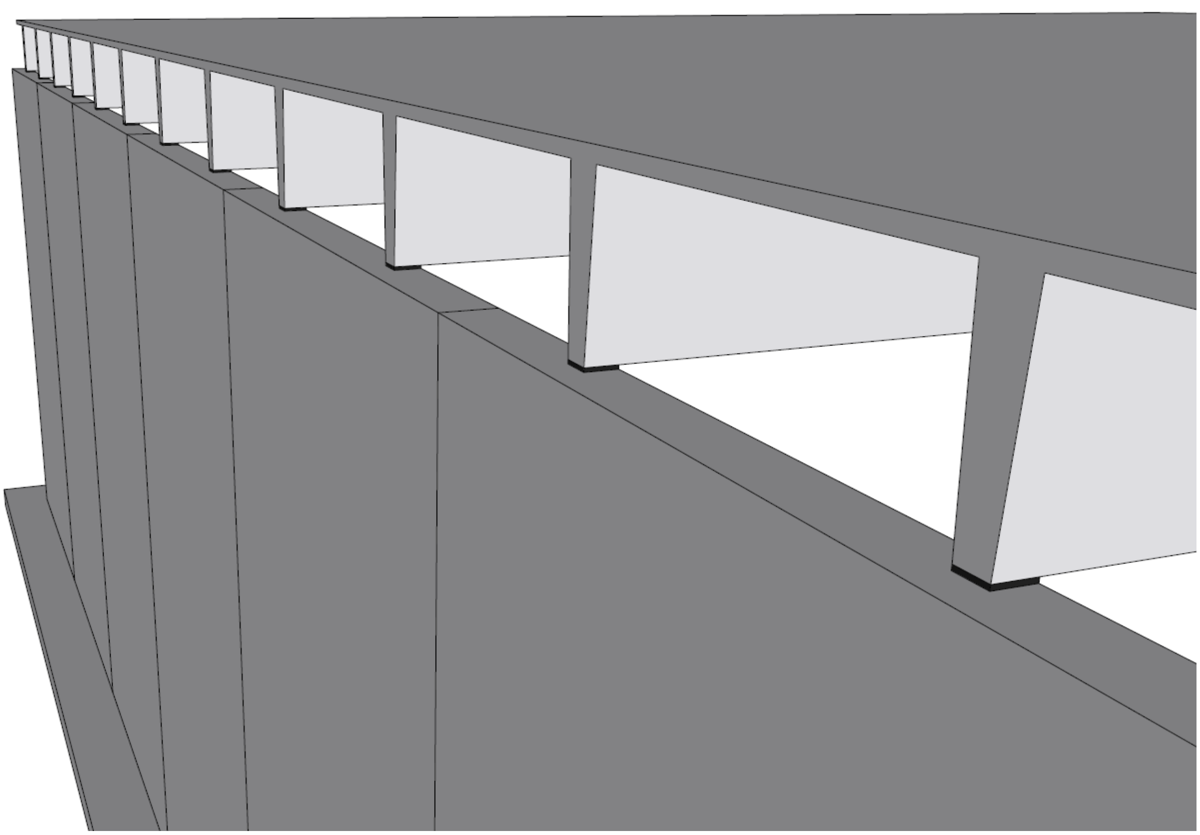
Precast Concrete Bearing Wall Panel Design (Alternative Analysis Method) (ACI 318-14)





Precast Concrete Bearing Wall Panel Design (Alternative Analysis Method) (ACI 318-14)

A structural precast reinforced concrete wall panel in a single-story building provides gravity and lateral load resistance for the following applied loads:

Weight of 10DT24 = 468 plf

Roof dead load = 20 psf

Roof live load = 30 psf

Wind load = 30 psf

The 10DT24 are spaced 5 ft on center. The assumed precast wall panel section and reinforcement are investigated after analysis to verify suitability for the applied loads then compared with numerical analysis results obtained from spWall engineering software program from StructurePoint.

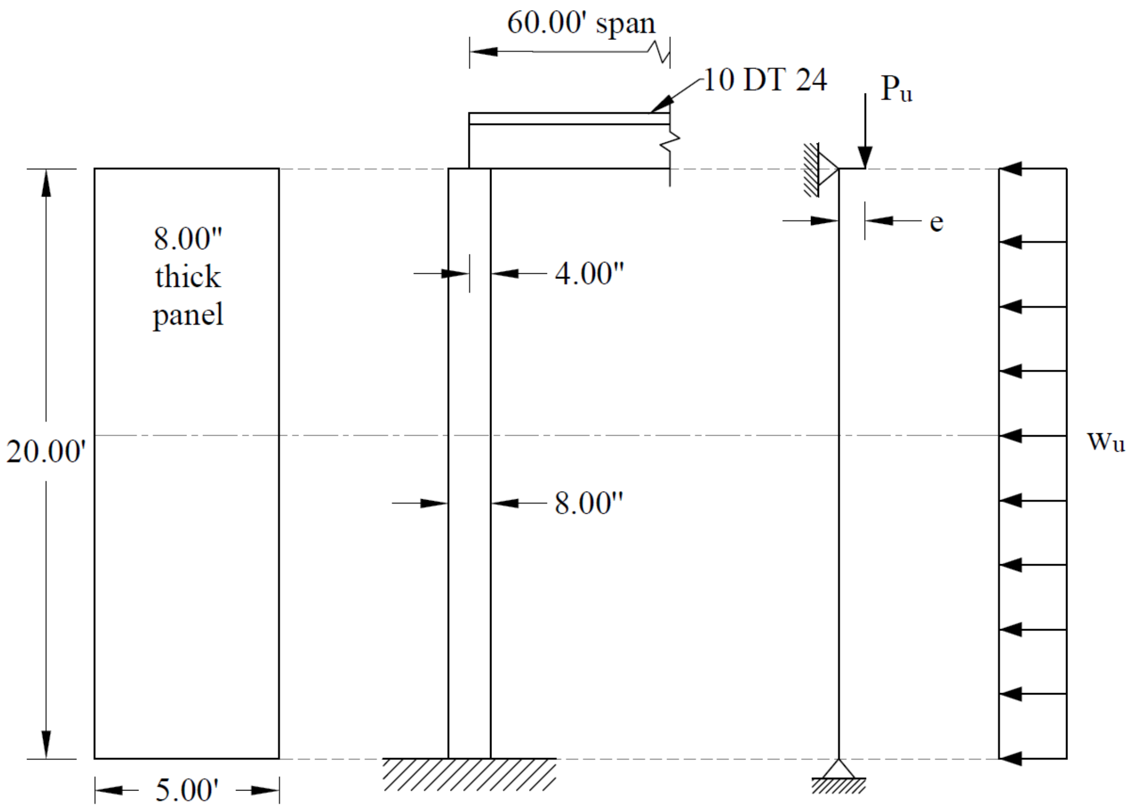


Figure 1 – Reinforced Concrete Precast Wall Panel Geometry

Contents

Code

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

Reference

- Notes on ACI 318-11 Building Code Requirements for Structural Concrete, Twelfth Edition, 2013 Portland Cement Association, Example 21.3
- spWall Engineering Software Program Manual v10.00, STRUCTUREPOINT, 2022

Design Data

f_c' = 4,000 psi normal weight concrete (w_c = 150 pcf)

f_y = 60,000 psi

Wall length = 20 ft

Assumed wall thickness = 8 in.

Assumed vertical reinforcement: single layer of #4 bars at 9 in. at centerline of wall.

($A_{s,\ vertical}$ = 0.20 / 9 in. x 12 in. = 0.27 in.^{2}/ft)

1. Minimum Vertical Reinforcement

$$\rho_l = \frac{A_{v,vertical}}{h \times s_1} = \frac{0.27}{12 \times 8} = 0.0028$$ ***ACI 318-14 (2.2)***

$$\rho_{l,\min} = 0.0012$$ ***ACI 318-14 (Table 11.6.1)***

$$\rho_l = 0.0028 \geq \rho_{l,\min} = 0.0012 \text{ (o.k)}$$

$$s_{l,\max} = \text{smallest of} \begin{Bmatrix} 3 \times h \\ 18 \text{ in.} \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 3 \times 8 \\ 18 \text{ in.} \end{Bmatrix} = \text{smallest of} \begin{Bmatrix} 24 \text{ in.} \\ 18 \text{ in.} \end{Bmatrix} = 18 \text{ in.}$$ ***ACI 318-14 (11.7.2.1)***

$$s_{l,provided} = 9 \text{ in.} \leq s_{l,\max} = 18 \text{ in.} \ (\textbf{o.k.})$$

2. Alternative Method for Out-of-Plane Slender Wall Analysis Applicability

Precast concrete walls can be analyzed using the provisions of Chapter 11 of the ACI 318-14. Most walls, and especially slender walls, are widely evaluated using the "Alternative Method for Out-of-Plane Slender Wall Analysis" in Section 11.8. The requirements of this procedure are summarized below:

- The cross section shall be constant over the height of the wall ***ACI 318-14 (11.8.1.1(a))***
- The wall can be designed as simply supported ***ACI 318-14 (11.8.2.1)***
- Maximum moments and deflections occurring at midspan ***ACI 318-14 (11.8.2.1)***
- The wall must be axially loaded ***ACI 318-14 (11.8.2.1)***
- The wall must be subjected to an out-of-plane uniform lateral load ***ACI 318-14 (11.8.2.1)***
- The wall shall be tension-controlled ***ACI 318-14 (11.8.1.1(b))***
- The reinforcement shall provide design strength greater than cracking strength ***ACI 318-14 (11.8.1.1(c))***
- P_u at the midheight section does not exceed 0.06 $f_c'A_g$ ***ACI 318-14 (11.8.1.1(d))***
- Out-of-plane deflection due to service loads including $P\Delta$ effects does not exceed $l_c/150$ ***ACI 318-14 (11.8.1.1(e))***

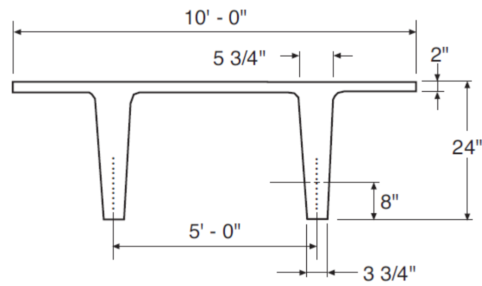
ACI 318 requires that concentrated gravity loads applied to the wall above the design flexural section shall be assumed to be distributed over a width: ***ACI 318-14 (11.8.2.2)***

a) Equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design section.
b) Not greater than the spacing of the concentrated loads.
c) Not extending beyond the edges of the wall panel.

$$\text{Distribution Width of Concentrated Loads} = \min \begin{Bmatrix} W + \dfrac{l_c}{2} \\ \text{spacing of the concentrated loads} \end{Bmatrix}$$ ***ACI 318-14 (11.8.2.2)***

$$\text{Distribution Width of Concentrated Loads} = \min\begin{Bmatrix} \dfrac{3.75}{12} + \dfrac{20}{2} = 10.3 \text{ ft} \\ 5.0 \text{ ft} \end{Bmatrix} = 5.0 \text{ ft}$$

3. Wall Structural Analysis



10DT24

Using 14.8 provisions, calculate factored loads as follows for each of the considered load combinations:

3.1. Roof load per foot width of wall

$$\text{Wall self-weight} = \frac{8}{12} \times 20 \times 150 = 2,000 \text{ plf}$$

$$D = \left[\frac{468}{2} + (20 \times 5)\right] \times \left(\frac{60}{2}\right) = 10,020 \text{ lbs / 5 ft} = 2,004 \text{ plf}$$

$$L = (30 \times 5) \times \left(\frac{60}{2}\right) = 4,500 \text{ lbs / 5 ft} = 900 \text{ plf}$$

$$\text{Eccentricity of the roof loads about the panel center line} = \frac{2}{3} \times 4 = 2.67 \text{ in.}$$

3.2. Calculation of maximum wall forces

The calculation of maximum factored wall forces in accordance with 11.8 is summarized in Figure 2 including moment magnification due to second order (P-Δ) effects.

P_{u1} = factored applied gravity load
P_{u2} = factored self-weight of the wall (total)
e = eccentricity of applied gravity load
w_u = factored uniform lateral load

$$P_u = P_{u1} + \frac{P_{u2}}{2}$$

$$M_u = M_{ua} + P_u\,\Delta_u = \frac{M_{ua}}{1 - \dfrac{5P_u \ell_c^2}{(0.75)48E_c I_{cr}}}$$

$$M_{ua} = \frac{w_u \ell_c^2}{8} + \frac{P_{u1} e}{2}$$

$$M_u = \frac{w_u \ell_c^2}{8} + \frac{P_{u1} e}{2} + \left(P_{u1} + \frac{P_{u2}}{2}\right)\Delta_u$$

$$\Delta_u = \frac{5M_u \ell_c^2}{(0.75)48E_c I_{cr}}$$

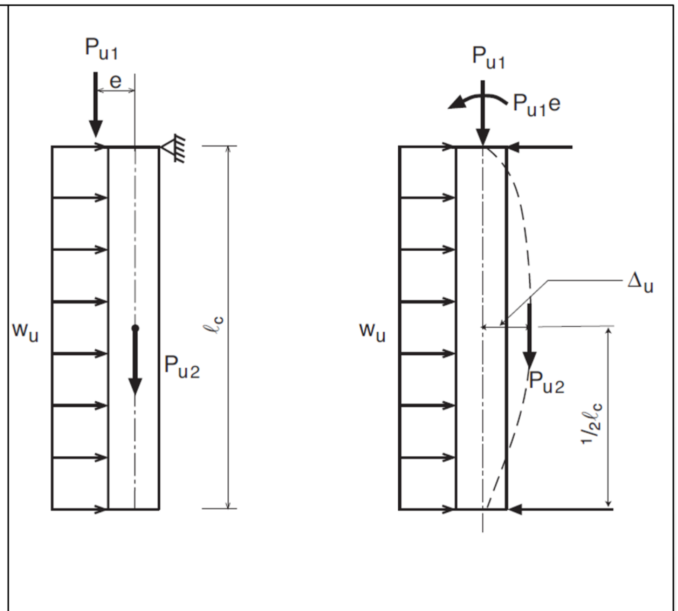


Figure 2 – Wall Structural Analysis According to the Alternative Design of Slender Walls Method (PCA Notes)

For load combination #1 (U = 1.4 D):

$$P_u = P_{u1} + \frac{P_{u2}}{2} = (1.4 \times 2{,}004) + \frac{(1.4 \times 2{,}000)}{2} = 4.21 \text{ kips}$$

$$M_u = \frac{M_{ua}}{1 - \dfrac{5 \times P_u \times l_c^2}{0.75 \times 48 \times E_c \times I_{cr}}}$$ **ACI 318-14 (11.8.3.1d)**

$$M_{ua} = \frac{w_u \times l_c^2}{8} + \frac{P_{u1} \times e}{2} = \frac{0 \times (20 \times 12)^2}{8} + \frac{2.81 \times 2.67}{2} = 3.74 \text{ in.-kips}$$

Where M_{ua} is the maximum factored moment at midheight of wall due to lateral and eccentric vertical loads, not including PΔ effects. **ACI 318-14 (11.8.3.1)**

$$E_c = 57{,}000 \times \sqrt{f_c'} = 57{,}000 \times \sqrt{4{,}000} = 3{,}605{,}000 \text{ psi}$$ **ACI 318-14 (19.2.2.1.b)**

$$I_{cr} = n \times A_{se,w} \times (d - c)^2 + \frac{l_w \times c^3}{3}$$ **ACI 318-14 (11.8.3.1c)**

$$n = \frac{E_s}{E_c} = \frac{29{,}000}{3{,}605} = 8.0 > 6.0 \ (\textbf{o.k.})$$ **ACI 318-14 (11.8.3.1)**

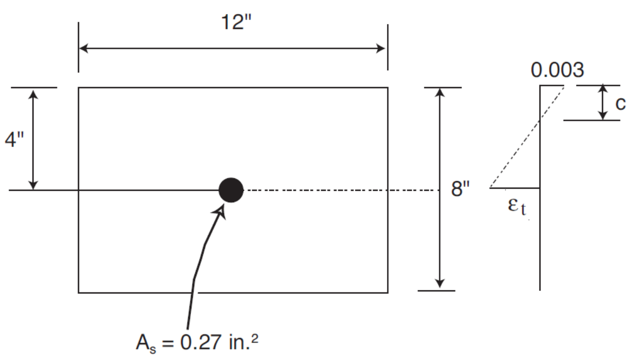
Calculate the effective area of longitudinal reinforcement in a slender wall for obtaining an approximate cracked moment of inertia.

$$A_{se,w} = A_s + \frac{P_u \times h}{2 \times f_y \times d} = 0.27 + \frac{4.21 \times 8}{2 \times 60 \times 4} = 0.34 \text{ in.}^2/\text{ft}$$ **ACI 318-14 (R11.8.3.1)**

The following calculation are performed with the effective area of steel in lieu of the actual area of steel.

$$a = \frac{A_{se,w} \times f_y}{0.85 \times f_c' \times l_w} = \frac{0.34 \times 60}{0.85 \times 4 \times 12} = 0.50 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{0.50}{0.85} = 0.58 \text{ in.}$$



$$I_{cr} = 8.0 \times 0.34 \times (4 - 0.58)^2 + \frac{12 \times 0.58^3}{3} = 32.43 \text{ in.}^4$$ **ACI 318-14 (11.8.3.1c)**

$$\varepsilon_t = \left(\frac{0.003}{c}\right) \times d_t - 0.003 = \left(\frac{0.003}{0.58}\right) \times 4.0 - 0.003 = 0.0176 > 0.005$$

Therefore, section is tension controlled **ACI 318-14 (Table 21.2.2)**

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

$$M_u = \frac{M_{ua}}{1 - \dfrac{5 \times P_u \times l_c^2}{0.75 \times 48 \times E_c \times I_{cr}}}$$ **ACI 318-14 (11.8.3.1d)**

$$M_u = \frac{3.74}{1 - \dfrac{5 \times 4.21 \times (20 \times 12)^2}{0.75 \times 48 \times 3,605 \times 32.43}} = 5.25 \text{ in.-kips}$$

The steps above are repeated for all the considered load combinations, Table 1 shows the factored loads at mid-height of wall for all of these load combinations.

Table 1 - Factored load combinations at mid-height of wall											
Load Combination	P_u, kips	M_{ua}, in.-kips	E_c, ksi	n	$A_{se,w}$, in.^{2}/ft	a, in.	c, in.	I_{cr}, in.^{4}	ε_t, in./in.	ϕ	M_u, in.-kips
1.4 D	4.21	3.74	3,605	8	0.34	0.50	0.58	32.4	0.0176	0.9	5.25
1.2 D + 1.6 L_r + 0.8 W	5.04	19.53	3,605	8	0.35	0.52	0.61	33.4	0.0168	0.9	29.38
1.2 D + 0.5 L_r +1.6 W	4.05	32.61	3,605	8	0.33	0.49	0.58	32.3	0.0178	0.9	45.22
0.9 D + 1.6 W	2.70	31.21	3,605	8	0.31	0.46	0.54	30.7	0.0193	0.9	38.80

3.3. Tension-controlled verification **ACI 318-14 (11.8.1.1(b))**

For this check use the largest P_u (5.04 kips) from load combination 2 to envelop all the considered combinations.

$$P_n = \frac{P_u}{\phi} = \frac{5.04}{0.9} = 5.61 \text{ kips}$$

$$a = \frac{A_{se,w} \times f_y}{0.85 \times f'_c \times l_w} = \frac{\dfrac{P_n \times h}{2 \times d} + A_s \times f_y}{0.85 \times f'_c \times l_w} = \frac{\dfrac{5.61 \times 8}{2 \times 4} + 0.27 \times 60}{0.85 \times 4 \times 12} = 0.530 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{0.530}{0.85} = 0.623 \text{ in.}$$

$$\varepsilon_t = \left(\frac{0.003}{c}\right) \times d_t - 0.003 = \left(\frac{0.003}{0.623}\right) \times 4.0 - 0.003 = 0.0163 > 0.005$$

Therefore, section is tension controlled **ACI 318-14 (Table 21.2.2)**

4. Wall Cracking Moment Capacity (M_{cr})

Determine f_r = Modulus of rapture of concrete and I_g = Moment of inertia of the gross uncracked concrete section to calculate M_{cr}

$$f_r = 7.5\lambda\sqrt{f_c^{'}} = 7.5 \times 1.0 \times \sqrt{4,000} = 474.3 \text{ psi}$$ ***ACI 318-14 (19.2.3.1)***

$$I_g = \frac{l_w h^3}{12} = \frac{12 \times 8^3}{12} = 512 \text{ in.}^4$$

$$y_t = \frac{h}{2} = \frac{8}{2} = 4 \text{ in.}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{474.3 \times 512}{4} \times \frac{1}{1000} = 60.72 \text{ in.-kip}$$ ***ACI 318-14 (24.2.3.5b)***

5. Wall Flexural Moment Capacity (ϕM_n)

For load combination #1:

$$M_n = A_{se,w} \times f_y \times \left(d - \frac{a}{2}\right) = 0.34 \times 60 \times \left(4 - \frac{0.50}{2}\right) = 75.82 \text{ in.-kip}$$

It was shown previously that the section is tension controlled → ϕ = 0.9

$$\phi M_n = \phi \times M_n = 0.9 \times 75.82 = 68.24 \text{ in.-kip} > M_u = 5.25 \text{ in.-kips } (\textbf{o.k.})$$ ***ACI 318-14 (11.5.1.1(b))***

$$\phi M_n = 68.24 \text{ in.-kip} > M_{cr} = 60.72 \text{ in.-kips } (\textbf{o.k.})$$ ***ACI 318-14 (11.8.1.1(c))***

Table 2 - Design moment strength check

Load Combination	M_n, in.-kips	ϕ	ϕM_n, in.-kips	M_u, in.-kips	11.5.1.1(b)	M_{cr}, in.-kips	11.8.1.1(c)
1.4 D	75.82	0.9	68.24	5.25 < ϕM_n	**o.k.**	60.72 < ϕM_n	**o.k.**
1.2 D + 1.6 Lr + 0.8 W	78.61	0.9	70.75	29.38 < ϕM_n	**o.k.**	60.72 < ϕM_n	**o.k.**
1.2 D + 0.5 Lr +1.6 W	75.29	0.9	67.76	45.22 < ϕM_n	**o.k.**	60.72 < ϕM_n	**o.k.**
0.9 D + 1.6 W	70.53	0.9	63.47	38.80 < ϕM_n	**o.k.**	60.72 < ϕM_n	**o.k.**

6. Wall Vertical Stress Check

Since load combination 2 provides the largest P_u (5.04 kips), load combination 2 controls.

$$\frac{P_u}{A_g} = \frac{5,044.8}{8 \times 12} = 52.55 \text{ psi} < 0.06 \times f_c' = 0.06 \times 4,000 = 240 \text{ psi} \ (\textbf{o.k.})$$ ***ACI 318-14 (11.8.1.1(d))***

7. Wall Shear Stress Check

In-plane shear is not evaluated in this example since in-plane shear forces are not applied in this example. Out-of-plane shear due to lateral load should be checked against the shear capacity of the wall. By inspection of the maximum shear forces for each load combination, it can be determined that the maximum shear force is under 0.50 kips/ft width. The wall has a shear capacity approximately 4.5 kips/ft width and no detailed calculations are required by engineering judgement. (See figure 11 for detailed shear force diagram)

8. Wall Mid-Height Deflection (Δ_s)

The maximum out-of-plane deflection (Δ_s) due to service lateral and eccentric vertical loads, including PΔ effects, shall not exceed l_c/150. Where Δ_s is calculated as follows: ***ACI 318-14 (11.8.1.1(e))***

$$\Delta_s = \begin{cases} \frac{2}{3}\Delta_{cr} + \frac{M_a - \frac{2}{3}M_{cr}}{M_n - \frac{2}{3}M_{cr}} \times \left(\Delta_n - \frac{2}{3}\Delta_{cr}\right) & \text{When} \quad M_a > \frac{2}{3}M_{cr} \\ \left(\frac{M_a}{M_{cr}}\right)\Delta_{cr} & \text{When} \quad M_a < \frac{2}{3}M_{cr} \end{cases}$$ ***ACI 318-14 (Table 11.8.4.1)***

Where M_a is the maximum moment at mid-height of wall due to service lateral and eccentric vertical loads including PΔ effects.

$$M_a = M_{sa} + P_s \Delta_s$$

$$M_{sa} = \frac{w_s \times l_c^2}{8} + \frac{P_{s1} \times e}{2} = \frac{0.030 \times (20)^2}{8} + \frac{(2.0 + 0.9) \times 2.7 / 12}{2} = 1.82 \text{ ft-kips} = 21.87 \text{ in.-kips}$$

$$P_s = P_{s1} + \frac{P_{s2}}{2} = (2.004 + 0.9) + \frac{2.0}{2} = 3.9 \text{ kips}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = 60.72 \text{ in.-kip (as calculated perviously)}$$ ***ACI 318-14 (24.2.3.5b)***

$$\Delta_{cr} = \frac{5}{48} \times \frac{M_{cr} \times l_c^2}{E_c \times I_g} = \frac{5}{48} \times \frac{60.72 \times (20 \times 12)^2}{3,605 \times 512} = 0.197 \text{ in.}$$ ***ACI 318-14 (11.8.4.3a)***

Δ_s will be calculated by trial and error method since Δ_s is a function of M_a and M_a is a function of Δ_s.

Assume $M_{sa} < \frac{2}{3} M_{cr}$

Assume $\Delta_s = \left(\frac{M_{sa}}{M_{cr}}\right)\Delta_{cr} = \left(\frac{21.87}{60.72}\right) \times 0.197 = 0.071 \text{ in.}$

$M_a = M_{sa} + P_s \Delta_s = 21.87 + 3.9 \times 0.071 = 22.15 \text{ in.-kips}$

$\Delta_s = \left(\frac{M_a}{M_{cr}}\right)\Delta_{cr} = \frac{22.15}{60.72} \times 0.197 = 0.072 \text{ in.}$ ***ACI 318-14 (Table 11.8.4.1)***

No further iterations are required

$M_a = 22.15 \text{ in.-kips} < \frac{2}{3} M_{cr} = \frac{2}{3} \times 60.72 = 40.48 \text{ in.-kips}$ $\quad$ (**o.k.**)

$\Delta_s = 0.072 \text{ in.} < \frac{l_c}{150} = \frac{20 \times 12}{150} = 1.60 \text{ in.}$ $\quad$ (**o.k.**)

The wall is adequate with #4 @ 9 in. vertical reinforcement and 8 in. thickness.

9. Precast Concrete Bearing Wall Panel Analysis and Design – spWall Software

spWall is a program for the analysis and design of reinforced concrete shear walls, tilt-up walls, precast walls and Insulate Concrete Form (ICF) walls. It uses a graphical interface that enables the user to easily generate complex wall models. Graphical user interface is provided for:

- Wall geometry (including any number of openings and stiffeners)
- Material properties including cracking coefficients
- Wall loads (point, line, and area loads)
- Support conditions (including translational and rotational spring supports)

spWall uses the Finite Element Method for the structural modeling, analysis, and design of slender and non-slender reinforced concrete walls subject to static loading conditions. The wall is idealized as a mesh of rectangular plate elements and straight line stiffener elements. Walls of irregular geometry are idealized to conform to geometry with rectangular boundaries. Plate and stiffener properties can vary from one element to another but are assumed by the program to be uniform within each element.

Six degrees of freedom exist at each node: three translations and three rotations relating to the three Cartesian axes. An external load can exist in the direction of each of the degrees of freedom. Sufficient number of nodal degrees of freedom should be restrained in order to achieve stability of the model. The program assembles the global stiffness matrix and load vectors for the finite element model. Then, it solves the equilibrium equations to obtain deflections and rotations at each node. Finally, the program calculates the internal forces and internal moments in each element. At the user's option, the program can perform second order analysis. In this case, the program takes into account the effect of in-plane forces on the out-of-plane deflection with any number of openings and stiffeners.

In spWall, the required flexural reinforcement is computed based on the selected design standard (ACI 318-14 is used in this example), and the user can specify one or two layers of wall reinforcement. In stiffeners and boundary elements, spWall calculates the required shear and torsion steel reinforcement. Wall concrete strength (in-plane and out-of-plane) is calculated for the applied loads and compared with the code permissible shear capacity.

For illustration and comparison purposes, the following figures provide a sample of the input modules and results obtained from an spWall model created for the reinforced concrete wall in this example.

In this model the following modeling assumptions have been made to closely represent the example:

1. 5' wide section of wall is selected to represent the tributary width effective under each of the double tee beam ribs.
2. Idealized continuous wall boundaries using a symmetry support along the vertical edges
3. Pinned the base of the wall assuming support resistance is provided in the X, Y, and Z directions
4. Roller support was used to simulate the diaphragm support provided by the double tee roof beams
5. The load is applied as a single point load under the double tee rib. This can also be applied as a line load or multiple point loads if the complete wall is modeled.

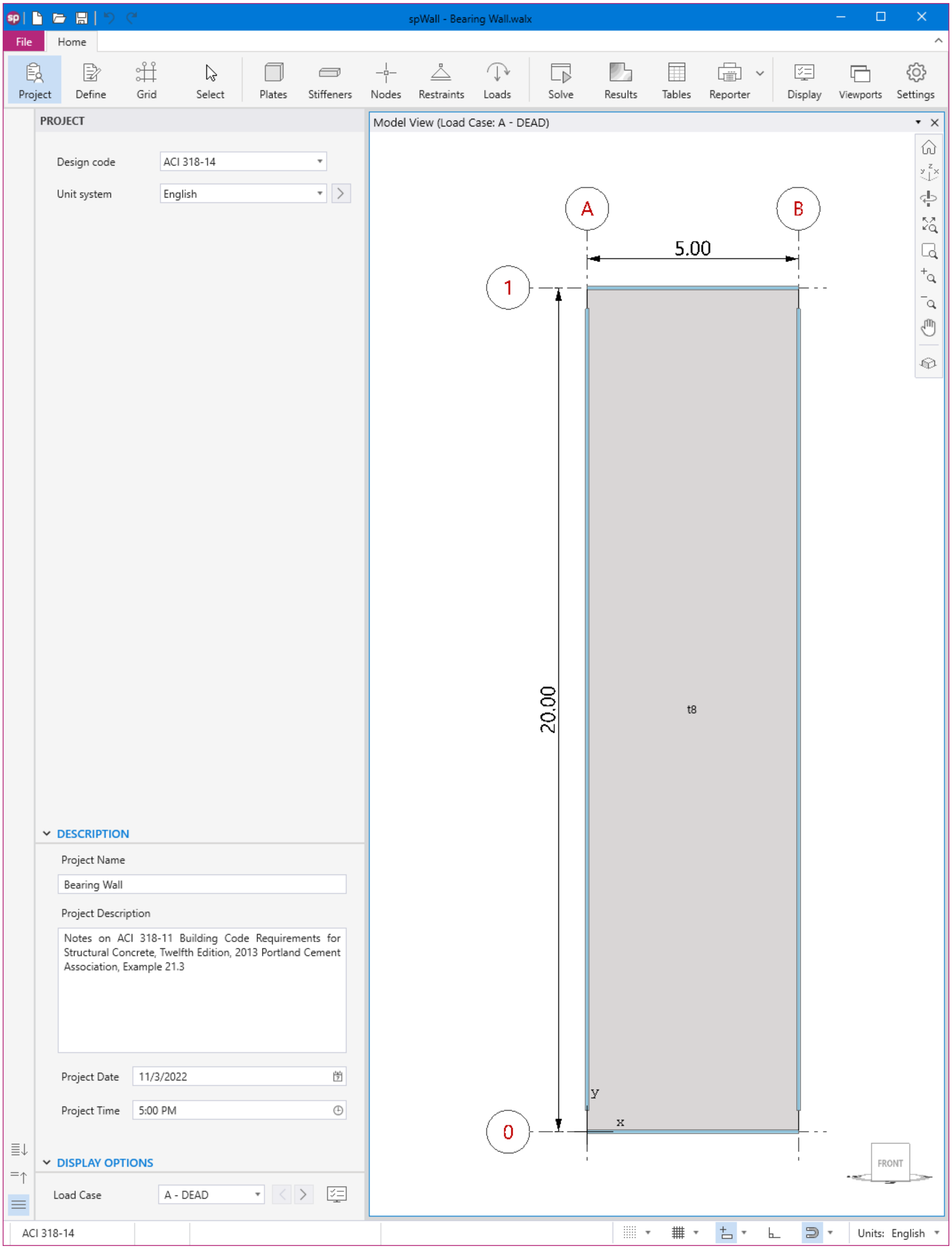


Figure 3 – spWall Interface

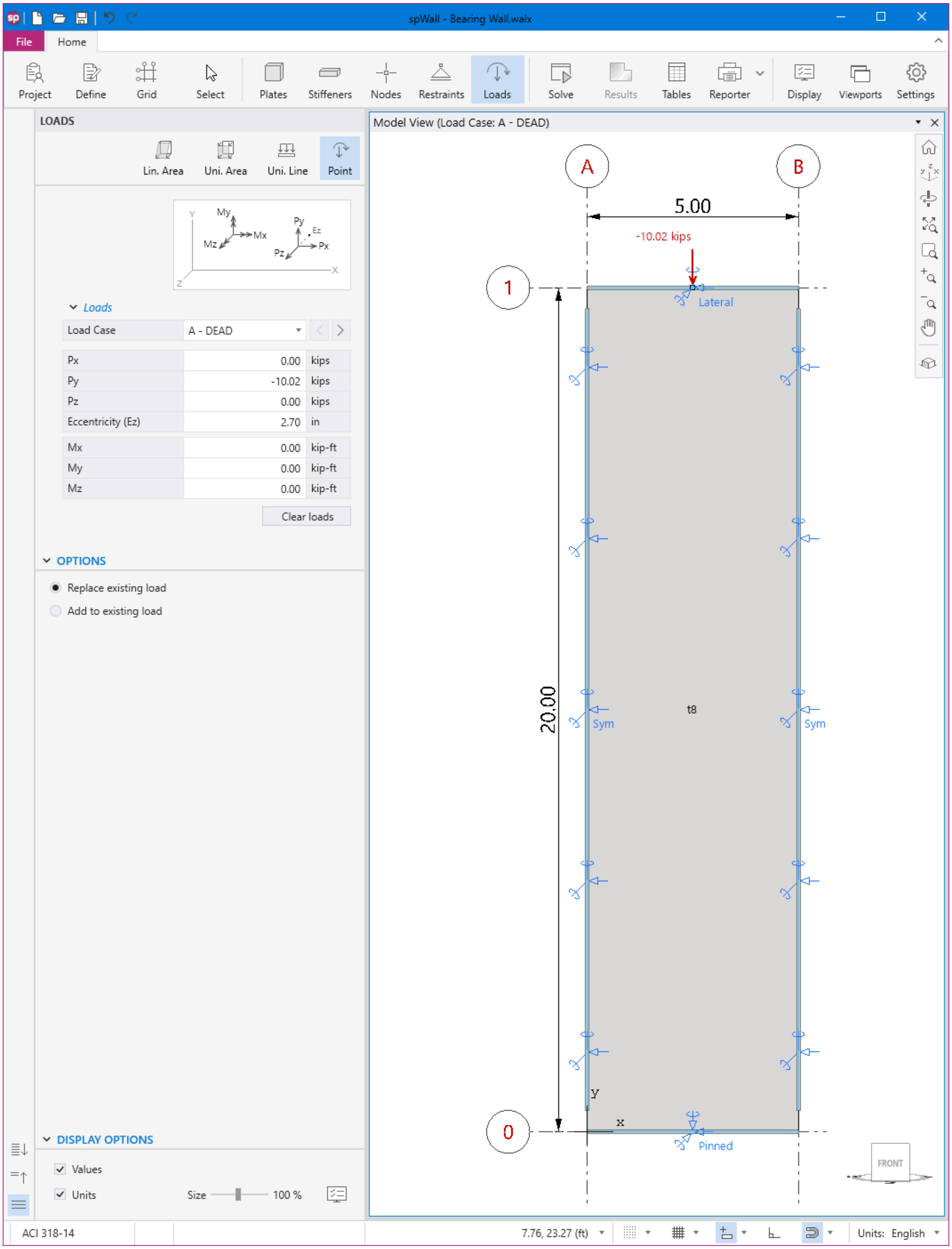


Figure 4 – Assigning Dead Loads for Bearing Wall (spWall)

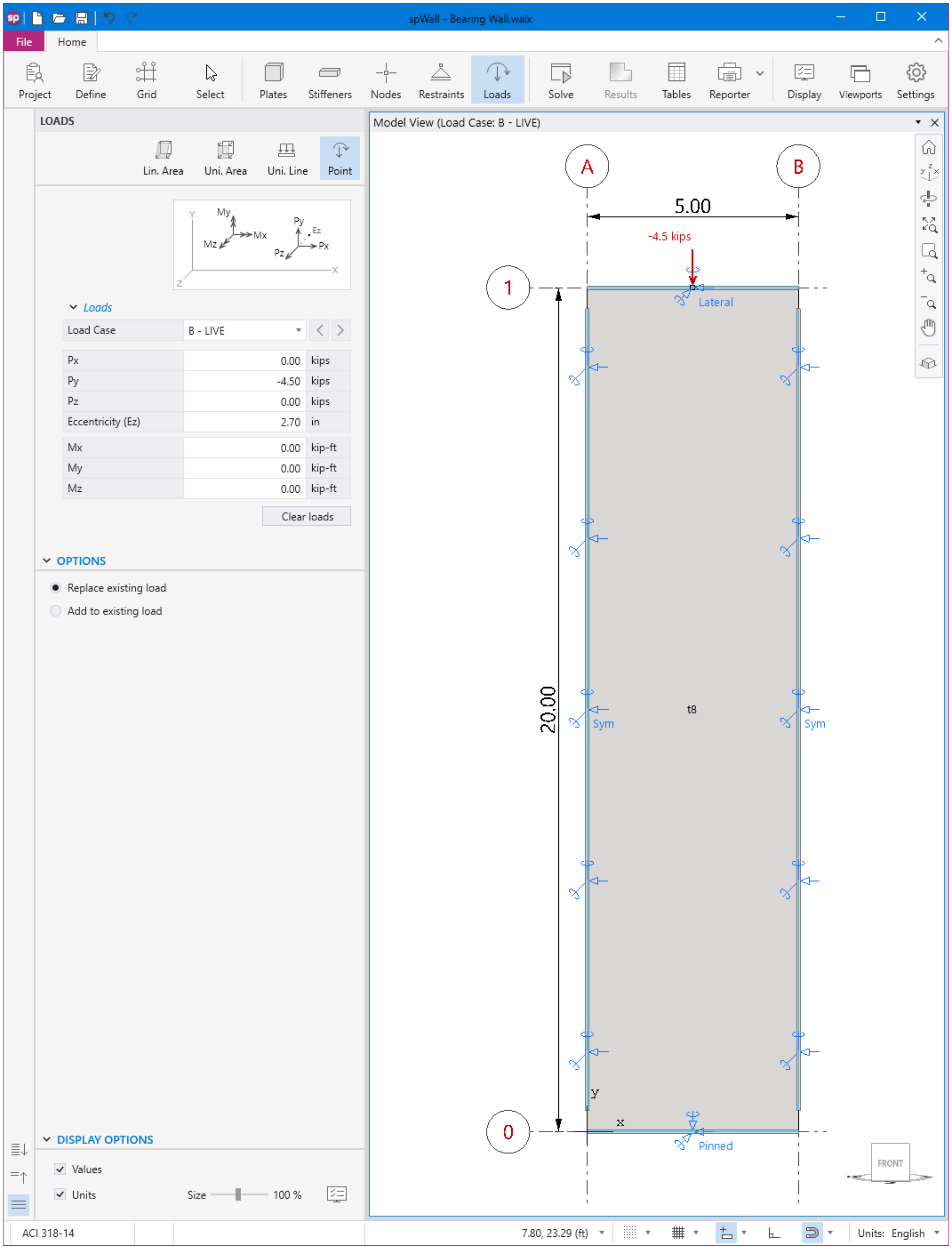


Figure 5 – Assigning Live Loads for Bearing Wall (spWall)

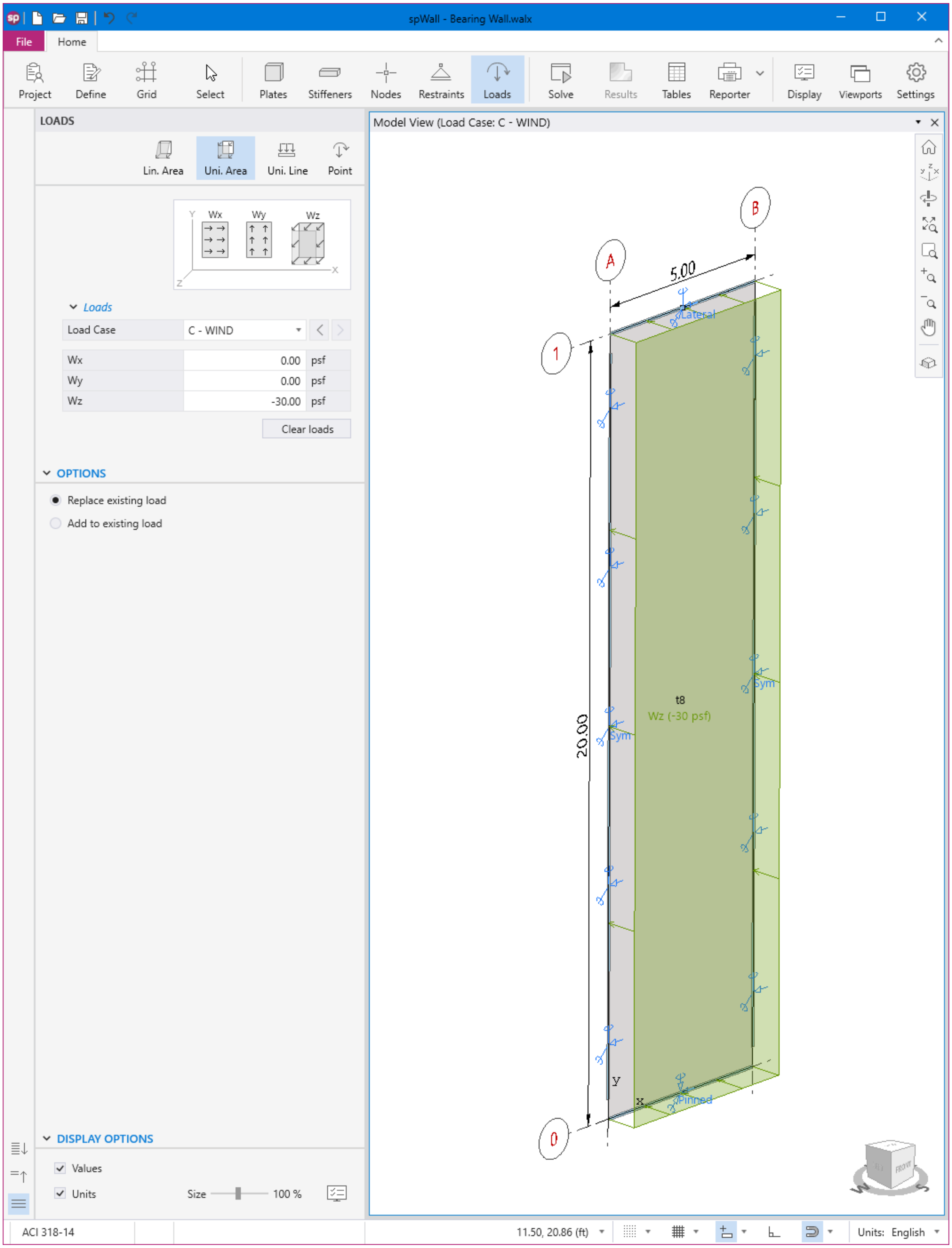


Figure 6 – Assigning Wind Loads for Bearing Wall (spWall)

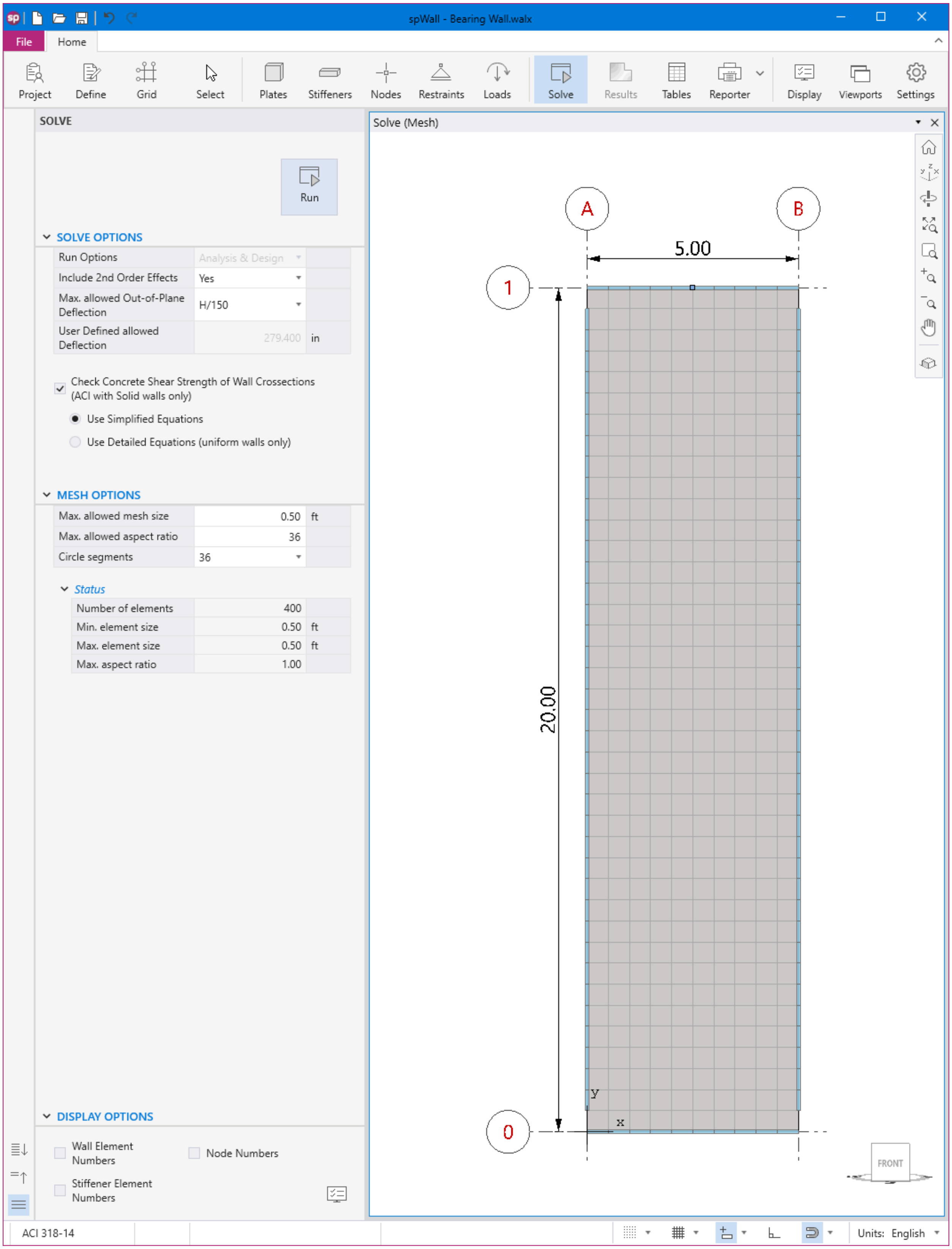


Figure 7 – Solve and Mesh Options (spWall)

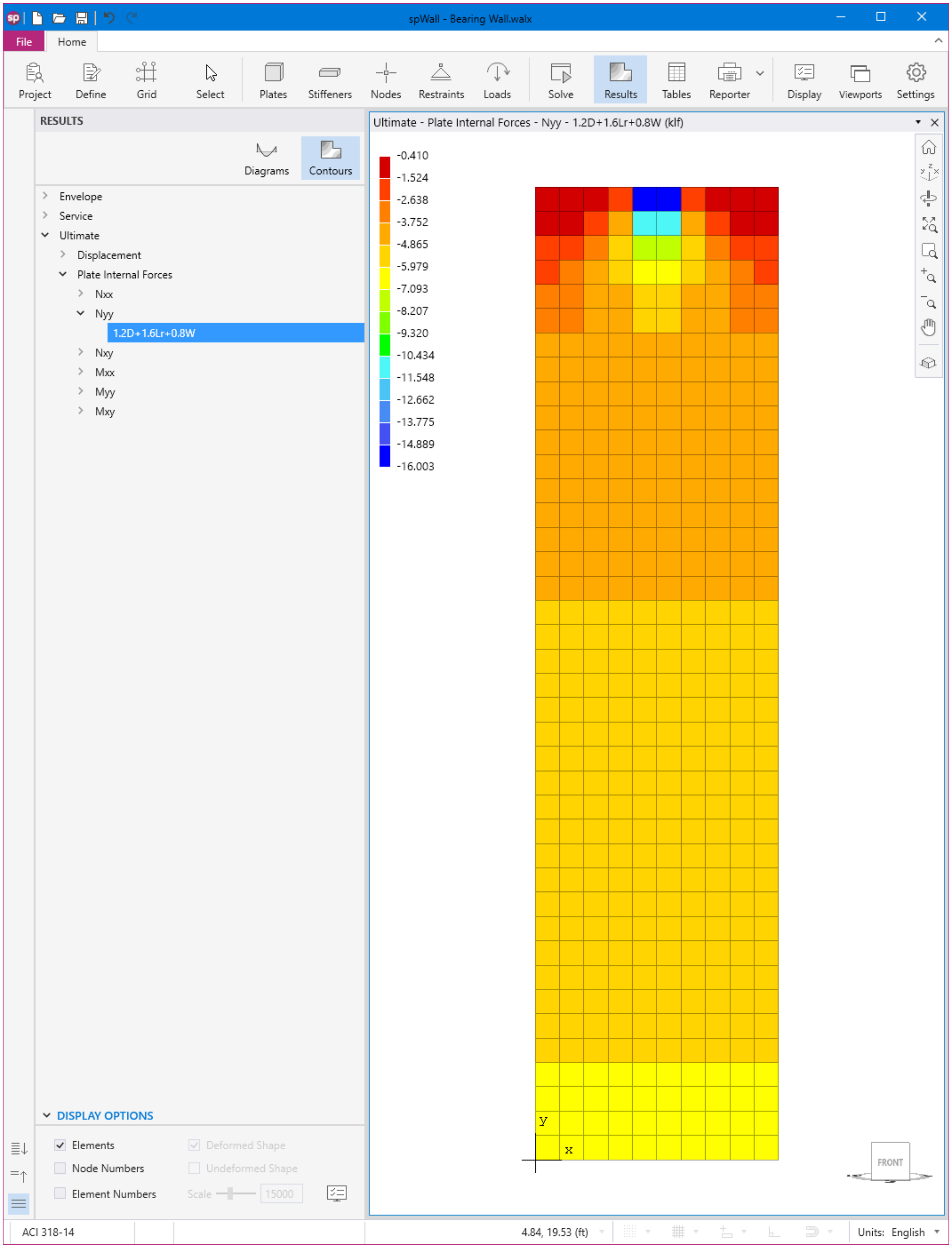


Figure 8 – Factored Axial Forces Contour Normal to Precast Wall Panel Cross-Section (spWall)

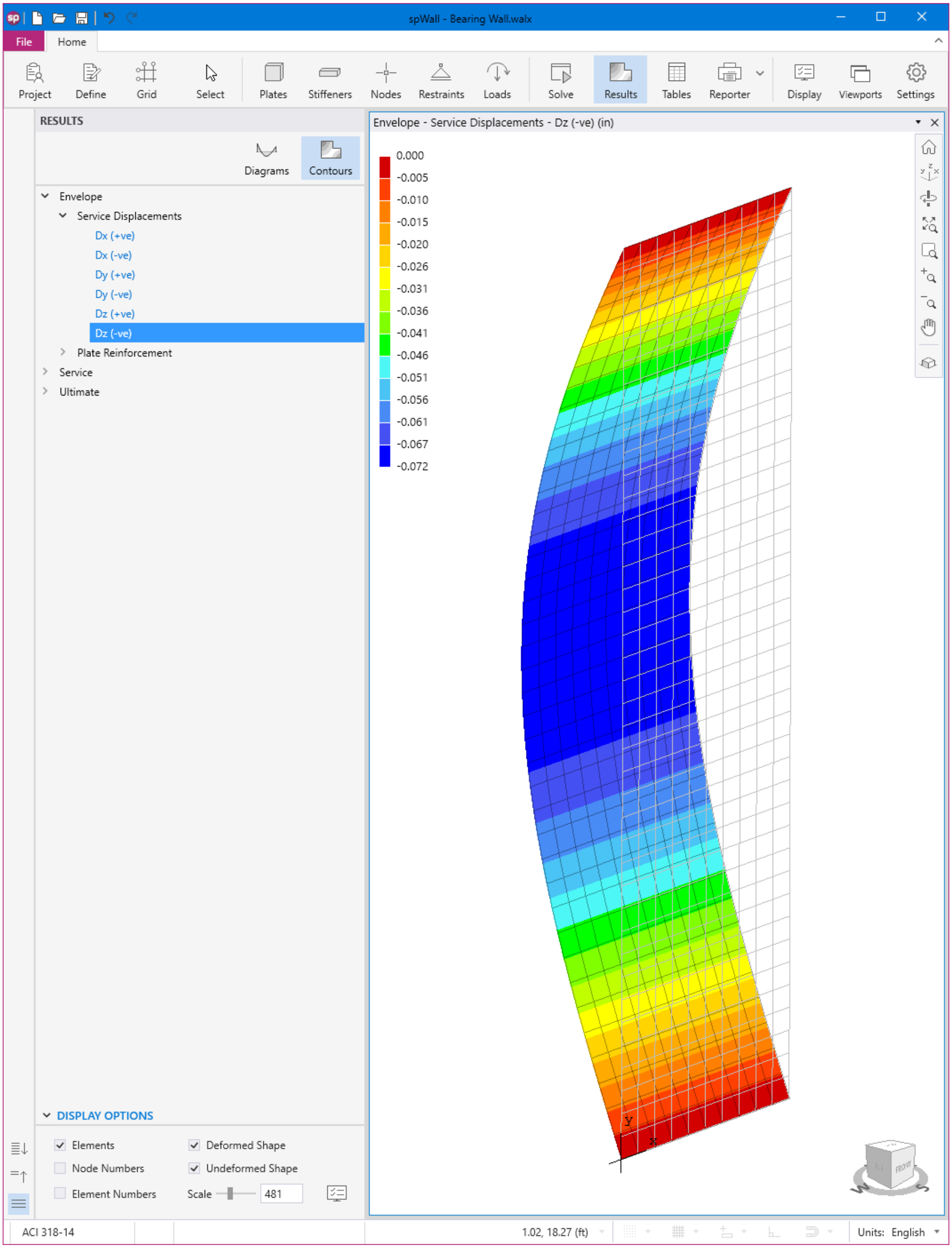


Figure 9 – Precast Wall Panel Lateral Displacement Contour (Out-of-Plane) (spWall)

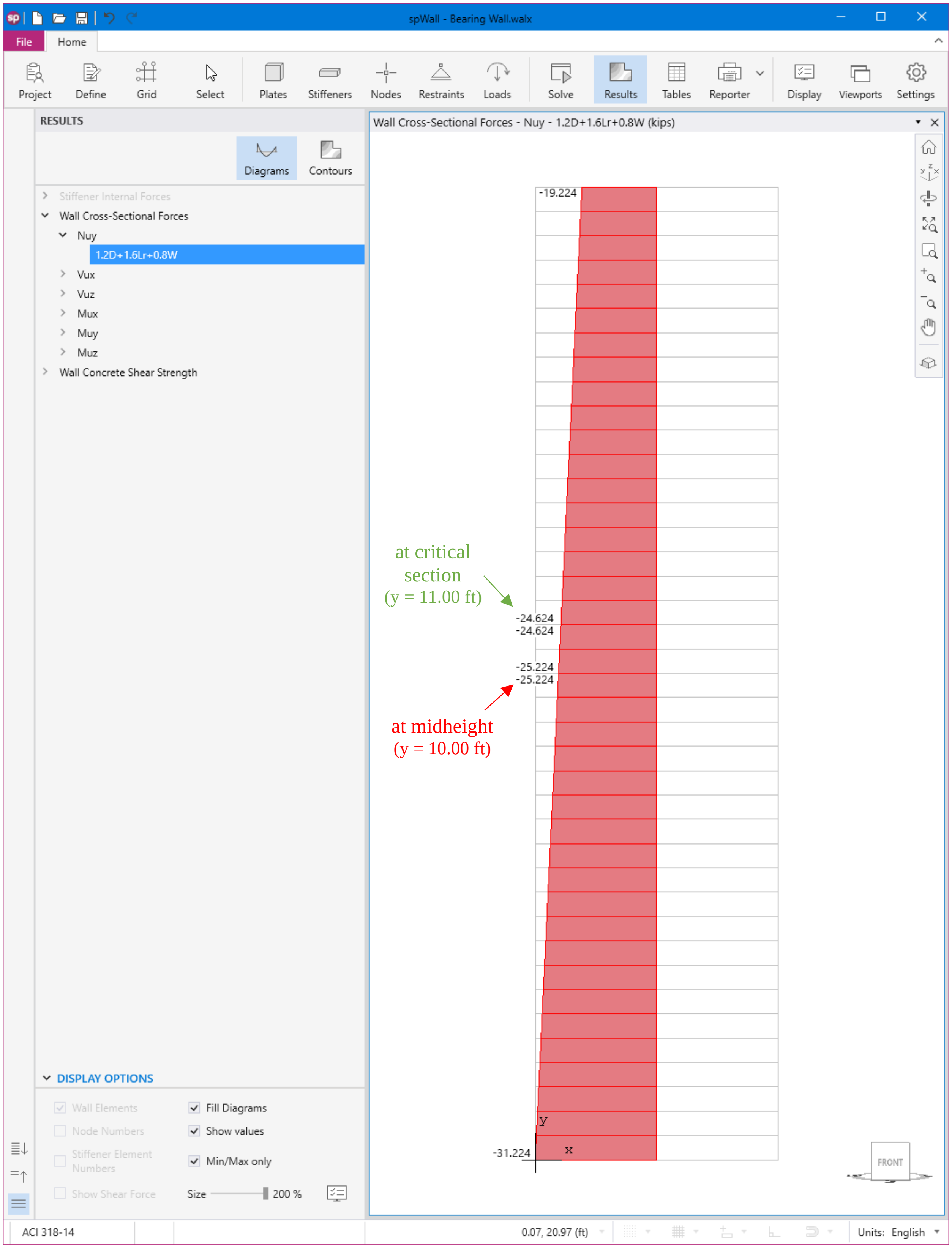


Figure 10 – Precast Wall Panel Axial Load Diagram (spWall)

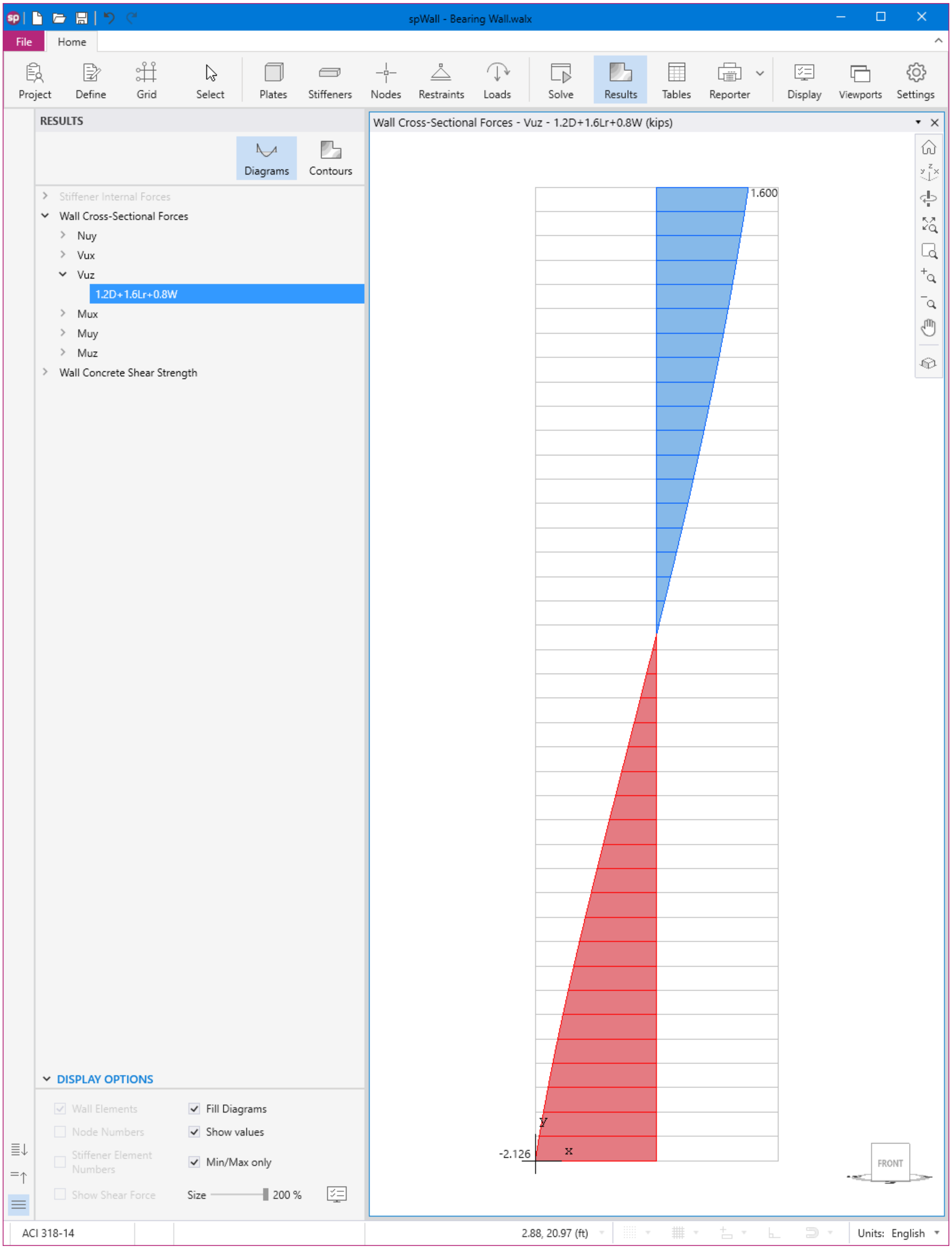


Figure 11 – Out-of-plane Shear Diagram (spWall)

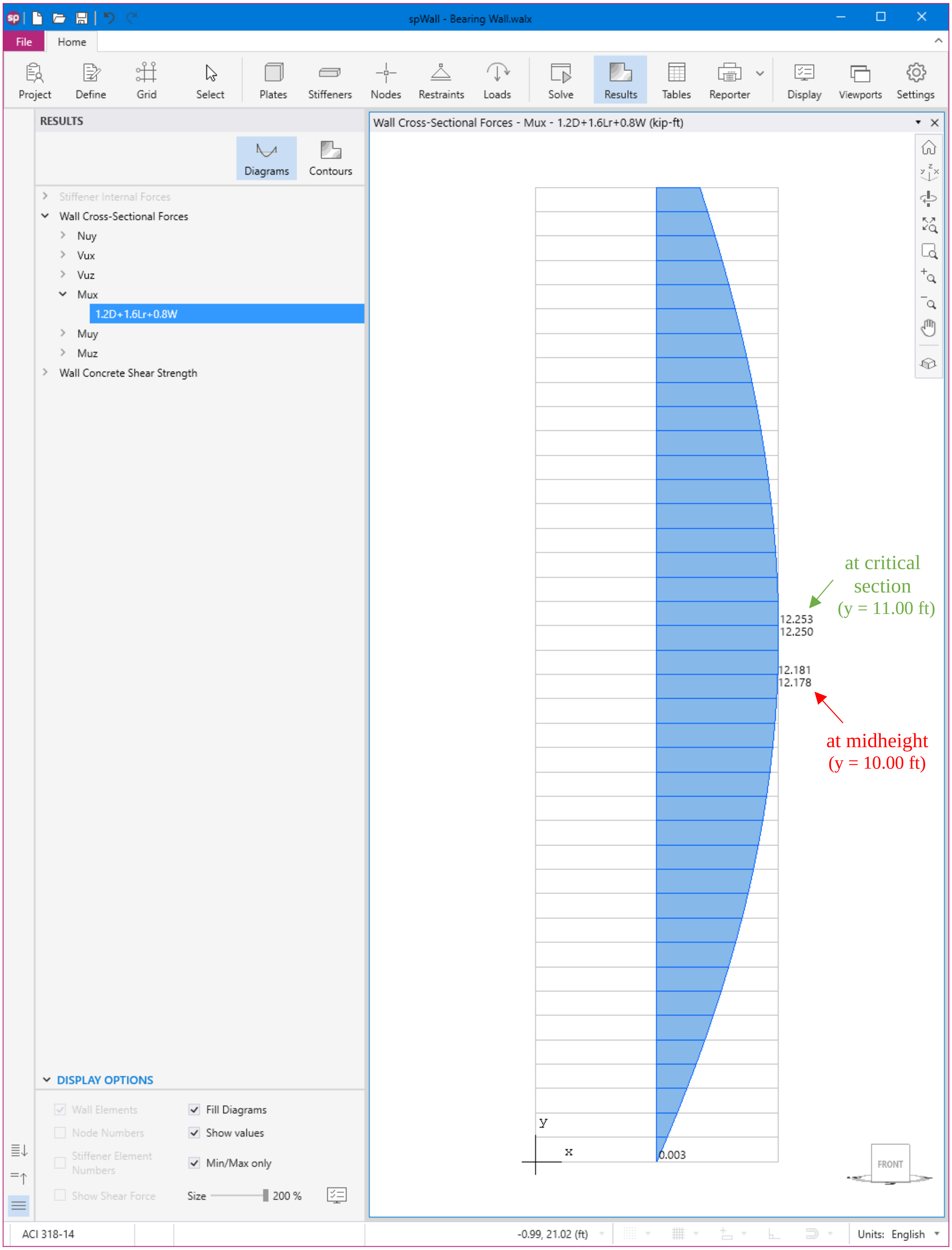


Figure 12 – Bearing Wall Moment Diagram (spWall)

spWall v10.00 (TM)
A Computer Program for Analysis and Design of Reinforced Concrete, Precast, and Tilt-up Walls

Contents

1. Project

1.1. General Information

File Name	Bearing Wall.walx
Project	Bearing Wall
Code	ACI 318-14
Units	English
Date	1/26/2023
Time	5:00 PM

1.2. Solver Options

Include 2nd order effects	Yes
Check out-of-plane service deflections	Yes
Maximum permissible out-of-plane deflections	1.600 in
Check concrete shear strength of wall crossection	Yes (Simplified Equations)

2. Definitions

2.1. Grid Lines

2.1.1. Vertical

Label	Coordinate-X	Spacing
	ft	ft
A	0.00	0.00
B	5.00	5.00

2.1.2. Horizontal

Label	Coordinate-Y	Spacing
	ft	ft
0	0.00	0.00
1	20.00	20.00

2.2. Objects

2.2.1. Plates

Label	Thickness	Concrete	Reinforcement	Design Criteria	Cracking Coeff.	Used
	in					
t8	8.00	C4	Gr60	W8_1C#4	PCC1	Yes

2.3. Properties

2.3.1. Concrete

Label	f'_c	W_c	E_c	v	Precast	Used
	ksi	pcf	ksi	-		
C4	4.0000	150.00	3605.0	0.20	No	Yes

2.3.2. Reinforcement

Label	f_y	E_s	Used	Label	f_y	E_s	Used
	ksi	ksi			ksi	ksi	
Gr60	60.0000	29000.0	Yes				

2.3.3. Plate Cracking Coefficients

Label	Service Combinations		Ultimate Combinations		Used
	In-plane	Out-of-plane	In-plane	Out-of-plane	
PCC1	1	1	1	0.0489	Yes

2.3.4. Plate Design Criteria

NOTE: Bar centroid location measured from Z-ve face for Back Curtain and Z+ve face for Front Curtain

Label	Curtains	Flags	Reinforcement Ratio				Reinforcement Location				Used
			Rmin (Hor)	Rmax (Hor)	Rmin (Ver)	Rmax (Ver)	Back H. (BH)	Back V. (BV)	Front H. (FH)	Front V. (FV)	
			%	%	%	%	in	in	in	in	
W8_1C#4	1		0.20	8.00	0.28	8.00	4.00	4.00	---	---	Yes

2.4. Restraints

2.4.1. Supports

	Translations			Rotations			
Label	Dx	Dy	Dz	Rx	Ry	Rz	Used
Pinned	Fixed	Fixed	Fixed	Free	Fixed	Fixed	Yes
Lateral	Fixed	Free	Fixed	Free	Fixed	Fixed	Yes
Sym	Fixed	Free	Free	Free	Fixed	Fixed	Yes

2.5. Load Case/Combo.

2.5.1. Load Cases

NOTE: Self weight is included under Case A.

Case	Type	Case Label	Load Defined?
A	Dead	DEAD	Yes
B	Live	LIVE	Yes
C	Wind	WIND	Yes

2.5.2. Load Combinations

Combo./Case	A	B	C	D	E	F	G	H	I	Combo Type
Type	Dead	Live	Wind							
Combo./Label	DEAD	LIVE	WIND							
1.2D+1.6L...	1.000	1.000	1.000	-	-	-	-	-	-	Ser.
1.2D+1.6L...	1.200	1.600	0.800	-	-	-	-	-	-	Ult.

3. Assignments

3.1. Nodes

ID	X Coord.	Y Coord.	Rigid Support	Spring Support
	ft	ft		
N1	2.50	20.00		

3.2. Plates

ID	Label	Shape	Top Left/Center X	Top Left/Center Y	Width (B)	Height (H)/Dia. (D)
			ft	ft	ft	ft
P1	t8	Polygonal	2.50	10.00	5.00	20.00

3.3. Stiffeners

ID	Label	Direction	Start X ft	End X ft	Start Y ft	End Y ft	Length ft	Rigid Support
S1	- Null -	Horizontal	0.00	5.00	0.00	0.00	5.00	Pinned
S2	- Null -	Horizontal	0.00	5.00	20.00	20.00	5.00	Lateral
S3	- Null -	Vertical	0.00	0.00	0.50	19.50	19.00	Sym
S4	- Null -	Vertical	5.00	5.00	0.50	19.50	19.00	Sym

3.4. Point Loads

Nodes ID	Load Case	Fx kips	Fy kips	Fz kips	Mx kip-ft	My kip-ft	Mz kip-ft	Ecc. in
N1	A	0.00	-10.02	0.00	0.00	0.00	0.00	2.70
	B	0.00	-4.50	0.00	0.00	0.00	0.00	2.70

3.5. Uniform Area Loads

Plate ID	Load Case	Wx psf	Wy psf	Wz psf
P1	C	0.00	0.00	-30.00

4. Results

4.1. Envelope

4.1.1. Plate Flexure Reinforcement

Coordinate System: Global

	Element	Curtains	Direction	Mu (x/y) kip-ft/ft	Nu (x/y) klf	Ld Comb.	As (x/y) in²/ft	Rho %	Tie
	191	1	Horizontal	0.48	-1.02	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	192	1	Horizontal	0.48	-1.02	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	193	1	Horizontal	0.48	-1.01	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	194	1	Horizontal	0.48	-1.01	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	195	1	Horizontal	0.48	-1.01	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	196	1	Horizontal	0.48	-1.01	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
at midheight	197	1	Horizontal	0.48	-1.01	1.2D+1.6Lr+0...	0.192	0.20	
(y = 10.00 ft)			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	198	1	Horizontal	0.48	-1.01	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	199	1	Horizontal	0.48	-1.02	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	200	1	Horizontal	0.48	-1.02	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.42	-5.07	1.2D+1.6Lr+0...	0.269	0.28	
	201	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.44	-5.01	1.2D+1.6Lr+0...	0.269	0.28	
	202	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.44	-5.01	1.2D+1.6Lr+0...	0.269	0.28	
	203	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
			Vertical	2.44	-5.01	1.2D+1.6Lr+0...	0.269	0.28	
	204	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	

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at midheight (y = 10.00 ft)

at critical section (y = 11.00 ft)

Element	Curtains	Direction	Mu (x/y) kip-ft/ft	Nu (x/y) klf	Ld Comb.	As (x/y) in²/ft	Rho %	Tie
		Vertical	2.44	-5.01	1.2D+1.6Lr+0...	0.269	0.28	
205	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.44	-5.02	1.2D+1.6Lr+0...	0.269	0.28	
206	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.44	-5.02	1.2D+1.6Lr+0...	0.269	0.28	
207	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.44	-5.01	1.2D+1.6Lr+0...	0.269	0.28	
208	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.44	-5.01	1.2D+1.6Lr+0...	0.269	0.28	
209	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.44	-5.01	1.2D+1.6Lr+0...	0.269	0.28	
210	1	Horizontal	0.49	-1.00	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.44	-5.01	1.2D+1.6Lr+0...	0.269	0.28	
211	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.95	1.2D+1.6Lr+0...	0.269	0.28	
212	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.95	1.2D+1.6Lr+0...	0.269	0.28	
213	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.95	1.2D+1.6Lr+0...	0.269	0.28	
214	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.96	1.2D+1.6Lr+0...	0.269	0.28	
215	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.96	1.2D+1.6Lr+0...	0.269	0.28	
216	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.96	1.2D+1.6Lr+0...	0.269	0.28	
217	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.96	1.2D+1.6Lr+0...	0.269	0.28	
218	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.95	1.2D+1.6Lr+0...	0.269	0.28	
219	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.95	1.2D+1.6Lr+0...	0.269	0.28	
220	1	Horizontal	0.49	-0.99	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.95	1.2D+1.6Lr+0...	0.269	0.28	
221	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.89	1.2D+1.6Lr+0...	0.269	0.28	
222	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.89	1.2D+1.6Lr+0...	0.269	0.28	
223	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.89	1.2D+1.6Lr+0...	0.269	0.28	
224	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.90	1.2D+1.6Lr+0...	0.269	0.28	
225	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.90	1.2D+1.6Lr+0...	0.269	0.28	
226	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.90	1.2D+1.6Lr+0...	0.269	0.28	
227	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.90	1.2D+1.6Lr+0...	0.269	0.28	
228	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.89	1.2D+1.6Lr+0...	0.269	0.28	
229	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.89	1.2D+1.6Lr+0...	0.269	0.28	
230	1	Horizontal	0.49	-0.98	1.2D+1.6Lr+0...	0.192	0.20	
		Vertical	2.45	-4.89	1.2D+1.6Lr+0...	0.269	0.28	

at midheight (y = 10.00 ft)

$\boxed{A_{s,avg} = 0.27 \text{ in.}^2}$

at critical section (y = 11.00 ft)

$\boxed{A_{s,avg} = 0.27 \text{ in.}^2}$

4.2. Service

4.2.1. Nodal Displacements

4.2.1.1. 1.2D+1.6Lr+0.8W

Coordinate System: Global

	Node	Dx	Dy	Dz
		in	in	in
	221	0.000	-0.001	-0.072
	222	0.000	-0.001	-0.072
	223	0.000	-0.001	-0.072
	224	0.000	-0.001	-0.072
	225	0.000	-0.001	-0.072
at midheight (y = 10.00 ft)	226	0.000	-0.001	-0.072
	227	0.000	-0.001	-0.072
	228	0.000	-0.001	-0.072
	229	0.000	-0.001	-0.072
	230	0.000	-0.001	-0.072
	231	0.000	-0.001	-0.072
	232	0.000	-0.002	-0.072
	233	0.000	-0.002	-0.072
	234	0.000	-0.002	-0.072
	235	0.000	-0.002	-0.072
	236	0.000	-0.002	-0.072
	237	0.000	-0.002	-0.072
	238	0.000	-0.002	-0.072
	239	0.000	-0.002	-0.072
	240	0.000	-0.002	-0.072
	241	0.000	-0.002	-0.072
	242	0.000	-0.002	-0.072
	243	0.000	-0.002	-0.071
	244	0.000	-0.002	-0.071
	245	0.000	-0.002	-0.071
	246	0.000	-0.002	-0.071
	247	0.000	-0.002	-0.071
at critical section (y = 11.00 ft)	248	0.000	-0.002	-0.071
	249	0.000	-0.002	-0.071
	250	0.000	-0.002	-0.071
	251	0.000	-0.002	-0.071
	252	0.000	-0.002	-0.071
	253	0.000	-0.002	-0.071

$D_{z,average}$ = 0.072 in.

$D_{z,average}$ = 0.071 in.

4.3. Ultimate

4.3.1. Plate Internal Forces

4.3.1.1. 1.2D+1.6Lr+0.8W

Coordinate System: Global

$N_{yy,avg}$ = 5.04 kips

$M_{yy,avg}$ = 2.43 kip-ft/ft

	Element	Nxx	Nyy	Nxy	Mxx	Myy	Mxy
		klf	klf	klf	kip-ft/ft	kip-ft/ft	kip-ft/ft
	191	-1.02	-5.07	0.00	0.48	2.42	0.00
	192	-1.02	-5.07	0.00	0.48	2.42	0.00
	193	-1.01	-5.07	0.00	0.48	2.42	0.00
at midheight (y = 10.00 ft)	194	-1.01	-5.07	0.00	0.48	2.42	0.00
	195	-1.01	-5.07	0.00	0.48	2.42	0.00
	196	-1.01	-5.07	0.00	0.48	2.42	0.00
	197	-1.01	-5.07	0.00	0.48	2.42	0.00
	198	-1.01	-5.07	0.00	0.48	2.42	0.00

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	Element	Nxx klf	Nyy klf	Nxy klf	Mxx kip-ft/ft	Myy kip-ft/ft	Mxy kip-ft/ft
	199	-1.02	-5.07	0.00	0.48	2.42	0.00
	200	-1.02	-5.07	0.00	0.48	2.42	0.00
	201	-1.00	-5.01	0.00	0.49	2.44	0.00
	202	-1.00	-5.01	0.00	0.49	2.44	0.00
	203	-1.00	-5.01	0.00	0.49	2.44	0.00
at midheight	204	-1.00	-5.01	0.00	0.49	2.44	0.00
(y = 10.00 ft)	205	-1.00	-5.02	0.00	0.49	2.44	0.00
	206	-1.00	-5.02	0.00	0.49	2.44	0.00
	207	-1.00	-5.01	0.00	0.49	2.44	0.00
	208	-1.00	-5.01	0.00	0.49	2.44	0.00
	209	-1.00	-5.01	0.00	0.49	2.44	0.00
	210	-1.00	-5.01	0.00	0.49	2.44	0.00
	211	-0.99	-4.95	0.00	0.49	2.45	0.00
	212	-0.99	-4.95	0.00	0.49	2.45	0.00
	213	-0.99	-4.95	0.00	0.49	2.45	0.00
	214	-0.99	-4.96	0.00	0.49	2.45	0.00
	215	-0.99	-4.96	0.00	0.49	2.45	0.00
	216	-0.99	-4.96	0.00	0.49	2.45	0.00
	217	-0.99	-4.96	0.00	0.49	2.45	0.00
	218	-0.99	-4.95	0.00	0.49	2.45	0.00
	219	-0.99	-4.95	0.00	0.49	2.45	0.00
at critical	220	-0.99	-4.95	0.00	0.49	2.45	0.00
section	221	-0.98	-4.89	0.00	0.49	2.45	0.00
(y = 11.00 ft)	222	-0.98	-4.89	0.00	0.49	2.45	0.00
	223	-0.98	-4.89	0.00	0.49	2.45	0.00
	224	-0.98	-4.90	0.00	0.49	2.45	0.00
	225	-0.98	-4.90	0.00	0.49	2.45	0.00
	226	-0.98	-4.90	0.00	0.49	2.45	0.00
	227	-0.98	-4.90	0.00	0.49	2.45	0.00
	228	-0.98	-4.89	0.00	0.49	2.45	0.00
	229	-0.98	-4.89	0.00	0.49	2.45	0.00
	230	-0.98	-4.89	0.00	0.49	2.45	0.00

$\boxed{N_{yy,avg} = 4.92 \text{ kips}}$

$\boxed{M_{yy,avg} = 2.45 \text{ kip-ft/ft}}$

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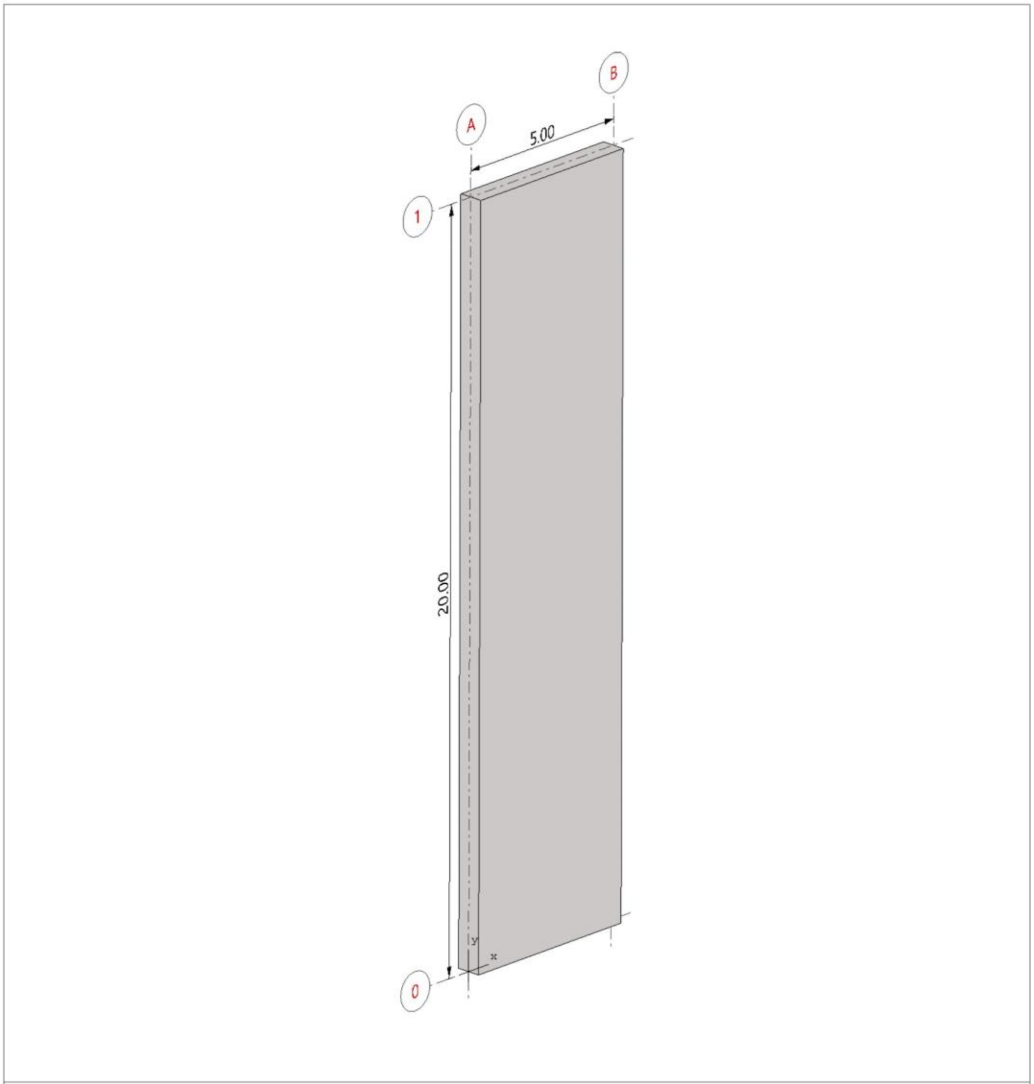
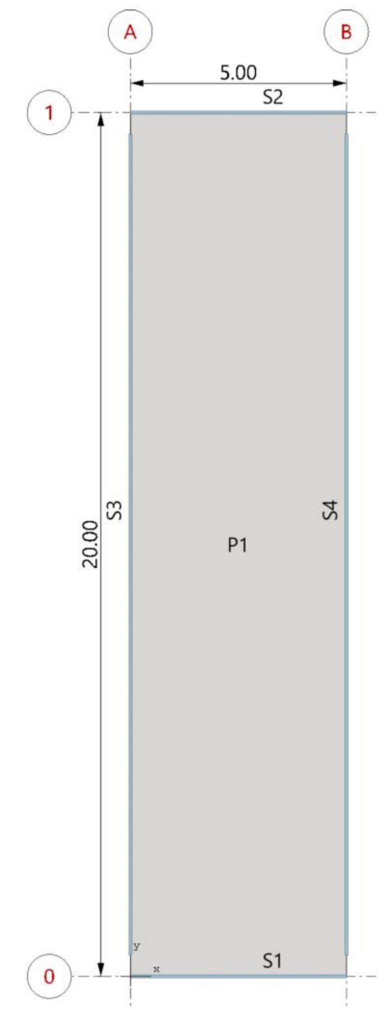
5. Screenshots

5.1. Extrude 3D view



Project:	Bearing Wall
Diagram:	Model View (Extrude)
	Plates

5.2. Plates & Stiffeners ID



Project: Bearing Wall

Diagram: Model View

Plates (ID); Stiffeners (ID)

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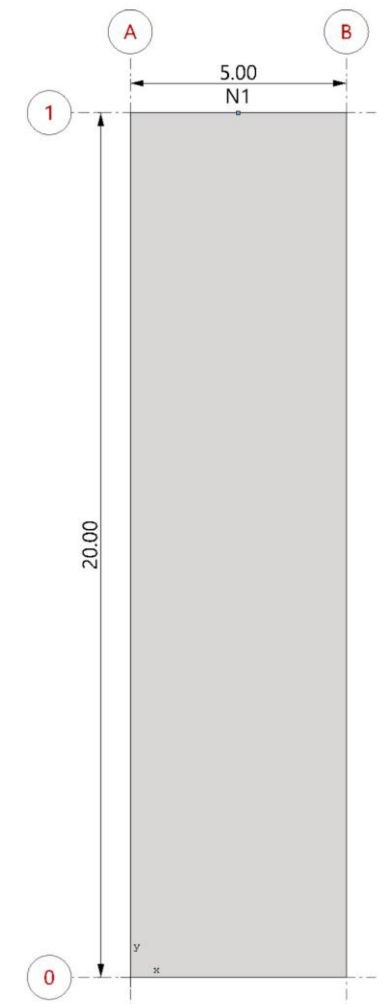
5.3. Nodes ID



Project:	Bearing Wall
Diagram:	Model View
	Plates; Nodes (ID)

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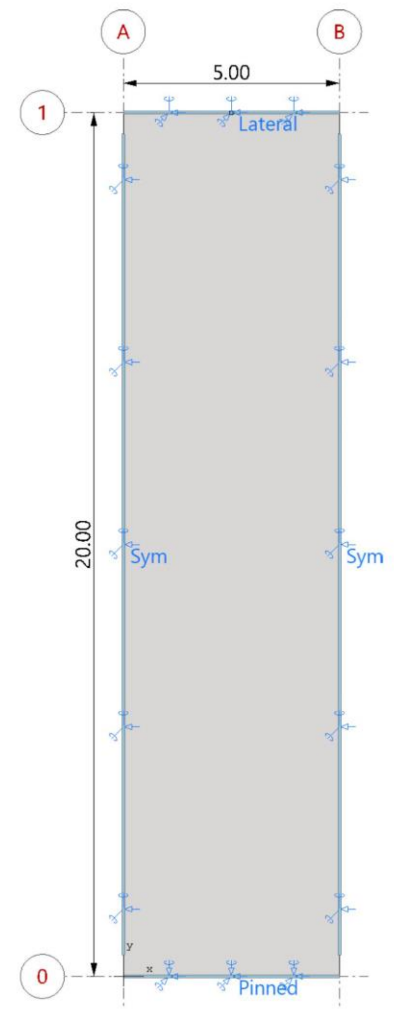
5.4. Restraints



Project:	Bearing Wall	
Diagram:	Model View	
	Plates; Nodes; Stiffeners; Restraints (Label)	

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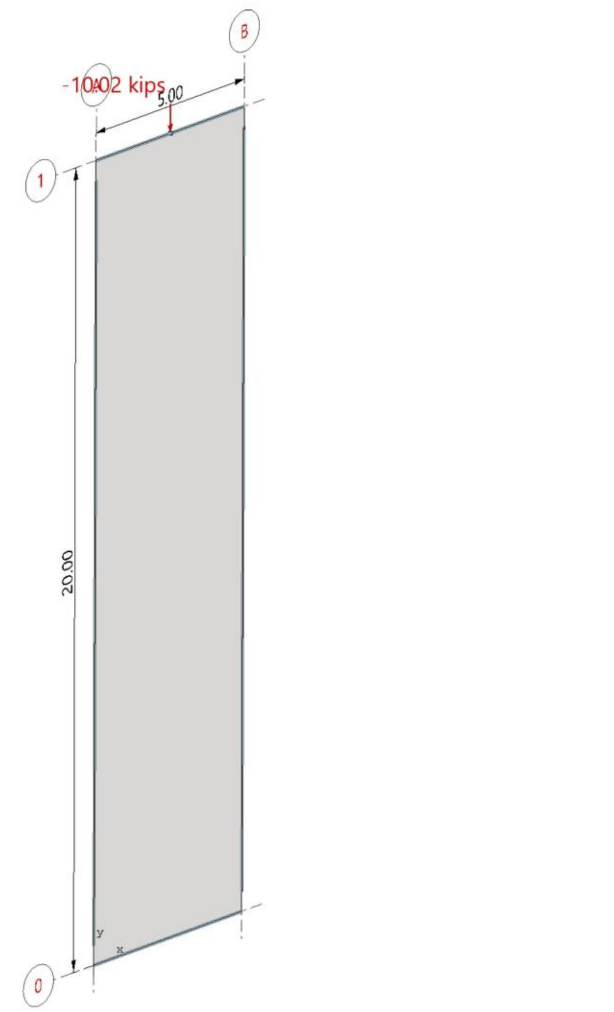
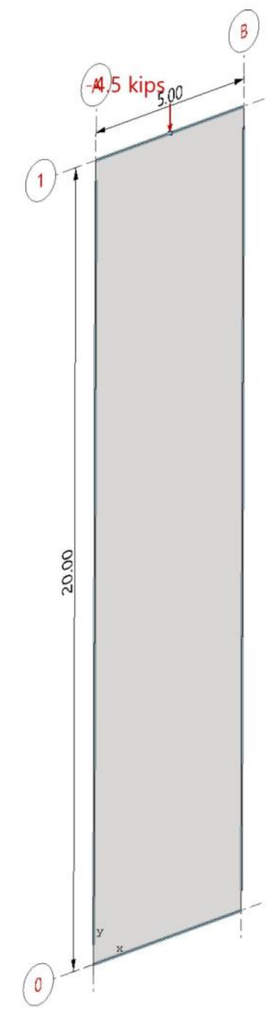
5.5. Loads - Case A - DEAD



Project:	Bearing Wall
Diagram:	Model View (Load Case: A - DEAD)
	Plates; Nodes; Stiffeners

5.6. Loads - Case B - LIVE



Project:	Bearing Wall
Diagram:	Model View (Load Case: B - LIVE)
	Plates; Nodes; Stiffeners

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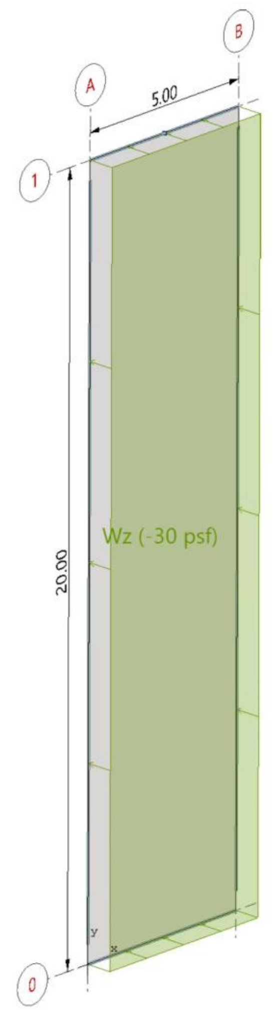
5.7. Loads - Case C - WIND



Project:	Bearing Wall
Diagram:	Model View (Load Case: C - WIND)
	Plates; Nodes; Stiffeners

10. Design Results Comparison and Conclusions

Table 3 – Comparison of Precast Wall Panel Analysis and Design Results (2nd Load Combination)				
Solution	M_u (kip-ft)	N_u (kips)	$A_{s,vertical}$ (in.^{2})	$D_{z,service}$ (in.)
Hand (at midheight)	2.45	5.04	0.27	0.072
spWall (at midheight)*	2.43	5.04	0.27	0.072
spWall (at critical section)**	2.45	4.92	0.27	0.071
* Values are taken at midheight (y = 10.00 ft) for comparison purposes with hand calculations. ** Values are taken at critical section (y = 11.00 ft) with maximum moment value.				

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

In column and wall analysis, section properties shall be determined by taking into account the influence of axial loads, the presence of cracked regions along the length of the member, and the effect of load duration (creep effects). ACI 318 permits the use of moment of inertia values of 0.70 I_g for uncracked walls and 0.35I_g for cracked walls.

ACI 318-14 (6.6.3.1.1)

In spWall program, these effects are accounted for where the user can input reduced moment of inertia using "cracking coefficient" values for plate and stiffener elements to effectively reduce stiffness. Cracking coefficients for out-of-plane (bending and torsion) and in-plane (axial and shear) stiffness can be entered for plate elements. Because the values of the cracking coefficients can have a large effect on the analysis and design results, the user must take care in selecting values that best represent the state of cracking at the particular loading stage. Cracking coefficients are greater than 0 and less than 1.

At ultimate loads, a wall is normally in a highly cracked state. The user could enter a value of out-of-plane cracking coefficient for plates of $I_{cracked}/I_{gross}$ based on estimated values of A_s. after the analysis and design, if the computed value of A_s greatly differs from the estimated value of A_s, the analysis should be performed again with new values for the cracking coefficients. To account for variations in material properties and workmanship, a factor of 0.75 can be used to reduce the calculated bending stiffness of the concrete section in accordance with ACI 318-14 Chapter 11.

At service loads, a wall may or may not be in a highly cracked state. For service load deflection analysis, a problem should be modeled with an out-of-plane cracking coefficient for plates of $I_{effective}/I_{gross}$.

Based on the previous discussion, the ratio between I_{cr} and I_g including a reduction factor of 0.75 is used as the cracking coefficient for the out-of-plane case for the ultimate load combinations. In this example, I_{cr} and I_g were found to be equal to 33.38 in.[4] and 512 in.[4] (Load Combination #2). Thus, the out-of-plane cracking coefficient for ultimate load combinations can be found as follows:

$$\alpha = \text{ cracking coefficient} = \frac{0.75 \times I_{cr}}{I_g} = \frac{0.75 \times 33.38}{512} = 0.04890$$

For the service load combinations, it was found that load combination #2 governs. M_a for this load combination was found to be equal to 22.15 in.-kips which is less than M_{cr} = 60.72 in.-kips indicating the section is uncracked ($I_{effective}$ = I_{gross}) and the cracking coefficient can be set to 1.0

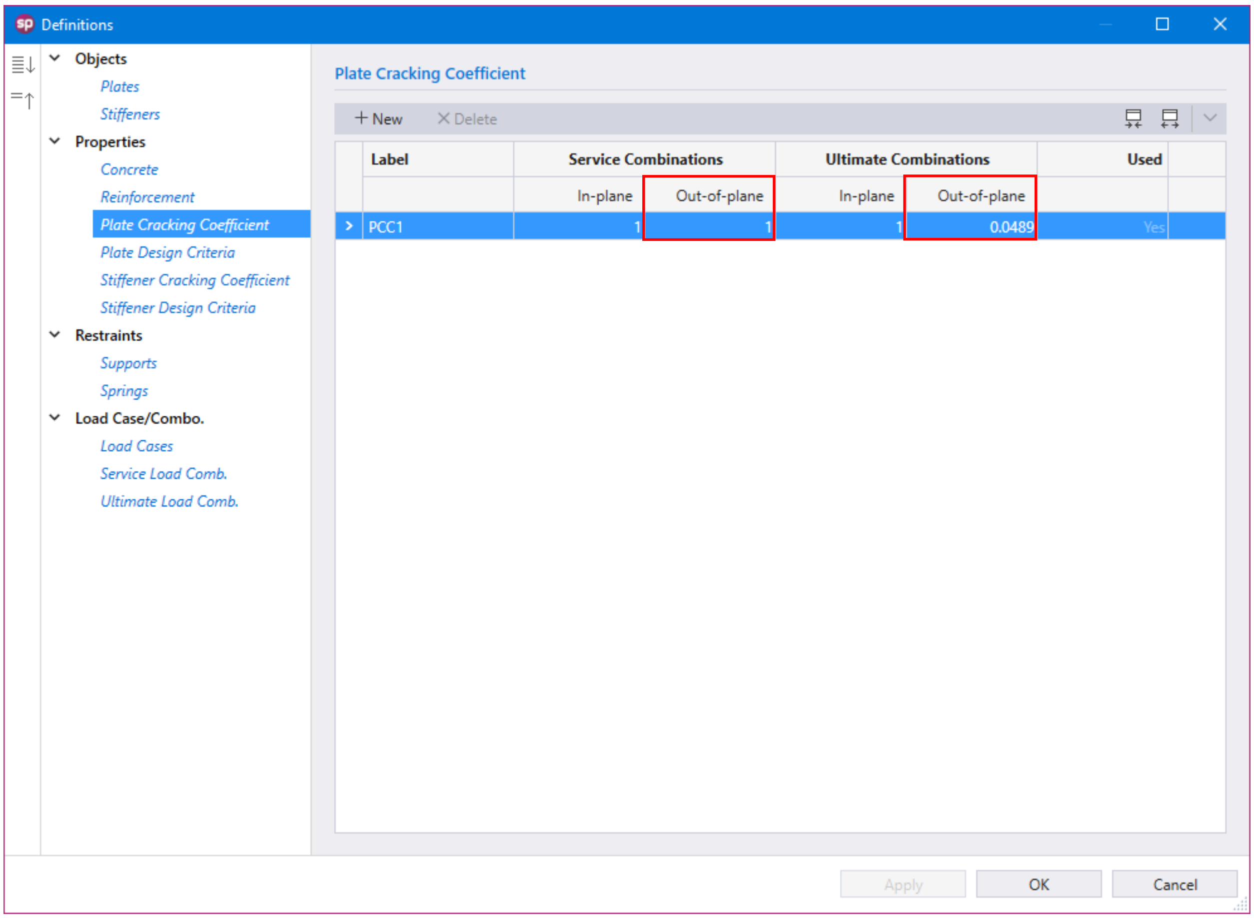


Figure 13 – Defining Cracking Coefficient (spWall)

In spWall, first-order or second-order analysis can be performed to obtain the design moment. In this model, the second order effects were included to compare the results with the hand solution results including the PΔ effects.

To further compare the program results with calculations above, the model was run again without the second order effects to compare the moment values with M_{ua}. Table 4 shows the results are also in good agreement.

Table 4 - Comparison of Precast Wall Panel First-Order Moments at midheight

Load Combination	M_{ua}, in.-kips	
	Hand	spWall
1.4 D	3.74	3.78
1.2 D + 1.6 L_r + 0.8 W	19.53	19.56
1.2 D + 0.5 L_r +1.6 W	32.61	32.64
0.9 D + 1.6 W	31.20	31.14

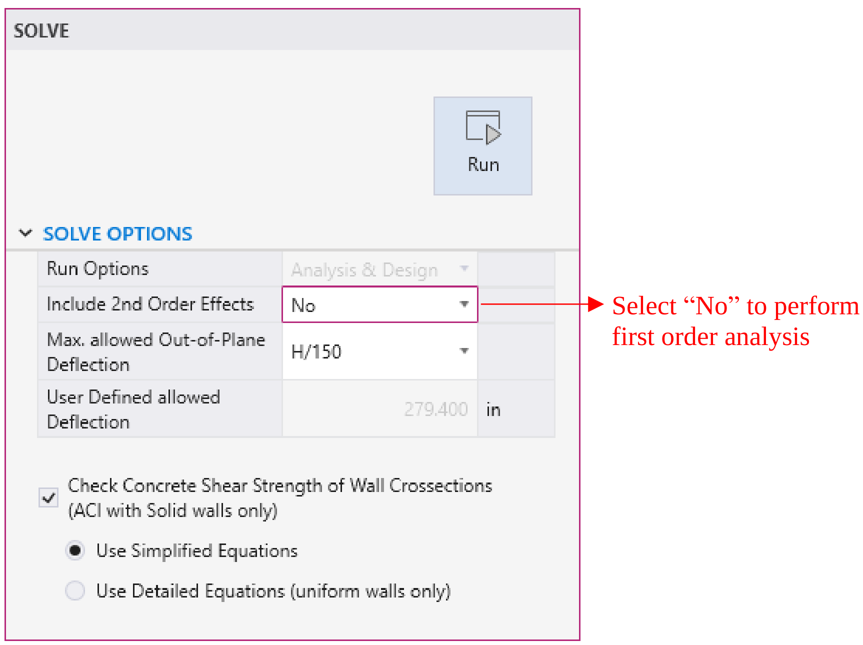


Figure 14 – Solver Module (spWall)